

A multilevel decision-making approach for the resilience assessment of industrial plants in seismic prone areas

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Summary

In the present work the problem of the seismic resilience of industrial plants, characterized by a sensitive interconnection among single facilities, has been investigated.

A brief introduction about the typical problem the industrial facilities are affected after a seismic event was provided. By this means it was possible to clarify the dimension of the problem and the extension, spatial and temporal, of the economic exposure industrial plants are affected. Then, the focus of the current method of analysis of industrial plants has been proposed, showing the differences among the vulnerability, risk and resilience. It was observed that: vulnerability methods are not suitable to provide information of the economic exposure; risk analysis can provide an insight about the economic exposure immediately after the seismic event, but does not provide any information about its temporal evolution; resilience analysis can fill the lack of information provided by the risk analysis, giving an overview of the temporal evolution of the economic exposure.

Notwithstanding the importance of the resilience analysis, a lack in the current scientific literature was observed, evidencing that only risk analysis is usually adopted, providing several methodologies, qualitative and quantitative as well.

Then, the aspect of the resilience analysis was deepened, specializing the general concepts for industrial plants. In particular the problem was faced from two sides. At first the behaviour of stand-alone facilities and the methods for the resilience assessment was analysed. Specific methods for the estimation of the consequences associated with the resilience analysis, i.e. the residual functionality $q(t)$, the recovery time t_{rec} and the recovery path, were defined. Therefore, the interest moved to inter-connected facilities, for which

the concepts of the reliability of systems are enriched with further aspects to make them applicable for the resilience assessment.

With such investigations, a detailed method, based on the PBEE framework, for the resilience assessment of industrial plants and facilities prioritization was proposed, filling the gap of current literature.

Notwithstanding the evident complexity to perform detailed analyses on a huge set of facilities, variously connected among themselves, could be too cumbersome, a multilevel procedure is proposed. The aim of such a procedure is to provide a global, but rough, overview of the results at the first level, performing simplified analyses (for the definition of the structural performances and the estimation of consequences), and specializing the results at the next levels by means of detailed analyses targeted on few critical structures. The multilevel procedure is intended to be a tool that simplifies the detection of critical elements within the whole Plant, performing on them detailed assessment.

The procedure is based on three levels:

- the *first level* is based on the execution of simplified analyses at the plant level; for the estimation of consequences a specific method, based on a tabular format is developed;
- the *second level* focuses on refining the structural performance and the estimation of consequences of critical facilities;
- the third level provides a specific definition of the assessment of the structural performance of the critical details, providing the effectiveness on the plant resilience of possible upgrading solutions, that aim at reducing the vulnerability, operating on the structure, or increasing the resilience, enhancing the method of interventions after the seismic damage.

Each level provides also a prioritization of facilities, based on the concept of Resilience Indicators (RI). The Resilience Indicators, from a practical perspective, scales the consequences of each facility to achieve an optimal resilience condition decided by the owner of the plant and defined through the break even point.

The Resilience Indicators allow to establish which facilities require in deep assessment from one step to the next.

Two case studies, representative of two actual industrial realities, a Steel Mill and a Chemical Plant, were carefully selected to apply and validate the proposed multilevel procedure. The Steel Mill is characterized by a huge in plan extension, and is characterized by drift sensitive facilities, easily to schematize as single degree of freedom structures. The Chemical Plant has a compact configuration that develops over the height; it is characterized by a

main braced building that support several vessels, mainly sensitive to peak floor acceleration.

The results of the detailed resilience assessment, applied on both the case studies, are used as a reference to validate the multilevel procedure.

The multilevel procedure provided results that well fit the detailed resilience assessment, regarding both resilience curve and the facilities prioritization. Furthermore, the convergence of results was achieved with a reduced number of steps.

Clearly, the application of the multilevel procedure strongly reduced the amount of detailed analyses, optimizing the in-field surveys and simplifying considerably the achievement of satisfactory results in terms of Plant's resilience.

List of Symbols and Abbreviations

ADRS	Acceleration Displacement Response Spectrum
a_g	peak ground acceleration
β	logarithmic standard deviation
CBF	Concentrically Braced Frame
CIF	Critical Industrial Facilities
CCPS	Center for Chemical Process Safety
CMS	Conditional Mean Spectrum
C.O.V.	Coefficient Of Variation
CR	Complete Reconstruction
DL	Damage Limitation
DS	Damage State
EAL	Expected Annual Losses
$EAL_{s,i}$	Expected Annual Loss associated with the i^{th} facility
EAL_p	Expected Annual Loss of the whole Plant
EAL_{iso}	Expected Annual Loss in the <i>iso-resilience</i> condition
EAL_{cum}	Cumulative Expected Annual Loss
EMS	European Macroseismic Scale
EPRI	Electric Power Research Institute
EDP	Engineering Demand Parameter
$F_{a,max}$	maximum horizontal force accounting for the seismic amplification over the height, according to VCI Guidelines
FRS	Floor Response Spectrum
FSD	First order Stochastic Dominance
γ	importance factor
g	acceleration of gravity

GA	Genetic Algorithm
IDA	Incremental Dynamic Analysis
IM	Intensity Measure
I_v	Vulnerability index
λ	mean annual frequency of the seismic action
LPG	Liquified Petroleum Gas
LP-HC	Low Probability-High Consequences
LR	Loss of Resilience
LS	Limit State
MAF	Mean Annual Frequency
MECE	Mutually Exclusive and Collectively Exhaustive
MRF	Moment Resisting Frame
NC	Near Collapse
P_f	Failure Probability
P_{f,RS_i}	probability of exceedance of the i^{th} resilience state
P_{LS_i}	Probability of exceedance of the i^{th} limit state
$q(t)$	Residual functionality
QRA	Quantitative Risk Assessment
\mathcal{R}	Reconstruction schedule
R	Resilience
RC	Reinforced Concrete
REDI	Resilience-base Earthquake Design Initiative
RI	Resilience indicators
RS	Resilience State
$\hat{\sigma}$	Standard deviation
\mathcal{S}	Network system
$S_a(T_1)$	spectral acceleration computed in correpondance of the period T_1
SD	Severe Damage
SRC	Standard Regression Coefficient

SSD	Second order Stochastic Dominance
T_a	Fundamental period of non-structural element
$t_{ave,i}$	Average recovery time of the i^{th} facility
t_{BEP}	Break even point
t_{ev}	Time at which a damage is triggered
T_{insp}	Time for the inspection
T_{LC}	Reference time
T_{sm}	Time for the introduction of safety measures
T_D	Time for the design of the intervention
T_R	Time for demolition and econstruction
t_{rec}	Recovery time
T_{rep}	Time for the replacement
UHS	Uniform Hazard Spectrum
VaR	Value at Risk

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Chapter 1

Statement of the problem

1.1 Introduction

Industrial plants are constituted of heterogeneous and complex facilities, embedded within physical networks and logical or physical interdependencies, in continuous evolution to keep up with the technological improvements.

Recent earthquakes (Kocaeli 1999, Fukushima, 2011, Emilia-Italy 2012) (Barka (1999), Buesseler et al. (2011), Krausmann and Cruz (2013b), Liberatore et al. (2013b), Sezen and Whittaker (2006) (see figures 1.1) and 1.2) and Grimaz et al. (2010)) highlighted the high seismic risks associated with industrial plants, showing that the size of the consequences might exceed the cost of the plant itself, resulting difficult to be sustained under the economic aspects. In fact, being the main characteristics of industrial plants the dense lay-out, the strong interdependencies among structures, and the presence of hazardous materials, a localized damage might spread over the whole plant, causing, next to the downtime of the process, wide losses at social and environmental level as well. In this context, specific models for the risk analysis of industrial plants results mandatory.

For the sake of clarity, a detailed overview of the Fukushima event is provided in the following to show the escalating scenario. In particular, the analysis is performed in terms of damage and failure mechanisms, socio economic impact and cascading effects.

The Industrial Plant mainly damaged has been the Cosmo Oil refinery. At the site of the refinery, PGA values of 0.114g were measured during the main shock and 0.099g during the aftershock (Krausmann and Cruz (2013b)).



Figure 1.1: The LPG tank farm at the Chiba refinery after the earthquake-triggered fires and explosions.



Figure 1.2: Collapse of the piperacks at the Tupras refinery after the Kocaeli earthquake.

Damage and failure mechanisms. The Cosmo Oil refinery includes a storage depot for liquefied petroleum gas (LPG) which is a highly flammable substance. LPG storage tanks are big pressurised spheres, held in place by legs supported by diagonal braces. Unknown to the refinery operator, the first earthquake shock damaged the braces on an LPG tank that at the time of the earthquake was filled with water for maintenance reasons. The second shock caused the weakened braces of the tank to buckle and subsequently the tank to collapse. This ruptured the LPG pipes connected to the tank and led to the release of flammable gases that might ignite (Cosmo Oil (2011a); Krausmann and Cruz (2013b)). Due to heat impingement by the fire on the other storage tanks, they ruptured via boiling liquid expanding vapour explosions (BLEVE). Overall, 5 major explosions were recorded.

Socio-economic impact. The fires and explosions at the Cosmo Oil refinery led to the evacuation of 1,142 residents. At the refinery, 6 injuries (1 severe, 5 minor) were recorded while at a facility adjacent to the LPG tank farm, where a fire was triggered via a domino effect, 3 persons suffered injuries, one of them severe (Krausmann and Cruz (2013b)). The explosions were so violent that pieces of tank insulation and sheet metal were found at a distance of over 6 km from the refinery's tank farm. In nearby residential areas, windows and cars were damaged (Cosmo Oil (2011a)). The fires at the refinery caused Cosmo Oil to declare a loss on disaster of over 5.7 billion ¥ (72 million US\$) for fiscal year 2010, which ended on 31 March, 2011 (Cosmo Oil (2011b)). For fiscal year 2011, the company reported a net deficit of 9.1 billion ¥ (114 million US\$) which was mostly due to the suspension of activities at the Chiba refinery and associated alternative supply costs (Cosmo Oil (2012)). Overall, the refinery suffered a 2-year business interruption due to the damage caused by the Tohoku earthquake (Cosmo Oil (2013)).

Cascading effects. The fire caused by ignition of LPG from the broken pipes propagated to the other pressurised tanks which suffered BLEVEs. The explosions damaged nearby crude oil tanks due to debris impact, causing asphalt to leak into the ocean (Cosmo Oil (2011a)). In addition, dispersion of flammable LPG vapours and burning missile projection from the explosions started fires in two neighbouring chemical facilities, with subsequent hazardous-materials releases (Krausmann and Cruz (2013b)).

Such a disproportionate collapse is imputable to the existing approaches adopted for the design of Industrial facilities. In fact, existing industrial plants are usually designed to optimize the aspects related with the production process. Particular attention is usually placed on the design of component to reduce mechanical failures and the effects associated with cascading events, e.g. explosions, fires, release of toxic materials, which damage extent to neighbor facilities, workers and environment might be difficult to manage and restore. To this aim, several guidelines have been developed and published about this field (Lees (2012), CCPS (1996), CCPS (2012) and CCPS (2013)), in order to provide a comprehensive and accurate enough methodology for the application of the quantitative risk assessment (QRA). Recognizing the importance to perform refined analyses, such guidelines evidence the need to assess industrial plants through risk-based methodologies, that focus on the estimation of the exposure, usually expressed in economic terms, that a system exhibit immediately after being damaged from a certain threat. Unfortunately, such approaches have two main drawbacks:

- the lack of data to model the exposure, transform risk-based procedures to vulnerability-based procedures, in which only the damage, expressed in terms of reduction of the structural capacity of the bearing system of the facilities, is obtained; with such approach, the element associated with the highest failure probability is the one that requires greater attention for mitigation measures;
- the apparent protraction of the consequences over the time leads to a total estimation of the economic exposure much more sensitive than the immediate one. From this perspective, the analysis of the resilience that incorporate within its main framework the assessment of the evolution of the exposure, from the triggering to the damage to the restoration of the full capacity of the system, is a more suitable approach.

Considering the interest to investigate the exposure an Industrial Plant is subjected after a seismic event, different approaches exists, see figure 1.3.

Vulnerability analysis is suitable to identify the structural damage due to the seismic action. When the focus is to assess the safety of people/workers, vulnerability analysis, supported by Standards of proven validity (EN 1990, 1990, Norme Tecniche per le Costruzioni (2008), EN 1998-3 (2005)) provides comprehensive and satisfactory results. The validity of the results obtained through the vulnerability analysis depends upon the reliability of the knowledge of the geometry and details of structural and non-structural elements obtained from surveys and the analysis of technical drawings. Anyway, vulnerability analysis cannot provide any information about economic exposure, associated, for instance, with the cost of replacement of damaged elements.

For the purpose to define the economic exposure to a seismic event, **risk analysis**, starting from the results of vulnerability analysis, can provide a link among the structural damage and the economic exposure, expressed in terms of monetary losses or percent of the total reconstruction cost. If the risk analysis is performed considering different levels of the seismic action a loss curve can be depicted, providing an effective overview of the exposure in function of the seismic action. The application of the risk analysis requires, along with a detailed knowledge of the system analysed, a precise identification, often in a probabilistic fashion (Deierlein et al., 2003a) of the decision variables chosen for the exposure estimation. However, risk analysis does not provide any information about the evolution of the exposure.

Resilience analysis aims at providing an overview about the temporal evolution of the exposure/consequences from the seismic event to the restoration of the safety and full operational condition of the system. Along with the information required to perform a risk analysis, further details about the capacity of the system to face a damage and intervene to reduce the consequences should be known. From this perspective, the resilience analysis is

clearly multidisciplinary. Several authors faced the problem and provided a framework of general validity for the application of the resilience analysis (Bruneau et al. (2003b) and Bruneau and Reinhorn (2004)), but, due to the huge amount of required variables, few practical and effective application are provided (Burton et al. (2015) and Cimellaro et al. (2010b)).

Observing the economic value of the system over the time (see figure 1.3), the vulnerability cannot provide any information about the exposure, immediate or delayed over the time. Risk analysis can provide only the exposure in the immediate instant of the seismic event. Resilience analysis can provide the temporal evolution of the exposure. Still looking at the aforementioned plane, the temporal evolution of the exposure is represented by a continuous curve, and the underlying area represents the Resilience of the system.

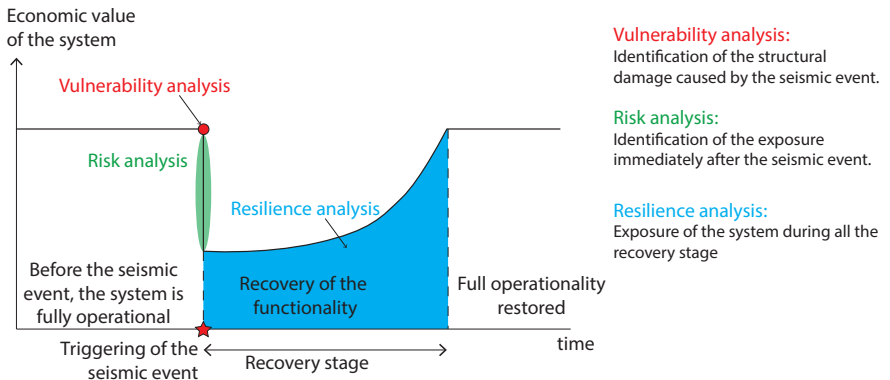


Figure 1.3: Schematic representation of vulnerability, risk and resilience.

Since the methods for the vulnerability assessment are well known and fully integrated within current Standards for the structural design and assessment of civil construction (Norme Tecnica per le Costruzioni (2008), EN 1998-3 (2005)) and considering the lack of reliable methodologies for the application of the resilience analysis of industrial installations, in the following the methods that can be currently adopted for the risk assessment of industrial plants are described in detail.

1.2 Risk assessment of industrial plants: state of the art

The Earthquakes of Fukushima (Japan, 2011), Izmit (Turkey, 1999) and Emilia Romagna (Italy, 2012) (Barka (1999), Buesseler et al. (2011), Krausmann and Cruz (2013b), Liberatore et al. (2013b), Sezen and Whittaker (2006) and Grimaz et al. (2010)) highlighted the weaknesses of industrial plants towards the seismic action, showing that the size of the consequences might exceed the cost of reconstruction of the plant itself, resulting difficult to be sustained under the economic aspects. Due to the presence of hazardous materials (hazmat) stored within the plant and the strong interdependencies among facilities, the localized damages caused by the earthquake might produce several additional scenarios, i.e. fires, explosion, toxic or radioactive dispersions, triggering disproportionate collapses and the inability to restore both the functionality of the entire plant and the safety of people in the surrounding area within a tight time interval. As consequence, seen the complex configuration of industrial installations, proper risk assessment methodologies have been developed through the past years.

In general, the codes (Seveso I (1982), Seveso II (1997), Seveso III (2003)) that regulate the safety of industrial plants, provide strict regulations predominantly for critical industrial facilities (CIF), defined on the basis of the amount of hazmat processed. The safety assessment usually focuses on the concept of societal risk acceptance criteria, developed through the ALARP (As Low As Reasonably Possible) principle (Swuste et al. (2016)) (see figure 1.4): in the space that relates the number of fatalities with the failure probability (PoE) of the accidental events three regions are defined, related with the totally acceptance of risk (low number of fatalities and/or very low (PoE)), the limit of acceptance and the unacceptable risk region. The typical ranges of component's mechanical failure probability are provided by technical literature (De Haag and Ale (2005), Lees (2012) and Van Den Bosh and Weterings (1997)) and are based on statistical inference on a huge number of real cases, properly monitored and reported, in order to use only valid data.

Within this perspective, the seismic assessment of industrial facilities has been usually confined to vulnerability assessment, considering each building as a standalone element disconnected from the whole system. The vulnerability assessment provides, as an outcome, the detection of the critical structural components that might yield to the global or local failure of the structure, when subjected to the seismic action. The outcomes are usually expressed in terms of maximum seismic intensity that the facility can withstand, or dealing with probabilistic framework (Pinto et al., 2004), in terms of failure probability. The latter outcome can be used in connection with event tree or fault tree models, to depict the failure probability of intercon-

nected facilities. Only in the 2012 the the Seveso III (2003) Directive imposed as compulsory the seismic assessment of CIFs within the risk assessment protocol typical of industrial plants.

Risk assessment procedures are commonly based on probabilistic frameworks, providing results exploitable for a decision-making process (ISO 31000 (2009), ISO 31010 (2010), Aven (2011)) usually finalized for the optimization of the economic resources to use for the improvement of the performance of the assessed system. The typical framework of risk assessment requires the estimation of three quantities:

- the threats the system can be subjected to, such as natural events (Douglas, 2007) or the effect of aging and corrosion on the structural integrity; in a probabilistic perspective, threats are expressed in the form of hazard curve, relating the intensity of the threats and its probability of exceedance (PoE);
- the definition of the weaknesses (vulnerability) of the system toward the identified threats;
- the exposures (consequences) the system is subject to when a specific damage events occur.

The convolution of the three quantities, typically yield to a loss curve, that express the losses in function of the PoE (or the mean annual frequency,

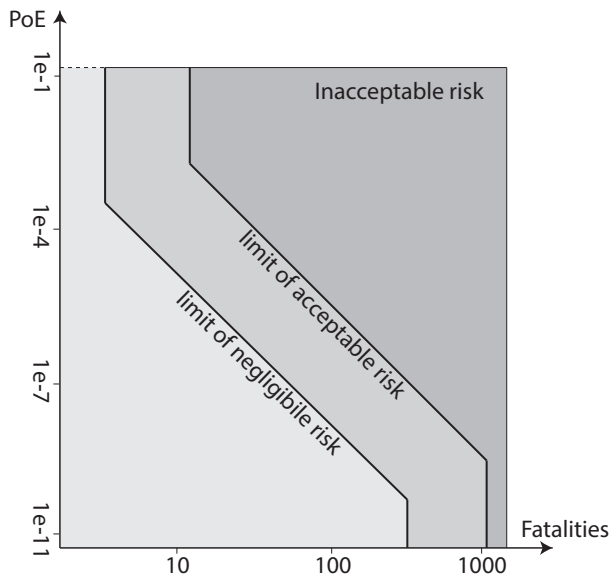


Figure 1.4: Example of ALARP state space.

MAF) of the threat event. Dealing with seismic risk assessment, is a common rule to classify losses in two main classes (Hamburger et al., 2012): direct losses, linked with the damage of structural and non-structural element as well, and indirect losses, that refers to loss of human lives, environmental damage or the downtime of the facility. Furthermore, the extent of the losses can be explained with synthetic scalar risk metrics (see figure 1.5). According to Yoshikawa and Goda (2013), the following scalar risk metrics are of particular interest:

- the EAL, Expected annual losses (Porter et al. (2004), Welch et al. (2014)), that expresses the losses as cost per year over the entire nominal life of the facility,
- the VaR (Value at Risk) (Artzner et al., 1999), representative of the losses in correspondence of a PoE usually fixed to 10^{-4} or 10^{-3} , and
- the Probable Maximum Losses (PML), that usually refers to the 90% quantile of losses within a return period of 475 years.

Their general validity makes them suitable for the application of any kind of the above-mentioned losses.

In the field of seismic risk assessment several approaches are available in the scientific literature. Starting from general validity models, customized procedures for the industrial plants have been developed. The approach focused on the evaluation of existing procedures can be either qualitative, ori-

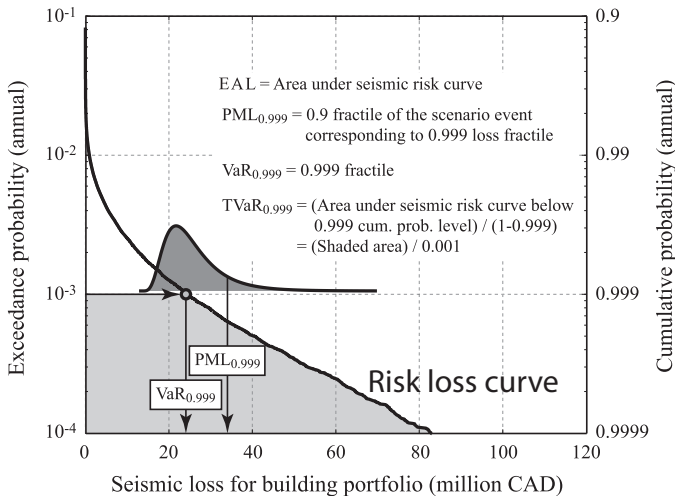


Figure 1.5: Loss curve and identification of the risk metrics (from Yoshikawa and Goda (2013)).

ented towards the assessment of large areas and rather fast to apply, or quantitative, oriented towards the detailed assessment of stand-alone facilities.

Qualitative procedures are characterized by fast procedures of general validity, resulting a suitable tool for the rapid assessment of large building portfolios. Obviously, providing only results of general validity, qualitative procedures cannot be exploited for building-specific assessment.

Quantitative procedures, on the contrary, are oriented on the detailed assessment of single facilities, considered as standalone elements disconnected from the context within they are embedded. The highly detailed analyses required for such procedures, being usually strongly time consuming, do not fit with the assessment of extended portfolio of facilities.

At the international level, the EPRI published two reports (EPRI (2000) and EPRI (2013)) focused on the seismic risk assessment of critical nuclear power plants through a quantitative procedure. The two reports provide a procedure tuned on the basis of the Performance Based Earthquake Engineering (PBEE) model (Deierlein et al. (2003b), Pinto et al. (2012), Fardis (2010) and Günay and Mosalam (2013)), developed by the Pacific Earthquake Engineering Research (PEER) center. Thereby, starting from a specific seismic hazard assessment (Cornell (1968), Douglas (2007), Kohrangi et al. (2017) and McGuire (1995)), the results of the vulnerability assessment conducted on all critical facilities and component are gathered within a fault tree model representative of the physical and logical dependencies within the plant. As outcome, the procedure yields the expected failure probability of the plant subjected to the seismic action, without providing an explicit estimation of the expected losses. For the purpose of the EPRI approach, the vulnerability assessment must provide fragility curves, expressing the PoE of a certain limit state at different values of seismic intensity. The most common and reliable approach for the development of fragility curves is represented by incremental dynamic analyses (IDA) (Vamvatsikos and Cornell, 2002), for which purpose the selection of a suitable set of natural accelerograms is required. Several methods for the selection of spectrum compatible natural accelerograms are available, depending on the expected outcome of the analyses. Recently Koharangi et al. (Kohrangi et al. (2017), Kohrangi et al. (2016a), Kohrangi et al. (2016b)) proposed an alternative procedure for the selection of natural accelerograms, adopting the Conditional Mean Spectrum (Baker, 2010 and Baker and Lee (2017)) built upon the average spectrum acceleration. The authors showed that the proposed approach yields fragility curves essentially independent from the site characteristics and thereby most representative of the effective seismic capacity of the facility. It should be noted that for structural typologies which hysteretic behaviour is known, nonlinear static procedures, much less time consuming than time histories analyses, can be adopted for the development of reliable fragility curves (D'ayala

et al. (2014), Fajfar (2002), Rossetto and Elnashai (2005), Vamvatsikos and Cornell (2005)).

Along with the quantitative procedures, a huge effort has been spent on the development of the qualitative procedures. Within this topic, several authors worked on the classification of industrial plants and facilities. Ciampi et al. (2004) provided a classification of industrial plants in three families, distinguishing among mechanical plants, power plants and process plants.

Mechanical plants, to whom heavy carpentry workshops, machinery factories, servicing lines belong to, are usually characterized by the use of large and heavy machines. The typical consequences caused by earthquakes are associated with localized damages to the plant itself. The release of dangerous materials is rather unlikely and limited to flammable product, that, producing BLEVE and fireballs (Van Den Bosh and Weterings, 1997), are associated with localized cascading effects (Reniers and Cozzani, 2013).

Power plants are responsible of the production of electrical energy using fossil or nuclear fuels as the primary source. The risk of dangerous releases toward the environment is strongly dependent on the fuel used. In thermal power plants using liquid or gaseous fuels the damage of equipment or piping determines the release of significant amounts of flammable material so that serious fires can develop; the consequences of such events remain however local. In the nuclear power plants any damage to the reactor, which is the central element of the plant, could lead to the release of radioactive materials which can easily contaminate a very large extension of territory; for these reasons, since the very beginning of adoption of the nuclear source, the construction of power plants has been accompanied with serious studies of seismic vulnerability.

All the installations performing chemical and/or physical chemical transformations of the materials belong to the last category of **Process plants**. The typology of equipment is very heterogeneous and complex and is characterized by a large variety of specific configurations, operative conditions and substances processed. It is then not easy to work out a simple scheme of the typologies of equipment belonging to this class, because they vary from case to case, depending on the process parameters required: different pressures, temperatures, masses of substances and, sometimes, the possibility of obtaining the same final product by different sequences of operations, determines different equipment and different layout arrangements.

Focusing on the classification of stand-alone facilities, the IPENZ regulation (Newmark and Hall, 1969) classified industrial buildings in two categories: "building like structures", having a structural configuration similar to typical civil use buildings, and "non-building like structures", to whom rigid structures, flat bottom tanks and support vessels belong to. Examples of industrial facilities are provided in figure 1.6.



(a) Building like structures.



(b) Horizontal vessels.



(c) slim vessels.



(d) Squat equipment.

Figure 1.6: Example of industrial facilities.

In their works, Paolacci et al. (2012) enriched the classification proposed by the IPENZ, defining five sub-classes of “non-building like structures”, broadly recognized by Fabbrocino et al. (2007): slim vessels, above ground squat equipment, squat equipment supported by columns, piping systems and support structures, slender industrial structures, squat equipment and supporting structures.

Hoffmeister et al. (2011) approached the problem to the classification of industrial facilities from a different perspective. Considered the huge variability of structural typologies and equipment configuration, the authors proposed a set of classification criteria, in order to account for all the typical aspects characterizing the industrial facilities. Thanks to its general validity,

the approach is flexible enough to be applied to different facilities without losing the consistency of the results. A similar approach has been adopted for the development of the European Macroseismic Scale model (EMS 98, Grünthal, 1998), providing a classification of the seismic vulnerability of typical existing civil use structures. Such classification has been adopted to further develop several risk-consistent models for the assessment of existing facilities (Giovinazzi (2005) and Lagomarsino and Giovinazzi (2004)), joining the vulnerability classification of the EMS-98 with the losses recorded from past events (Kircher et al., 1997).

Qualitative procedures for the risk assessment of industrial plants usually deal with LPG plants, focusing on the geographical extension of the damages or losses as well as on the escalating scenarios of domino effects (Antonioni et al., 2009a). The RAPID-N model (Girgin and Krausmann, 2013) implements a probabilistic risk assessment methodology by a web-based software tool. With such a tool, only the critical structures and the physical interdependencies are accounted for. The input required parameters refers to topology of tanks, their content and their seismic fragility. For the latter a generally valid curves, derived from statistical regression of recorded damages from real events, are used. Requiring the same input data, Fabbrocino et al. (2005b) developed a procedure aimed at the definition of risk plot of areas characterized by constant probability of injuries or loss of human lives.

1.3 Open problems

Nowadays, the seismic assessment of industrial plants is usually carried out through the evaluation of the plant vulnerability, or in special cases, of the seismic risk.

The approaches based on the seismic vulnerability can just provide information regarding the maximum seismic action the structure can withstand, adopting either linear or nonlinear procedures. These approaches are very well defined also from point of view of current Standards and give a general idea of consequences in terms of structural and non-structural damage. On the other side, seismic risk procedures, gathering the seismic hazard and the seismic vulnerability with the economic exposure, shall provide an overview of the monetary losses the structure can be subjected to. Therefore, the seismic risk evaluation comprises the vulnerability assessment and besides the information on the structural non-structural damage, supplies also an overviews of the economic exposure, direct (associated with structural and non-structural element damage) and indirect (related with downtime or unsafe placarding).

The applicability of the PBEE framework has to frequently face off with lack of available data to model the economic exposure, leading to simplifying the problem to code based procedures: the structural vulnerability, expressed in terms of failure probability, is directly linked to generally valid and predefined values, usually representative of the loss of human lives (Sinković et al., 2016). On the other side, *qualitative procedures* lack in providing detailed results, resulting too general to be adopted in detailed decision-making procedures.

Otherwise, when reliable exposure models are available, the economic losses can be explicitly calculated. In this case, most of the approaches, as the one adopted within the STREST project [P4], focus on **low-probability high-consequences** (LP-HC) events, characteristic for catastrophic events, **cutting out high-probability (HP) events** from the problem. This approach, consistent with the actual requirements of insurance companies and stakeholders, is adequate for built environment and infrastructural systems that fulfill the modern seismic regulations. However, HP events might cause consequences representative of a substantial portion of the average annualized losses, commonly expressed through the EAL parameter. This is particularly true when existing facilities, whose structural capacities do not comply with actual seismic regulations, are the focal point of the risk assessment.

Focusing on the procedures specific for industrial plants, **current methodologies** of general validity exist for the **risk assessment associated with mechanical failures** of single industrial components. On the contrary, only few procedures are available for the seismic risk assessment of industrial plants. Moreover, they are specifically tuned for major hazard LPG plants (Antonioni et al., 2009b and Fabbrocino et al., 2005a), so that they clearly cannot be applied to other typologies of industrial installations.

Current technical literature provides extremely detailed guidelines to perform a quantitative risk analysis of industrial plants, considering the mechanical failure of component and equipments as a principal hazard (Lees (2012), CCPS (2013), CCPS (2012)). Technical offices of Industrial Plants often recognized the invaluable validity of the former guidelines, adopting safety measures and emergency plans perfectly tuned to face off every kind of mechanical failure the Plant is vulnerable to. Anyway, the structural performance toward the seismic action of the facilities is frequently neglected, providing to it a reduced importance than equipment and components. It seems that such importance hierarchy is governed by as simple as dangerous thought: the higher the added economic value of the production the higher is the importance, disregarding that the structural failure might lead to eco-

conomic losses, direct and -obviously- indirect¹, far more important than a mechanical failure that can be solved in few days.

Along the above-mentioned aspects, the definition of a comprehensive global approach for the risk assessment, capable to properly correlate the typical aspects of an industrial plant and, at the same time, to provide results easy to use by the staff in charge of risk management, is complicated by three further aspects:

1. Industrial facilities are completely different than civil use ones, meaning that also the typical source of vulnerability are totally different. As a consequence, existing qualitative procedures, based on the prior knowledge of the structural vulnerabilities, cannot be applied, for most of the cases, to industrial building portfolios.
2. The current lack of knowledge associated with the seismic performance of industrial component, makes difficult the definition of consistent limit states to be exploited in seismic risk assessment. Thereby the actual procedures, with particular reference to the quantitative ones, are hardly applicable to industrial facilities.
3. Seismic damage to industrial plants involves different dimensions of losses: indirect losses associated with the plant downtime along with environmental and social losses, in particular when hazardous materials are stored in the plant, usually dominate over direct structural losses. Thereby the time required to stem the losses and reduce them becomes of fundamental importance. Such awareness arose after recent earthquakes (Krausmann and Cruz (2013a), Liberatore et al. (2013a) and Grmaz et al. (2010)), which evidenced that seismic risk methodologies are inadequate when dealing with highly interconnected systems, as for the case of industrial plants, in which the failure of a component might lead to tremendous losses and cascading events, which effects have heavy rebounds over the time. Furthermore, the value of the facility/equipment is usually equal to a small ration of the gross annual profit.
4. Plant's owners tend to spend capital in the improvement of equipment, asking for a minimal control on structures in order to achieve the minimum required safety level. Such aspects cause a lack of knowledge about the behaviour of Industrial Plant, when subjected to seismic events, frequent or extremely rare as well. As a consequence, exposes the Plant's owner to tremendous economic risks:
 - a protracted shutdown of the plant can lead either to unbearable economic fines and to a devaluation of the final product within the European or global market;

¹ Associated with the shut-down or the necessity to rearrange the production flow to ensure the productivity of the Plant.

- the local failure of a facility, although it does not cause the complete interruption of the production process, might lead to the loss of workers' life, of invaluable importance, from the perspective workers' safety regulations and also for continuity of the production; in fact, the loss of an expert employer might mean a forced slowdown of the productivity.

For that last reason, a resilience-based procedure is more adequate than a risk-based one. In fact, while the risk assessment focuses on the losses "*measured*" at the instant of the damage event, the procedure based on the resilience assessment (Bruneau et al. (2003a), Cimellaro et al. (2010a), Tierney and Bruneau (2007) and Zobel and Khansa (2014)) account for the evolution of the losses from the triggering of the threat to the restoration of the safety and full functionality. For such purpose, the proactive restoration measures the industrial plant can offer during the emergency phase, play a fundamental role on the resilience assessment.

In a perspective of limited economic resources, when the main interest is to identify "*few*" elements, the improvement of which increase the seismic performance of the whole plant, a resilience-based framework for the seismic assessment of industrial plants should be able to deal with all the main involved variables.

Considering that Industrial Plants are usually characterized by a consistent amount of inter-connected facilities, the application of detailed analyses on the whole Plant is unthinkable. On the other side, qualitative approaches cannot be applied on the plant, if their results are intended to be applied in a subsequent decision-make stage.

From this perspective, the adoption of a procedure based on multi-level of increasing complexity framework, surely ease the assessment of the seismic resilience of industrial facilities. In fact, the first levels could be based on a simplified approach, requiring a "*rough*" definition of the variables, aiming at providing a general picture of the Plant's seismic resilience. Such a snapshot can be refined through the further levels, focusing the detailed analyses only on "*few*" critical component, individuated on the previous levels.

Chapter 2

Objectives and methodology

2.1 Objectives

The objective of the thesis is twofold. First, a methodology for the Resilience assessment of industrial plants, providing, through a detailed methodology, a hierarchy among facilities, is developed. Then, on the basis of the approaches defined for the resilience assessment, a multilevel procedure is proposed. The idea is to provide, at the first level, a rough overview of the Resilience of the entire Plant, defining the critical elements. The next levels provide a detailed overview of the critical facilities, enriching at the same time the estimation of the Plant Resilience.

Such a procedure, inserted in a perspective of restricted economic resources, intends to optimize the number of detailed analysis, usually highly time consuming, only on "*few*" critical structures, providing also quantitative results that can be applied in a decision-making stage.

The proposed resilience-based approach for the assessment of industrial plants should deal with several aspects, associated with the structural behaviour of industrial facilities and the consequences associated with the damage to structures caused by the seismic action in a totally revised fashion. In particular the following aspects are accounted for:

1. **Heterogeneity of existing industrial facilities.** Despite the few structural typologies frequently adopted, the seismic performance of existing industrial facilities strongly depends upon the decision made at the moment of design. In fact, an influencing parameter on the design choices is represented by possible upgrading provisions of the facility, often requiring that the structure itself must withstand loads higher than the ones that it actually bears.

2. **Optimization of the in-situ surveys in the plant**, to perform only on the selected critical facilities.
3. **dependencies among structures**, given by the specific geometrical layout and the organization of the production process within the whole plant. The former aspect is significant with cascading effects and domino events. The latter defines the redundancy of the system. In fact, the more in parallel processes are available the higher is the resilience of the plant against unforeseen events.
4. **Multidimensionality of the problem** in terms of consequences: direct losses, affected by the damages of both structural and non-structural elements; indirect losses, associated with the partial loss of functionality or the complete downtime of the plant within a specific interval of time, need to be accounted for. Due to the multidimensionality of the concept of resilience, the procedure foresees a mutual cooperation between the engineers' team, in charge to perform the assessment, and the technical employee of the industrial plant, which expertise about the function of each single facility within the whole production process is mandatory to provide reliable results.
5. **Prioritization of facilities**, in order to concentrate the detailed analyses only upon critical facilities.
6. Provide information regarding the **feasibility of upgrading strategies** to enhance the resilience.

2.2 Methodology

The achievement of such objectives is developed in three connected steps, briefly described in the following.

In the **first step** the concepts of Resilience currently available in literature are enriched with the purpose to perform the seismic assessment of Industrial Plants. From the organizational perspective, Industrial plants can be seen as a set of stand-alone facilities deeply inter-connected among themselves to ensure the continuity of the production quality. For such a reason, the additional concepts are developed for both stand-alone facilities and inter-connected systems.

For stand-alone facilities, the typical decision variables used to estimate the resilience, i.e. the recovery time, the residual functionality and the recovery path, has been revised, to be suitable for the resilience assessment of industrial plants. Furthermore, the resilience states are proposed as natural alternative to damage states, to make the PBEE framework consistent with

the resilience assessment.

Regarding the inter-connected systems, the methods of the system reliability, currently available in literature cannot be directly adopted for the resilience assessment of Industrial plants. Hence, the additional proposed concepts, consider how the facilities are connected among themselves and which scheme to operate on the damage structure is better to be adopted. In such a way, a comprehensive estimation of the seismic Resilience of industrial installations can be executed.

In the **second step**, the multilevel procedure is developed. The procedure consists of three levels, to deal with the variables that have to be taken into account when the interest is to assess the seismic resilience. Each level provides a resilience-based hierarchy for the prioritization of industrial facilities. Such a prioritization is suitable for the decision-making stage. In fact it provides the facilities that need to be further assessed in the following steps and which ones might require additional interventions to increase the resilience.

The *first level* provides a resilience-based hierarchy of all the facilities of the Industrial Plant, useful to establish which one needs to be assessed more in deep. The structural analyses are performed adopting only the information obtained from rapid in-situ surveys, construction drawings and original calculations (when available), enough to build simplified models.

The *second level* focuses on the critical units/facilities. Their influence on the resilience is deepened, providing a thorough enrichment of their knowledge, obtained with detailed in-situ surveys, structural analyses on complete models and providing a detailed estimation of the feasible operation to restore the initial conditions after the damage occurs. Furthermore, the effectiveness on the plant's resilience of alternative paths that can be adopted to obviate the complete or partial failure of a facility/component, is investigated, if necessary. All the sources, that might increase the resilience of the plant are accounted for, such as the chance to modify some spatial and temporal dependencies among facilities, without compromising the fully operational condition of the Plant. As an outcome the hierarchy among facilities is updated with the results of the refined analyses and the weak elements, either structural or equipments, are individuated.

The *third level* focuses on the specific analysis, by means of refined analyses, of the critical details. The effectiveness of seismic upgrading interventions on the plant's resilience, or the introduction of measures that increase the resilience, are also properly assessed. From this perspective, the third level provides a direct link with subsequent steps that focus on the design of strategies aimed at mitigating the loss of resilience.

In the **third step** the resilience assessment is performed for two exemplary case studies: a steel mill, characterized by a wide in plan extension, and a chemical plant, much more compact and dense of equipment. Both the detailed and the multilevel procedures are applied. The obtained results are compared to validate the reliability of the multilevel procedure.

The core of the thesis develops on the following chapters.

Chapter 3 provides a description of a general model to assess the resilience of dependent systems, typical of industrial plants, proposing a model for the detailed resilience assessment of complex connected systems. Among the aspects related with the application of the procedure, a specific risk metric, suitable for the resilience assessment, is proposed, in order to provide synthetic but precise information about the resilience, that can be easily exploited by the plant's owner, or the local political authorities, in the decision-making process.

Chapter 4 focuses on the conceptual development of the multi-level procedure, describing in detail the focus of each level and the method of application of the whole procedure. The model adopted to provide a definition of the resilience-based hierarchy among facilities is suitable for the prioritization of facilities and exploitable in the seismic resilience mitigation stage.

Chapters 5 and 6 aim at applying and validating the multilevel procedure on two case studies, a steel mill and a chemical plant, representative of typical existing industrial plants but enough heterogeneous among themselves. A detailed seismic resilience analysis is performed, providing the basic results for the validation of the multilevel procedure. Also the multilevel procedure is applied, at each of its three main levels.

Then, in **chapter 7**, the results obtained from the application of the multilevel procedure on both the case studies are compared with the detailed ones, in order to identify and judge the deviations associated with each level of the procedure.

Chapter 3

Seismic Resilience assessment of Industrial Plants

3.1 The concept of resilience

Resilience, according to the dictionary, means *the ability to recover from (or to resist being affected by) some shock, insult or disturbance*. Resilience in general is defined as the ability of systems to rebound after severe disturbances, or disasters.

Various attempts have been made to provide a comprehensive definition of resilience. However, the current definition is extremely vague and cannot be applied in the field of disaster risk reduction.

Manyena (2006) in his research, reviews the concept of resilience in terms of definitional issues, its relationship with the concept of vulnerability, its application in the field of disaster management and risk deduction, and suggests that Resilience could be viewed as the *“intrinsic capacity of a system, community or society predisposed to a shock or stress to adapt and survive by changing its non-essential attributes and rebuilding itself”*.

As regards the relation between resilience and vulnerability, we can say that vulnerability is closely associated to the level of resilience, being a complementary aspect of the preparedness of the system, community or society in relation with specific hazards or disturbances.

The term was first used in psychology and psychiatry in the 1940s.

In physics and engineering, the term resilience describes the property of a material to absorb energy when it is elastically deformed and then, when unloaded, to recover energy (Klein et al., 2003).

Holling (1973), established the concept in the field of ecology, stating that the resilience of an ecological system is *“a measure of the persistence of systems and of their ability to absorb change and disturbance and still maintain the*

same relationships between populations or state variables. Stability represents the ability of a system to return to an equilibrium state after a temporary disturbance; the more rapidly it returns to equilibrium and the less it fluctuates, the more stable it would be”.

The researches in the field of resilience have continued to study it deeper and in a wider way. An extended literature review has elaborated upon resilience for years (table 3.1). Primarily, resilience has been defined in the context of the speed of systems going towards equilibrium (Adger, 2000), capability to cope and bounce back, ability to adapt to new situations (Comfort, 1999), to be inherently strong and flexible and adaptive (Bruneau et al., 2003a), and to withstand external impacts and recover with least outside interferences (Mileti, 1999).

Observing the problem from a PBEE perspective, suitable decision variables should be defined to properly represent the Resilience. In particular, is commonly accepted by literature (Cimellaro (2006), Cimellaro et al. (2010a), Cimellaro and Reinhorn (2010), Cimellaro et al. (2010b) and Cimellaro (2016)) to adopt three **decision variables**:

- the recovery time t_{rec} , that represents the time required to restore the condition of the system immediately before the damage;
- the Residual functionality $q(t)$ is a quantity representative of the condition of the system, e.g. the monetary loss, the productivity etc;
- the recovery path $\mathcal{P}_{q(t) \rightarrow q_{100\%}}$, representing the shape with which the functionality $q(t)$ returns to its pre-damage value.

Known those variables, Resilience, hereinafter called R , is estimated by means of the integral of equation (3.1), in which t_{ev} is the time at which the damage event is triggered, briefly depicted in figure 3.1(a).

$$R = \frac{1}{T_{LC}} \cdot \int_{t_{ev}}^{t_{ev}+t_{rec}} q(t) dt \quad (3.1)$$

Where T_{LC} represents a reference interval of time intrinsic with the system, e.g. 1 year. It should be noted that R ranges between 0 and 1: high values of R mean that the system has a great recovery capacity and can restart to *work* almost immediately.

Instead of the Resilience, it is possible to refer to the Loss of Resilience (LR). Referring to figure 3.1(b), considering a generic recovery path, the LR can be easily expressed as in equation (3.2). From a semantic perspective, LR is the complementary part of the resilience and it has the dimension of a time.

Table 3.1: Literature review about resilience definition

Author	Definition
Holling (1973)	Ecological systems resilience is a measure of the persistence of systems and of their ability to absorb change and disturbance and still maintain the same relationships between populations or state variables
Horn and Orr (1998)	Resilience is the ability of a system to withstand stresses of environmental loading. . . [it is] a fundamental quality found in individuals, groups, organizations, and systems as a whole
Haines et al. (2002)	Resilience is the ability of system to return to its optimal condition in a short period of time. Considering resilience one of four strategies for hardening a system, together with security, redundancy and robustness
Mileti (1999)	Local resiliency with regard to disasters means that a locale is able to withstand an extreme natural event without suffering devastating losses, damage, diminished productivity, or quality of life and without a large amount of assistance from outside the community
Comfort (1999)	Resilience is the capacity to adapt existing resources and skills to new situations and operating conditions
Adger (2000)	Social resilience is the ability of groups or communities to cope with external stresses and disturbances as a result of social, political, and environmental change
Gunderson et al. (2002)	Engineering resilience is the speed of return to the steady state following a perturbation ecological resilience is measured by the magnitude of disturbance that can be absorbed before the system is restructured
Fiksel (2003)	Resilience is the essence of sustainability the ability to resist disorder
Bruneau et al. (2003b)	Resilience is defined in terms of three stages: the ability of a system to reduce the probability of an adverse event, to absorb the shock if the adverse event occurs, and to quickly re-establish normal operating conditions. So resilience thus encompasses the four characteristics of robustness, redundancy, resourcefulness, and rapidity. Are considered four types of resilience: technical; organizational; economic; and social
Allenby and Fink (2005)	Resiliency is defined as the capability of a system to maintain its functions and structure in the face of internal and external change and to degrade gracefully when it must
Rose and Liao (2005)	Regional economic resilience is the inherent ability and adaptive response that enables firms and regions to avoid maximum potential losses
Manyena (2006)	Evaluating all the possible definitions provided from the 1990s to nowadays, resilience could be viewed as the intrinsic capacity of a system, community or society predisposed to a shock or stress to adapt and survive by changing its non essential attributes and rebuilding itself

$$LR = \int_{t_{ev}}^{t_{ev}+t_{rec}} (1 - q(t)) dt \quad (3.2)$$

3.2 Seismic resilience of inter-dependent facilities

The estimation of the seismic resilience of inter-dependent systems requires a specific approach. First, the stand-alone facilities are assessed, in order to define their seismic resilience; second, the seismic resilience associated with

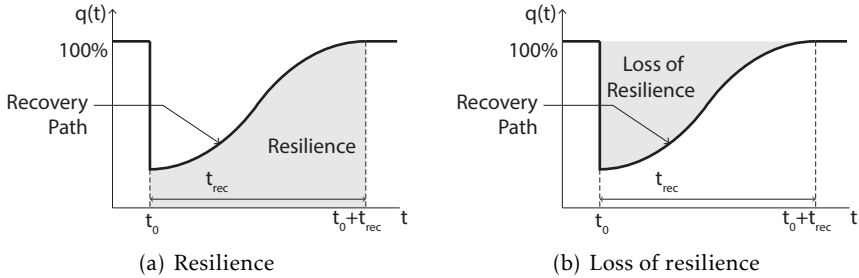


Figure 3.1: Graphical meaning of a) resilience and b) loss of Resilience.

the whole Plant is evaluated, gathering the results of the single facilities.

3.2.1 Stand-alone facilities

The seismic resilience of the stand-alone facilities can be estimated adopting the PBEE framework, accounting for the decision variables previously defined for the resilience evaluation.

In general, aiming at obtaining a detailed estimation of the resilience/losses, the resilience-based analysis of each facility is approached with a **component-based** (Hamburger et al., 2012) approach, in which the loss/resilience function is obtained convolving the building level fragility curves with the cumulative cost of a given damage state.

From the perspective of the PBEE framework, the resilience assessment of an industrial facility, considered as stand-alone part disconnected from the network of facilities representing the whole Plant, requires the following steps:

- Definition of limit states consistent with the current Standards for the seismic assessment of existing facilities (e.g. the EN 1998-3 (2005)), through which define the EDPs (Engineering Demand Parameters); this step is compulsory for the estimation of the vulnerability.
- Definition of Resilience states, explanatory of the resilience loss metric.
- Estimation of the consequences, in a perspective of resilience assessment; quantitatively, a comprehensive definition of the resilience requires three decision variables, which magnitude is linked with damage expected on the system: the functionality $q(t)$; the recovery time t_{rec} and the recovery path $\mathcal{P}_{q(t) \rightarrow q_{100\%}}$; for the sake of clarity, see figure 3.2. Dealing with

industrial facilities, the functionality of the system is represented by the effective production rate of a damaged facility.

3.2.1.1 Limit states

A suitable selection of the limit states, required to perform structural and fragility analysis, ensures the consistency of the resilience assessment with the modern seismic regulations. In particular, dealing with existing facilities, the the three limit states proposed by the EN 1998-3 (2005) are used:

- Damage limitation (DL): non-structural elements exhibit limited damage, while structural elements do not show any substantial plastic deformation;
- Severe damage (SD): structural elements exhibit substantial plastic damage, but they do not reach the failure;
- Near collapse (NC): structural elements reach the failure due to excessive plastic deformation; the failure of a structural component does not prejudice the stability towards the gravity loads.

The global collapse is considered achieved, when one of the following conditions are attained (Hamburger et al., 2012):

- sideways failure, or lateral dynamic instability, characterized by the loss of lateral stiffness and the development of P- Δ instability phenomenon;
- loss of vertical load carrying capacity in gravity or seismic-force-resisting frames, due to excessive drift induced by the seismic action;
- more than 25% of the seismic resistant elements reached the NC limit state.

The first three limit states are mutually exclusive and collectively exhaustive (MECE), while NC and Collapse are two alternatives.

3.2.1.2 Damage states

Within the PBEE framework it is common to deal with damage states (DS). DS provide an estimation of the damage measured in terms of structural weakening and can be directly linked to the direct losses associated with the damage of structural and non-structural elements. Similarly, the European Macroseismic scale (EMS98, Grünthal (1998)), defined five grades of damage. For example, for RC structures, the following grades are defined:

- **Grade 1**, *negligible to slight damage (no structural damage, slight non-structural damage)*: fine cracks in plaster over frame members or in walls at the base; fine cracks in partitions and infills.
- **Grade 2**, *moderate damage (slight structural damage, moderate non-structural damage)*: cracks in columns and beams of frames and in structural walls; cracks in partition and infill walls; fall of brittle cladding and plaster; falling mortar from the joints of wall panels.
- **Grade 3**, *substantial to heavy damage (moderate structural damage, heavy non-structural damage)*: cracks in columns and beam column joints of frames at the base and at joints of coupled walls; spalling of concrete cover, buckling of reinforced rods; large cracks in partition and infill walls, failure of individual infill panels.
- **Grade 4**, *very heavy damage (heavy structural damage, very heavy non-structural damage)*: large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns; collapse of a few columns or of a single upper floor.
- **Grade 5**, *destruction (very heavy structural damage)*: collapse of ground floor or parts (e.g. wings) of buildings.

3.2.1.3 Estimation of the Consequences

Recovery time, Residual functionality and the recovery path represents the decision variables for the estimation of the consequences, expressed in terms of resilience.

Recovery time. The recovery time is the interval of time required to completely restore the full functionality. Considering, for instance, the case of industrial facilities, whose functionality is reduced after the damage induced by a seismic event, the following quantities, necessary for the computation of the recovery time t_{rec} , can be easily outlined:

- time for the inspection (T_{insp}): after a seismic event, T_{insp} represents the time required, by a team of experts, to detect the entity of the damage on a structure; for null-to-limited damages the inspection can be easily performed by the staff of the Plant's technical office, ensuring immediate execution. When the damage cannot be considered negligible, the expert team opinion is required, prolonging the time for the inspection.
- time for the introduction of safety measures (T_{sm}): immediately after the detection of the damage, T_{sm} represents the time interval required to introduce temporary measures, aimed at guaranteeing the safety of work-

ers in the surrounding and avoiding, at the same time, the aggravation of the damaged condition of the facility;

- time for the design (T_D): once the damage is detected, T_D represents the time interval required to design the operations required to restore the structural integrity according to the actual standards (e.g the Eurocodes);
- time for the replacement (T_{rep}): it represents the time interval required to actualize the operations required to restore the structural integrity, replacing or operating on the damaged elements;
- time for demolition and reconstruction (T_R): when the facility results heavily damaged, resulting unfeasible restoring the structural integrity trough a series of target interventions, or the structural collapse has been attained, T_R represents the time interval required to demolish and re-build the whole facility to its previous undamaged condition.

Recovery path. As can be seen in the works of Cimellaro et al. (2010a), Cimellaro and Reinhorn (2010) and Cimellaro et al. (2010b), the recovery path can be represented as a curve with constant or varying slope along the recovery time. With the purpose of providing a mathematical formulation that represents how the facility, immediately after the seismic damage, restore its functionality, the authors proposed uniform, trigonometric and exponential paths to simulate how the facility restores the full functionality, synthetically depicted in figure 3.2.

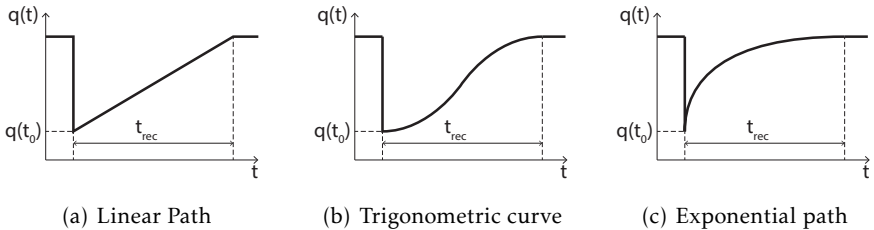


Figure 3.2: Different shapes of the recovery path: a) linear path, b) trigonometric path and c) exponential path. Readapted from Cimellaro et al. (2010a)

Residual functionality. The residual functionality, recorded immediately after the seismic damage ($q(t_0)$, see figure 3.2), can be set on the basis of the safety of workers in the surrounding of the damaged facility. It should be noted that the current literature does not provide enough information to link the Residual functionality with the seismic damage.

Table 3.2: Coefficient of variation for the recovery time phases (readapted from Almufti and Willford (2013) and Lin and Wang (2017b)).

Recovery time phase		C.O.V.
Inspection	1 day	0.2
	7 days	0.54
Re-design	DL	0.4
	SD	0.4
	NC	0.32
Replacement	all LS	0.4

Uncertainties associated with the decision variables. The recovery time is modelled as a stochastic variable, with a lognormal distribution. For the sake of clarity, being the amount of informations provided by the technical office of the Plant wasn't enough to develop statistics about the dispersion of the expected recovery times, the values provided by the ARUP's Guideline *REDI, Resilience-based Earthquake Design Initiative* (Almufti and Willford, 2013, Lin and Wang, 2017a, Lin and Wang, 2017b), were used in addition. Table 3.2 summarizes the standard deviation adopted for each aliquot of the recovery time.

For all the facilities the inspection time is expected equal to 1 working day in the case of negligible damage, i.e. when DL limit state is achieved, otherwise is assumed equal to 7 days.

3.2.2 Connected systems

The analysis of connected systems is faced adopting the general concept that belongs to the reliability of systems, expanded with additional concepts that allow the estimation of the resilience, such the restoration schedule. Moreover, a suitable scalar risk metric is selected for a comprehensive synthesis of the information embedded within the results of the resilience analysis.

3.2.2.1 Reliability of systems

To explain how to calculate the loss of resilience of the whole Industrial Plant, starting from the resilience curve of each stand-alone facility, it is useful to recall the basic concept about the reliability of structural systems (Pinto et al., 2004).

For the purpose of simplification, let's consider a system constituted by n components which behaviour can be described as binary, i.e. characterized just by two possible states, operational (safe) or not-operational (failed). The state of each component is described by a binary state indicator \mathcal{S}_i , characterized by two discrete possible values: 1 that describe the functioning and 0 describing the failure. The functioning of the system is represented by the binary indicator \mathcal{S}_S .

Basically, two arrangements of the components might exists. If the system is not characterized by redundancy, the failure of one component leads to the failure of the system. This is a *series* system, which state function is:

$$\mathcal{S}_S = \prod_{i=1}^n \mathcal{S}_i = \min_{i \in \{1 \dots n\}} (\mathcal{S}_i) \quad (3.3)$$

Instead, a redundant system is a *parallel* system, and its state function is:

$$\mathcal{S}_S = 1 - \prod_{i=1}^n (1 - \mathcal{S}_i) = \max_{i \in \{1 \dots n\}} (\mathcal{S}_i) \quad (3.4)$$

That means that failure of a single component does not lead to failure of the whole system.

Generally speaking, a set \mathcal{C} of components whose failure leads to failure of the system is called *cut set*. Thus, when \mathcal{C} is constituted by the minimal number of elements whose failure leads the system to fail, \mathcal{C} is called the *minimal cut set*. On the contrary, a set \mathcal{P} of components whose survival ensures the survival of the system is called *path set*. When \mathcal{P} is constituted by the minimum number of components to ensure the survival of the system, \mathcal{P} is called *minimal path set*.

A general binary system \mathcal{S}_G is represented by the arrangement of n_c subsystems s_c , connected in series, and n_s subsystems s_s , connected in parallel. The state system of \mathcal{S}_G can be calculated as:

$$\mathcal{S}_G = \prod_{i=1}^{n_c} \left[1 - \prod_{j \in \mathcal{S}_s} (1 - \mathcal{S}_j) \right] = \min_{i \in \{1 \dots n_c\}} \left[\max_{j \in \{1 \dots n_s\}} (\mathcal{S}_j) \right] \quad (3.5)$$

3.3 A proposal for Resilience assessment of Industrial Plants

In order to correctly perform the resilience assessment of Industrial Plants, some additional concepts are introduced for the assessment of stand-alone facilities and the system network. In particular, for what concerns the stand-alone facilities, the concept of resilience states are introduced, derived from the damages states adopted within the PBEE framework, to provide a better lexical consistency with the resilience assessment framework. Furthermore, additional considerations about recovery time, t_{rec} , the recovery path and the Residual functionality are introduced to achieve a correct estimation of resilience of stand-alone industrial facilities. At the system network level, the resilience assessment of the system network is enriched with the concepts associated with the definition of the *optimal restoration schedule*. It represents the theoretical best planning of operation for the restoration of the safety and operational condition of the Plant. Various possible situations are analysed through some practical applications.

The aspects briefly described above, concerning the stand-alone facilities and the system network, are inserted within a PBEE framework, to obtain a detailed estimation of the seismic resilience of Industrial Plants and the facilities prioritization. The application of such a framework requires an exhaustive knowledge level of each facility, the execution of refined numerical analysis, along with the detailed knowledge of the stochastic distribution of the consequences.

3.3.1 Stand-alone facilities

3.3.1.1 Resilience states

For a lexical consistency, instead of speaking about damage states, it is more convenient to deal with resilience states (RS), each of them made up of a specific meaning, suitable for a quantitative resilience assessment.

The proposed RS are reinterpreted from the classification suggested in Burton et al. (2015) and the classification of the building damage, due to the seismic action, provided by the EMS98, to make them consistent with the limit states, provided by the actual seismic regulations (e.g. the Eurocodes framework for the European Union). The four proposed resilience states have general validity, meaning that they can be adopted for the assessment of any

kind of construction.

The four proposed resilience states are mutually exclusive and collectively exhaustive (MECE), meaning that the occurrence of any one type of performance state will preclude the occurrence of the next performance state.

The resilience states are described in terms of the expected damage to the facility.

RS₀: full functionality. The RS₀ represents the minimum damage threshold after an earthquake to guarantee the full functionality of the facility. Such condition means that the structures behave elastically toward the seismic excitation, and all the equipment didn't observed any internal damage. The absence of damage can be surveyed by the technical office; during that time, the functionality has to be suitably dropped down to the partial functionality level.

RS₁: partial functionality with limited interventions. The RS₁ represents a condition in which the facility can partially exploit its function, due to a limited damage to structural elements and, if necessary, to non-structural elements, which integrity is required to satisfy safety requirement. A typical example of such condition is represented by damaged non-structural partitioning walls for confining processes that involve hazardous materials or might yield to harmful production wastes. The functionality must be lowered, in order to reduce the risk of injuries/casualties to the workers, avoiding the worsening of the actual damaged condition of the facility. Being in a damaged condition, the structure has to be repaired. All the interventions must be designed by a technical expert, ensuring the minimum safety requirements prescribed by actual regulations. The functionality can be considered totally restored when the post-damage interventions are fulfilled.

RS₂: partial functionality with substantial interventions. Similarly to the RS₁, RS₂ depicts a condition of partial functionality of the assessed facility. While for the RS₁ the damage is limited, for the RS₂, the structural and non-structural elements of the facility experience severe damage. The functionality can be still restored, but with much more time consuming post-damage interventions.

RS₃: complete loss of functionality. The RS₃ comprises two mutually exclusive states, RS_{3,a} and RS_{3,b} represents the condition of extended damage: RS_{3,a} is typical of near collapse situation, while RS_{3,b} represent the worst case of total collapse. In both the cases the functionality must be dropped to zero. Depending on the structural typology, the functionality might be restored either with repair/replacement, when the damage is concentrated

on few elements and the rest of the whole facility, including equipment and component as well, is intact, either with a complete demolition and reconstruction of the whole facility, in the case that collapse has been attained.

3.3.1.2 Recovery time

Clearly, the seismic damage induces the functionality reduction and depends upon the mean annual frequency of the seismic event. Considering the case of non-collapse, T_{insp} , T_{sm} , T_D and T_{rep} depend upon the entity of the damage and requires detailed inspections at the beginning. In the case of collapse, the time for demolition and reconstruction does not depend upon the entity of the seismic action that trigger the collapse and can be seen as an intrinsic properties of the facility itself. Due to the strict connection with the structural damage, the recovery time depends upon the mean annual frequency (MAF or λ) of the seismic event. For that reason, in the following it will be used the notation $t_{rec}(\lambda)$. The total recovery time t_{rec} is obtained as the sum of a set of the former quantities, chosen in function of the expected damage of the assessed facility.

3.3.1.3 Recovery path

The recovery paths commonly adopted in current literature are suitable for the modeling of the resilience associated with civil use buildings. In fact, the functionality of such buildings strongly depends upon the integrity of non-structural elements. Depending on the configuration of the building, the complete damage on a non-structural element might only compromise the functionality of few areas of the building itself. Thereby, restoring some non-structural elements, the functionality is partially restored and the recovery path might change its slope within the whole recovery time.

For industrial facilities the problem is rather different. As evidenced by previous earthquakes, the damage usually occurs on structural elements. In particular, the functionality of the equipments and components depends on the integrity of the main bearing structures, frequently requiring limited interstorey drift, as for the case of pipelines or belt conveyors in absence of adequate expansion joint. As a direct consequence, until the structural integrity is not completely restored, the functionality cannot increase. Therefore, in contrast to what shown in figure 3.2, the most suitable shape of the recovery path is characterized by a constant zero slope, as shown in figure 3.3.

Introducing the concept of alternative paths, that might be adopted after a seismic damage, the recovery path can assume a stepped shape, as shown in

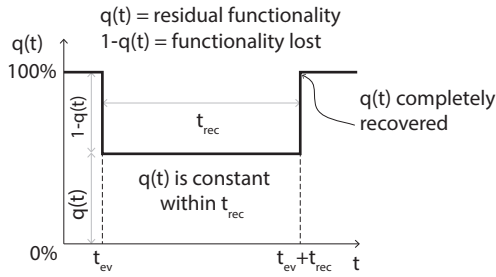


Figure 3.3: Constant recovery path.

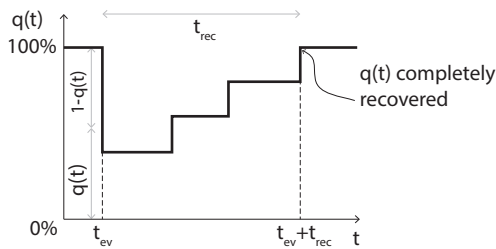


Figure 3.4: Stepped recovery path.

figure 3.4. An example of such situation is represented by the adoption of trucks to overcome the failure of belt conveyors. In this case, it is possible to assist to the following recovery pattern:

- all the belt conveyors fail and trucks are used to maintain active the production chain; the functionality is reduced because the truck solution is either less efficient and more expensive;
- the functionality of part of the belt conveyors is restored; therefore, the functionality of the system partially increases, although some belt conveyors are still shut-down.
- the functionality of the rest of the belt conveyors is restored; thus, the full functionality of the system is fully restored.

3.3.1.4 Residual functionality

The residual functionality is a complex quantity to be assessed. For the peculiar concern to industrial facility, the residual functionality can be defined as the percentage at which the specific *duty* can be exploited by the facility it-

self. Depending on the entity of the structural damage, the facility's residual functionality needs to be reduced for two main reasons: to avoid the spread of the damage that could worsen the condition of the facility itself; to ensure the safety of workers in the surrounding. Also the safety of technical experts in charge to execute the inspection of the facility after the seismic event is included. In particular, the inspection of the facility is mandatory after any kind of seismic event to assess the structural safety, although no damages are apparently visible.

It should be noted that the current literature does not provide enough information to link the residual functionality with the seismic damage. Resulting particularly difficult to perform specific analyses to define a robust connection among the Residual functionality and the seismic damage, some hypotheses are introduced, aiming at providing a definition consistent with the semantic meaning of limit states, currently used in modern Standards (Hamburger et al. (2012), EN 1998-1 (2005), EN 1998-3 (2005) and Norme Tecniche per le Costruzioni (2008)):

- the intact condition of the facility, prior of any seismic event, is associated with the full functionality (100%);
- moderate seismic damage (DL) is associated to a partial residual functionality between the 50% and the 80%;
- severe seismic damage (SD) is associated with a partial residual functionality between the 20% and the 50%;
- both near collapse damage (NC) and collapse yield to a null residual functionality.

The functionality depends upon two variables: the time, which dependency is given by the assumed recovery path, and the intensity of the seismic action, which dependency is provided by the entity of the damage. Anyway, as already shown for the recovery time, focusing on the recovery time period and adopting a constant recovery path, the time dependency can be neglected; for that reason, in the following the notation $q(\lambda)$ will be used.

3.3.1.5 Join performance states and consequences DVs

Four performance states have been proposed:

- RS_0 : full functionality;
- RS_1 : partial functionality with limited interventions;
- RS_2 : partial functionality with substantial interventions;
- RS_3 : complete loss of functionality;

Table 3.3: Quantitative meaning of the Resilience states in terms of recovery time and residual functionality.

Resilience state	Recovery time	Residual functionality
RS₀	$T_{RS_0} = T_{insp}$	$q_0(t) = \text{full}$
RS₁	$T_{RS_1} = T_{insp} + T_d + T_{rep}$	$q_1(t) = \text{partial (50 - 80 \%)}$
RS₂	$T_{RS_2} = T_{insp} + T_d + T_{rep}$	$q_2(t) = \text{partial (20 - 50 \%)}$
RS_{3,a}	$T_{RS_{3,a}} = T_{insp} + T_d + T_{rep}$	$q_{3,a}(t) = \text{null (0 \%)}$
RS_{3,b}	$T_{RS_{3,b}} = T_{insp} + T_R$	$q_{3,b}(t) = \text{null (0 \%)}$

- RS_{3,a} when the facility achieves the NC limit state;
- RS_{3,b} when the facility collapses.

Each of them is provided of a specific meaning, that can be translated quantitatively in terms of consequences decision variables (DVs). Table 3.3 reports the connection of the resilience states with the decision variables representative of the consequences. It should be noted that T_{RS_1} to $T_{RS_{3,a}}$ are only apparently equivalent. In fact the time for replacement and interventions vary with the damage extension. Regarding the residual functionality is provided with possible ranges, e.g 50-80% for $q_1(t)$. The univocal value must be agreed, by common agreement, between the engineer responsible for carrying out the resilience assessment and the technical office of the plant. Depending on the expected damage, the first provides indispensable indications on the safety of the workers in operation, while the latter provides information on the maximum functionality of the structure.

3.3.1.6 Loss of Resilience

Seen the above considerations about the three decision variables, the resilience (R) and the loss of resilience (LR) can be estimated as in the following:

$$R = \frac{1}{T_{LC}} \cdot \int_{t_{ev}}^{t_{ev}+t_{rec}} q(\lambda) dt \quad LR = \int_{t_{ev}}^{t_{ev}+t_{rec}} (1 - q(\lambda)) dt \quad (3.6)$$

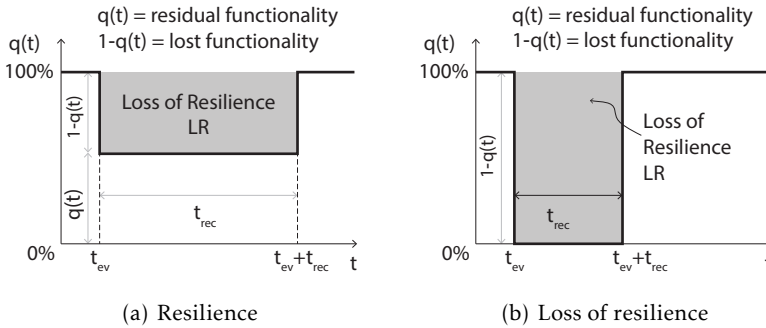


Figure 3.5: Comparison of two loss of resilience equivalent conditions: a) characterized by partial Residual functionality for a longer time; b) characterized by complete loss of functionality for a shorter time.

where t_{ev} is the precise instant at which the damage, due to the seismic event, occurs.

It should be noted that, as it is conceived, LR provides a direct estimation of the time (evaluated, for instance, as number of days) at which the Plants works at a partial functionality, or with a simple proportional scaling, the time of complete shut-down of the Plant (see figure 3.5).

The days of complete shut-down of the Plant can be directly associated with the expected indirect losses caused by the shut-down of the Plant. That aspect makes the loss of resilience better than the resilience itself, providing a parameter that can be directly related with economic losses. For that reason, loss of resilience (LR) will be used in this study instead of the resilience.

3.3.1.7 Analytical modeling of the Resilience

The resilience assessment aims at providing an univocal relation among a suitable decision variable and the MAF (or λ) of the seismic action. For each intensity measure of the seismic action, the damage measure, computed from the fragility curves, are then linked to the consequence states, t_{rec} and $q(t)$. Fixed the intensity measure, the passage from the fragilities, towards the damage measure, to the consequence estimation can be graphically depicted by means of the event tree graph, as briefly depicted in figure 3.7.

Considering $[LS_0 \ LS_1 \ LS_2 \ LS_3]$ respectively equal to the DL, SD, NC and

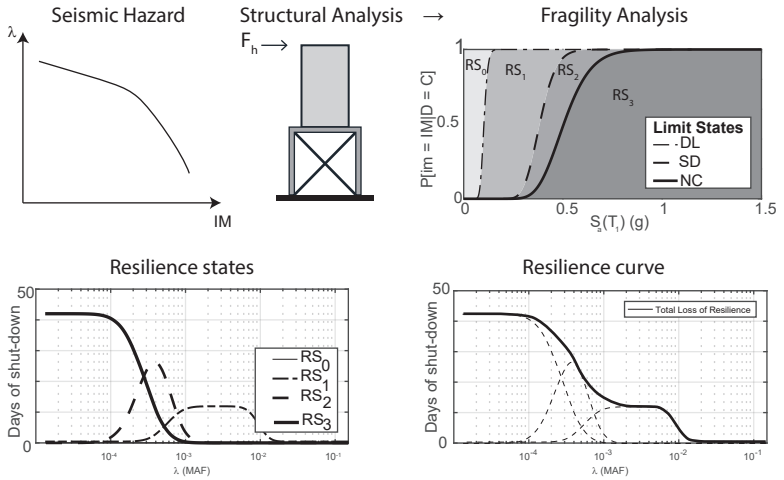


Figure 3.6: Graphical interpretation of the estimation of the resilience curve.

collapse limit states, the probabilistic measure of the Resilience States (RS) is computed as

$$P_{f,RS_i}(\lambda) = \begin{cases} 1 - P_{LS_i}(\lambda) & \text{if } i = 0 \\ P_{LS_i}(\lambda) & \text{if } i = 3 \\ P_{LS_{i-1}}(\lambda) - P_{LS_i}(\lambda) & \text{otherwise} \end{cases} \quad (3.7)$$

Where λ is the mean annual frequency (MAF) of the seismic action. The loss of resilience LR is easily computed as show in eqs (3.8).

$$LR(\lambda) = \sum_{i=0}^3 lr_{RS_i}(\lambda) = \sum_{i=0}^3 P_{f,RS_i}(\lambda) \cdot (t_{rec,i}(\lambda) \cdot q_i(\lambda)) \quad (3.8)$$

where $lr_{RS_i}(\lambda) = P_{f,RS_i}(\lambda) \cdot (t_{rec,i}(\lambda) \cdot q_i(\lambda))$ represents the loss of resilience associated which each resilience state. $LR(\lambda)$ is called resilience curve.

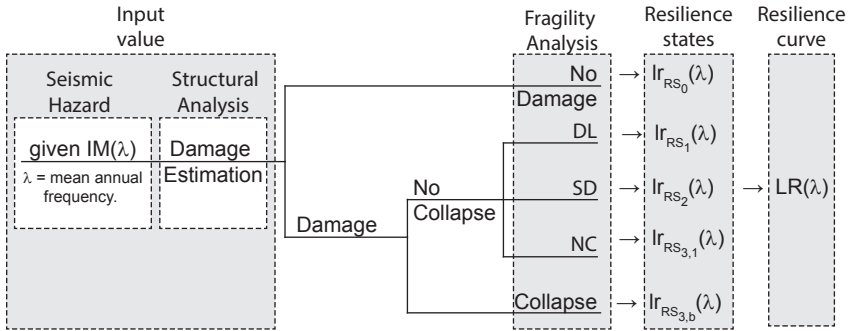


Figure 3.7: Adoption of the event tree for the estimation of the resilience curve.

3.3.2 Interconnected systems

All the above-mentioned concepts related with the reliability of systems can be transferred from the reliability assessment to the resilience (or Residual resilience) assessment of an Industrial Plant. To this aim, further details are required and in particular the system network of the plant, the reconstruction schedule and a smooth state function of each facility. Furthermore, in order to provide a comprehensive estimation of the resilience, suitable scalar risk-based metric, for a fast but detailed synthesis of the information included within the resilience curve, is also proposed.

System network. Aiming at describing the series or parallel connection among facilities, the block diagram is suitable for the purpose. The effects of an event on stand-alone blocks can be represented through the event tree. Within the block diagram the production units can be defined: a set of facilities, variously connected, used for the common production outcome.

Generally, it is rather easy to detect, within the whole plant, specific production units (subsystems), composed by different facilities connected in series and in parallel as well. Thus, is possible to detect *minimal cut sets* and *minimal path sets* within the Plant.

It should be noted that, for economic and space-saving reasons, existing industrial plants were frequently designed with a low redundancy, preferring series production units.

Restoration schedule. The operations aimed at restoring the functionality of the facilities after a damage can be planned in several different ways, depending on the configuration of the Plant, the availability of resources. Tech-

nically, infinite possibilities of scheduling the reconstruction plans might exist. Some of them are impracticable, due to temporal problem and interferences among different operations. Therefore, for the purpose of the analyses, among all the feasible plan, the selected reconstruction plan is the one characterized by the minimum required time and is called *optimal reconstruction schedule*.

Similarly to possible kind of connections among facilities, a *series* or *parallel* scheduling of interventions can be adopted. Intuitively, when series interventions are expected, the total intervention time is given by the sum of each single intervention; on the contrary, when parallel interventions are expected, the total intervention time is equal to the maximum among all the single interventions.

The resourcefulness can be estimated through the *reconstruction schedule*, representing the temporal order with which interventions on facilities are foreseen and possible after a damage.

State function. Instead of having a binary state function, equal either to 0 or 1, each facility has a smooth state functions lr_i , see equation (3.8). The function has two bounds: the lower bound is equal to 0 and means that no losses are expected; the upper bound is equal to the shut-down time of the facility due to complete reconstruction. Clearly, as shown in figure 3.8, the higher is the vulnerability of the facility, the greater is the range of frequencies at which the function is closed to the upper bound.

Knowing the system network of the Plant means knowing the redundancy of the Plant itself. In particular it is possible to provide the following observations:

- generally, the higher are the *in series* connections among facilities, the lower is the redundancy of the system toward the seismic action;
- the higher are the *in parallel* connections among facilities, the higher is the redundancy of the system toward the seismic action;
- to effectively ensure adequate redundancy to the system, *in parallel* production processes must be foreseen for subsystems more vulnerable to the seismic action.

3.3.3 Definition of the risk metric

For a comprehensive estimation, the information provided through the Resilience curve should be summarized with a suitable synthetic index.

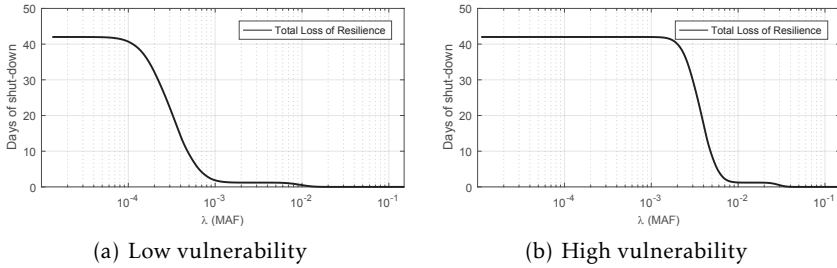


Figure 3.8: Comparison of the resilience curve of two facilities, respectively characterized by low and high vulnerability.

To this aim, as previously discussed, of particular interest are the Value at Risk (Var) and its variants, e.g. the tail Value at Risk (tVar), and the expected annualized losse (EAL). The first provides a precise estimation of the maximum expected losses and is usually obtained through the definition, on the loss/Resilience curve, of the value associated with a MAF of 10^{-4} - 10^{-3} . Anyway, the VaR does disregard the shape of the loss/Resilience curve at high frequency ranges.

Such a lack of knowledge can be solved adopting the EAL, which, integrating the losses over the entire range of frequencies considered, provides an overview of the expected losses per year over the entire nominal life of the Plant. The negative aspect relies on the fact that the EAL is not able to account for neither the shape of the loss/Resilience curve neither the concentration of losses in correspondence of low-probability high-consequences events. For that reason, is said that EAL provides a risk-neutral approach. Being the industrial facilities not designed against the seismic action, their weaknesses are frequently triggered at very high earthquake MAF. Therefore, the interest is to provide a thorough overview of the losses/Resilience over the whole range of frequencies and the use of the EAL is considered the most suitable metrics.

3.3.4 Practical example for the calculation of Resilience

Summarizing, the estimation of the resilience requires:

1. four smooth state function $lr_{RS,i}$, representing resilience states of each stand-alone facility;
2. the system network of the Plant;
3. the reconstruction schedule.

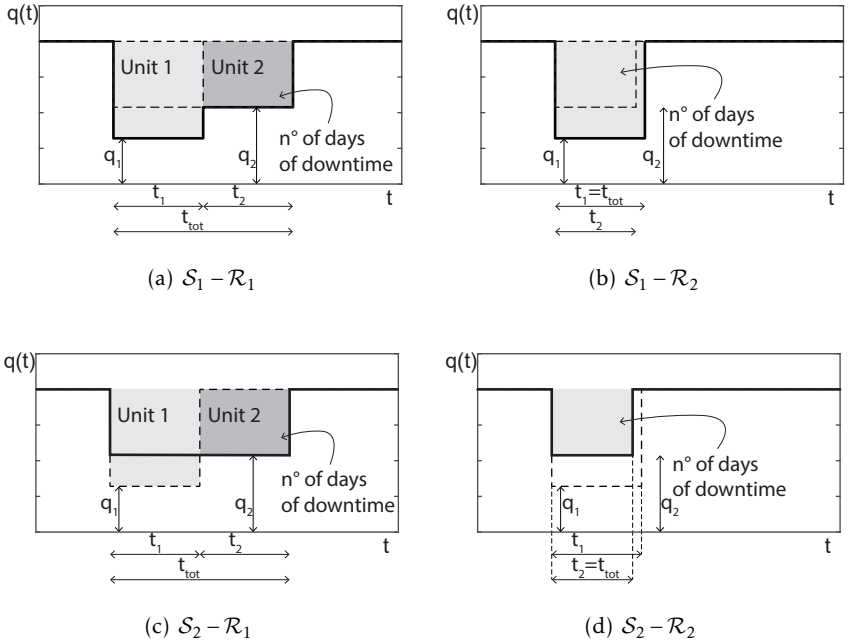


Figure 3.9: Recovery paths considering different dependencies among facilities and different restoration schedule.

Considering, for instance, two systems \mathcal{S} and two reconstruction schedules \mathcal{R} . Each system is composed of two unit, u_1 and u_2 . The system \mathcal{S}_1 is constituted by an in series network, while the system \mathcal{S}_2 is constituted by an in parallel network. The reconstruction schedule \mathcal{R}_1 foresees 2 in series operations, while \mathcal{R}_2 foresees in parallel operations. The following hypotheses are applied:

- the residual functionality of unit u_1 , $q_1(\lambda)$, is lower than the residual functionality of unit u_2 , $q_2(\lambda)$;
- according to the definition of the optimal reconstruction schedule, the operations begins with the unit with the lower residual functionality.

Figure 3.9 provides a graphical description four possible situations.

The resilience curve of the two systems is calculated as in the following:

System \mathcal{S}_1 and reconstruction schedule \mathcal{R}_1 :

- the residual functionality of the system, $q_{\mathcal{S}_1, \mathcal{R}_1}(\lambda)$, is given by the minimum of the two units:

$$q_{\mathcal{S}_1, \mathcal{R}_1}(\lambda) = \min(q_1(\lambda), q_2(\lambda)) \quad (3.9)$$

- the recovery time of the system, $t_{rec, \mathcal{S}_1, \mathcal{R}_1}(\lambda)$, is given by the sum among the recovery times of each unit:

$$t_{rec, \mathcal{S}_1, \mathcal{R}_1}(\lambda) = \sum_{i=1}^2 t_{rec, i}(\lambda) \quad (3.10)$$

System \mathcal{S}_1 and reconstruction schedule \mathcal{R}_2 :

- the residual functionality of the system, $q_{\mathcal{S}_1, \mathcal{R}_2}(\lambda)$, is given by the minimum among the two units:

$$q_{\mathcal{S}_1, \mathcal{R}_2}(\lambda) = \min(q_1(\lambda), q_2(\lambda)) \quad (3.11)$$

- the recovery time of the system, $t_{rec, \mathcal{S}_1, \mathcal{R}_2}(\lambda)$, is given by the maximum among the recovery times of each unit:

$$t_{rec, \mathcal{S}_1, \mathcal{R}_2}(\lambda) = \max(t_{rec}(\lambda)) \quad (3.12)$$

System \mathcal{S}_2 and reconstruction schedule \mathcal{R}_1 :

- the residual functionality of the system, $q_{\mathcal{S}_2, \mathcal{R}_1}(\lambda)$, is given by the maximum of the two units:

$$q_{\mathcal{S}_2, \mathcal{R}_1}(\lambda) = \max(q_1(\lambda), q_2(\lambda)) \quad (3.13)$$

- the recovery time of the system, $t_{rec, \mathcal{S}_2, \mathcal{R}_1}(\lambda)$, is given by the sum of the recovery times of each unit:

$$t_{rec, \mathcal{S}_2, \mathcal{R}_1}(\lambda) = \sum (t_{rec}(\lambda)) \quad (3.14)$$

System \mathcal{S}_2 and reconstruction schedule \mathcal{R}_2 :

- the residual functionality of the system, $q_{\mathcal{S}_2, \mathcal{R}_2}(\lambda)$, is given by the maximum of the two units:

$$q_{\mathcal{S}_2, \mathcal{R}_2}(\lambda) = \max(q_1(\lambda), q_2(\lambda)) \quad (3.15)$$

- the recovery time of the system, $t_{rec, \mathcal{S}_2, \mathcal{R}_2}$, is given by the minimum among the recovery times of each unit:

$$t_{rec,S_2,R_2}(\lambda) = \min(t_{rec}(\lambda)) \quad (3.16)$$

Known $q_{S_1}(t, \lambda)$ and $t_{rec,S_1}(\lambda)$, the estimation of the Resilience curve is straightforward:

$$LR_{S_i,R_i}(\lambda) = \int_{t_{ev}}^{t_{ev}+t_{rec,S_i,R_i}(\lambda)} 1 - q_{S_i,R_i}(\lambda) dt \quad (3.17)$$

3.3.5 Detailed Resilience assessment and facilities prioritization

The aforementioned aspects, for what concern the stand-alone facilities and the system network, inserted within a PBEE framework allow the estimation of the seismic resilience of Industrial Plants, constituted of facilities interconnected among themselves.

As it is conceived, the PBEE provides a deep insight of stand-alone facilities. Anyway, when the focus moves to connected system, the post-processing and interpretation of results can be difficult, in particular when the aim is to provide a prioritization of facilities for a subsequent decision-making stage.

This section aims at providing a suitable method for the resilience assessment of industrial plants, providing a hierarchy among facilities, exploitable to prioritize the interventions required.

The proposed approach develops over two steps:

1. the first aims at drawing the resilience curve, its confidence bounds and an estimation of the EAL curve;
2. the second is a sensitivity-based assessment aimed at identifying the weight of each facility on the resilience of the whole plant.

Step 1. The resilience curve and its confidence bounds is built by means of a Monte Carlo Analysis. The input variables for the estimation of the consequences, i.e. the recovery time and the residual functionality, can be suitably assumed to be lognormally distributed. Such a decision, also adopted within the Hamburger et al. (2012) and by Almufti and Willford (2013), perfectly suits the practical bounds of the involved variables: zero, for the lower shelf, and infinity for the upper one.

Considering input variables as lognormally distributed, also the final re-

sults can be hypothesized as lognormally distributed. Thus, the confidence bounds, e.g. the 16% and 84% quantiles, can be calculated reducing to the problem to the individuation of two extremal quantiles, e.g. the 16% and the 84% ones, or numerically by means of parametric bootstrap (Efron, 1987).

This step provides as outcome:

- the resilience curve of the plant and its confidence bounds,
- the EAL of the plant and the parameters to define their lognormal distribution, i.e. the mean and the standard deviation .
- the EAL of each facilities and the parameters to define the lognormal distribution.

Step 2. In the second stage the weight of each facility on the EAL of the whole plant is assessed. In particular a linearized method, based on standard regression coefficients (SRC), is used. Given the input vector \mathbf{X} , the output Y can be computed through a multiple linear regression:

$$Y \approx \alpha_0 + \sum_{i=1}^M \alpha_i \cdot X_i \quad (3.18)$$

If the input variables X_i are independent, the total variance of the model can be estimated by:

$$\hat{\sigma}_y^2 = \sum_{i=1}^M \alpha_i^2 \cdot \sigma_X^2 \quad (3.19)$$

where σ_Y^2 and σ_X^2 are respectively the variances of the response and of each input.

The SRC indices can be calculated as:

$$SRC_i = \frac{\hat{\alpha}_i \cdot \sigma_i}{\sigma_Y} \quad (3.20)$$

Considering the EAL of each facility ($EAL_{s,i}$) as input variables lognormally distributed and the EAL associated with the whole plant (EAL_p) as output variable, the proposed approach can be easily applied and the calculation of the SRC indexes is straightforward.

The SRC indexes can be associated with a practical meaning. In fact, they can be seen as the weight with which each plant influences the resilience of the Plant, expressed in terms of EAL: the higher the SRC index the higher is the importance of the facility and, therefore, it requires particular attention during a decision-making stage.

Chapter 4

The multilevel procedure

4.1 Organization of the multilevel procedure

The multilevel procedure is structured on a three levels framework and aims at assessing the resilience of industrial plants.

The framework is conceived to provide a global overview of the resilience at the Plant's level, adopting simplified models at the first level, enriching the results in the following levels, detailing the in-field surveys, the structural analyses and the estimation of the consequences.

All the three levels provide a resilience-based prioritization of facilities, for which a suitable optimization procedure is developed. The results of such a prioritization, expressed in terms of *Resilience Indicators* (RI), are used to define the critical facilities that should be analysed in the following steps of the procedure.

Being the resilience a multidimensional problem, the procedure is structured on the basis of a mutual collaboration among different actors: the technical office of the plant and the engineer team in charge to perform the structural analysis. The former provides all the informations about the consequences, acquiring the information from historical data available or by means of expert judgment. The latter provides its expertise about in-field surveys, structural analysis at any level of detail and the ability to gather all the information, providing a synthetic, yet effective, estimation of the Plant's resilience.

The structure of the procedure is graphically shown in figure 4.1 and the main aspects are described in the following:

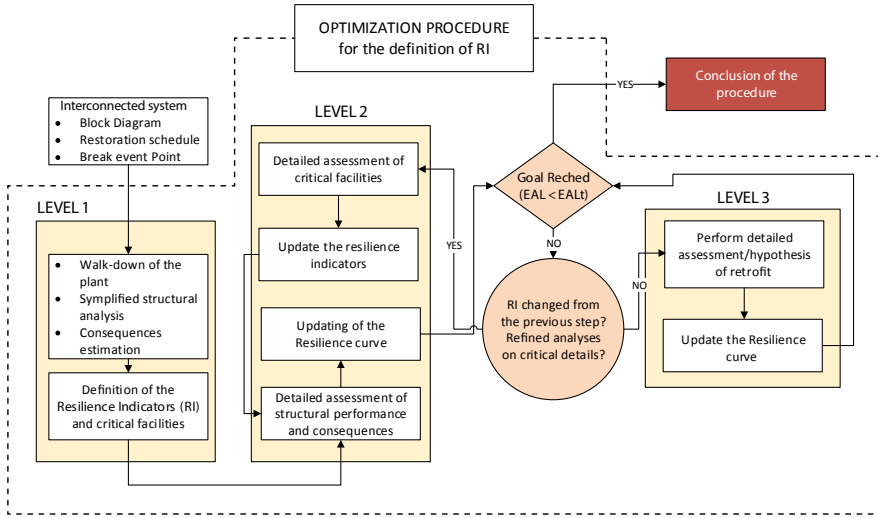


Figure 4.1: Framework of the multilevel procedure.

1. **Identification of the interconnected system**, i.e. the schematization of the system network (block diagram) representative of the industrial plant, the restoration schedule, according to what described in sect. 3.3.2 and the definition of the break even point (maximum shut-down due to unforeseen events that can be withstand by the plant without disproportionate economic losses).
2. **Application of the multilevel procedure**. Each level of the procedure adopts the framework described in sect. 3.3. All the involved variables are treated as deterministic. In particular, the consequences are estimated considering their mean value. In the following a synthetic overview of each level is provided.

Level 1. The first level operates at the plant size and aims at providing a general overview of the plant's resilience, basing the analyses on few details. This level is structured to be rapid to apply, in order to ease the updating of the results when required. It is structured on the following three steps, that will be analysed later in detail:

- rapid survey of the plant (*walk-down*); this step aims at providing a minimum knowledge of the plant and its facilities; regarding the facilities that have a technical documentation (drawings and reports), the *walk-down* aims at validating the available information; if the technical documentation is missing, the *walk-down* is

planned to provide a minimum information about the structural configuration and the structural details;

- definition of simplified models for a fast structural analysis; the mechanical parameters of the structural materials are assumed equal to the design value imposed by Regulations used at the time of construction; simplified approaches for the structural analyses of drift sensitive and peak floor acceleration sensitive structures are provided;
- simplified estimation of the consequences (*consequence matrix*).

Figure 4.2 shows the duties that are, at the first level, of competence of the technical office of the Plant and to team of Engineers in charge to perform the resilience assessment.

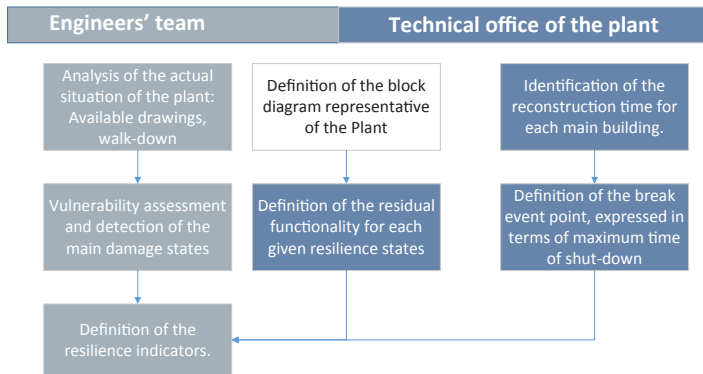


Figure 4.2: Interaction between the Plant's technical office and the Engineers' team for the level 1.

Level 2. The critical facilities that mainly influence the seismic resilience of the plant are assessed through detailed analyses. Specific in-field surveys are planned to achieve a detailed knowledge of the structural details. Then, detailed structural analyses and a precise estimation of the consequences are performed. Since a detailed knowledge of the facilities is achieved, the mean value of the mechanical characteristics, obtained from either destructive or non-destructive tests, should be adopted. For what concern the consequences, alternative paths that can be activated to avoid the reduction of the functionality of a facility/unit are also accounted for, if present.

Figure 4.3 shows the duties that, at the second level, are of competence of the technical office of the Plant and to team of Engineers in charge to perform the resilience assessment.

Level 3. The third level focuses (i) on the precise definition of the structural performance of critical details, through highly refined analyses and (ii) on the identification of possible upgrading solutions aimed at increasing the resilience.

3. The Resilience Indicators are defined by means of an **optimization procedure** that require the definition of the target condition, in the following referred as break even point (t_{BEP}), representative of the maximum downtime period the Plant's owner can face off, without making any economic losses. The optimization procedure aims at providing synthetic indexes, the Resilience Indicators, that represent how much the resilience of each stand-alone facility should be scaled down to achieve the break even point at the plant level. By this means, the RIs include within themselves two aspects: (i) the actual condition of each facility, for what concern seismic vulnerability and consequences; (ii) the final condition, in terms of resilience, that is expected each facility should achieve to reach the break even point. Applying the optimization procedure, a theoretical resilience curve that allow to achieve the break even point is also given. Dealing with inter-connected systems, characterized by facilities with different seismic vulnerability and associated consequences, the definition of the Resilience indicators is not straightforward, therefore a specific optimization procedure is proposed.

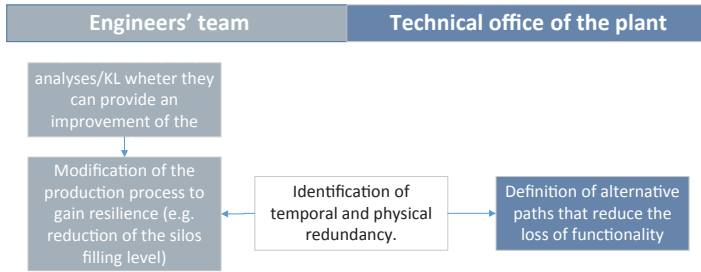


Figure 4.3: Interaction between the Plant's technical office and the Engineers' team for the level 2.

After the application of the first level at the Plant size, the second and third levels interact *circularly* among themselves (see figure 4.1). Briefly, the procedure is applied as in the following:

- the simplified analyses are performed at the Plant level to provide a general overview of the resilience;

- the detailed levels are applied on the structure associated with the highest Resilience Indicators;
- at the end of each detailed level, it is checked whether the target condition is attained;
- if the target condition is not attained, further detailed investigations on the remaining critical facilities need to be performed. If at the end of the second level, the hierarchy among facilities strongly changes, the operator can decide to continue to apply the second level and verify if there is a gain in terms of resilience.

In such a way, there is no need to define fixed thresholds that establish the structures that require further detailed investigations, avoiding useless cumbersome analyses

4.2 Level 1

The level 1 aims at providing a general overview of the resilience within the whole plant, adopting simplified models for the structural analyses and the consequences estimation. In order to achieve a suitable accuracy of the results, an important effort should be paid on the selection of the simplified models, leading the level 1 as the most complex of the whole procedure.

The first level consists of:

- *walk-down* and structural inspections of the Plant, aimed at obtaining a minimum knowledge of the structural configuration and details of each stand-alone facility;
- analysis of the structural performance based on simplified numerical models, aimed at defining the structural vulnerability of each stand-alone facility;
- simplified model for the estimation of the consequences, through the *consequences matrix*;

4.2.1 "Walk-down" and structural inspections

Several proposals are available in literature for the walk-down of industrial facilities. It's worth mentioning the methodologies proposed by Selvik et al.

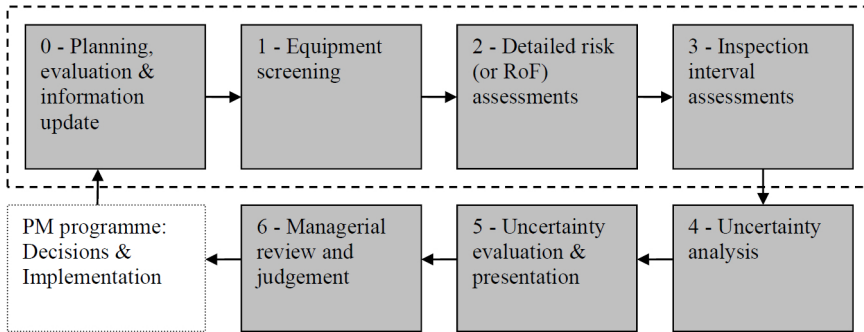


Figure 4.4: Framework for the extended methodology (the RBI methodology is indicated by the dashed line). Readapted from Selvik et al. (2011).

(2011) and by the T.C.S.E.D.P.¹ (2011).

Selvik et al. (2011) proposed a model for the risk-based inspection (RBI) planning, focusing on mechanical equipment, in particular piping networks. The inspections are prioritized based on risk, adopting as a decision variable metrics the product of the failure probability and the expected consequences, assessed in a qualitative manner, following what already suggested by international standards (API (2002), Aven and Renn (2009), Veritas (2002), NORSOK (2001)). Such approach, based on six steps (see figure 4.4), aims at identifying the uncertain factors of possible mechanical failure of components, categorizing them in three categories, low, medium and high, adopting a sensitivity analysis.

T.C.S.E.D.P. (2011) extend the concept of walkdown of existing facilities, proposing a methodology focused on the vulnerability assessment, widely adopted in California petrochemical industries. Such a methodology was developed to face off to the increasingly requirements of safety that Petrochemical facilities have to ensure, enriching previous existing models, such as the California Accidental Release Prevention (*CalARP*) program requirements, the *OSHA*² laws and the *EPA*³ laws. The core fo the overall methodology is characterized by the following steps:

- a) Meetings with owners, operators, regulators, process safety engineers, or other appropriate parties to discuss objectives of the evaluation and to establish performance requirements of facility.

¹ Task Committee on Seismic Evaluation and Design of the Petrochemical Committee of the Energy Division of the American Society of Civil Engineers

² Occupational Safety and Health Administration.

³ Environmental Protection Agency.

- b) Identification of equipment, structures and piping of interest.
- c) Establishment of damage categories. Walkdown efforts are often required to be coordinated with a process safety team to establish requirements for use in the consequence analyses. The damage is provided in a qualitative way (no damage, minor and repairable damage, irreparable damage, potentially catastrophic).
- d) Collection of site data, including seismic hazard data, fault locations, information about the seismic design of existing facilities.
- e) Walkdown evaluation of components.
- f) Review of drawings as necessary, to check adequacy of reinforced concrete structures, to verify anchorage details, or to identify configurations which cannot be visually reviewed due to obstructions, fireproofing, insulation.
- g) Identification of items for analytical review. These may include "worst-case" items or any items that appear to be seismically vulnerable.
- h) Documentation of "poor" or "questionable" items for owners or regulators.
- i) Recommendation of structural or mechanical fixes or other efforts which would mitigate risk from items listed above.

The approach proposed by T.C.S.E.D.P. (2011) certainly provides a comprehensive methodology for the seismic walkdown, but is mainly based on qualitative judgments, forcefully influenced by the experience of the technician in charge to perform the assessment.

Thus, the walk-down model proposed by T.C.S.E.D.P. is improved to fit within the multilevel procedure, introducing some further detailed aspects. A minimum knowledge of the structural configuration of each facility, the production process and the interdependencies among facilities, is required to pursue enough reliable results. Usually, those aspects can be provided by the technical office of the Plant. Anyway, some issues might arise when the knowledge of the structural configuration of facilities is of particular concern. In fact, original drawings of facilities are often missing, with particular concern to rather old existing facilities. Moreover, it is common that some structural modifications, made to fit with new, more performing, equipment are executed without specific design of the intervention. Thereby, relying only on the information provided by the original drawings might lead to a lack in the knowledge of the structural configuration of the facilities.

A strict cooperation of the Engineers' team in charge to perform the seismic resilience assessment and the technical office is compulsory to achieve the minimum knowledge of the Plant. Such a collaboration aims at optimizing the plant walk-down and inspections, focusing only on the critical details. In order to achieve a satisfactory minimum knowledge of the plant

and considering the aim to define the seismic resilience of the Plant, the following procedure is adopted:

- a) Execution of the seismic hazard analysis of the site of interest, in compliance with actual seismic regulations.
- b) Execution of archive research, aimed at collecting the overall technical documentation, i.e. drawings and technical report, internally redacted by the technical staff of the plant, or externally redacted. Whether technical documents are not available or incomplete, such information can be obtained knowing the age of construction, from which the design regulation adopted for the design is derived.
- c) Assign univocal ID to each facility of the Plant.
- d) Being available a reduced amount of information, the Engineers' team identifies the Critical element that might yield a plausible collapse mechanism.
- e) The staff of the Plant's technical office checks that the technical drawings are reliable and adequately up to date;
- f) The inspections are performed on the critical elements:
 - (i) if the drawings are up to date, the consistency of the actual geometry with the drawings is randomly checked of few elements;
 - (ii) if the drawings are not up to date, a detailed survey of the geometry is performed to verify the actual geometry of the structural elements.

The *walk-down* has to be customized for each facility, depending on its structural configuration, its expected collapse mechanism.

Typical industrial facilities are, for instance, the followings:

- Supporting structures: RC or steel structures, with different structural configurations, i.e. moment resisting frames (MRF) and concentrically braced braces (CBF);
- vessels, atmospheric and in pressure;
- industrial Warehouses;
- piperacks;
- pipelines;
- belt conveyors.

4.2.2 Analysis of the structural performance and simplified Models

The concern of this part of level 1 is to provide a reliable estimation of the seismic performance of the assessed facility. The performance is estimated with simplified models but accurate enough to correctly represent the inelastic structural behaviour. Such models are realized on the basis of the restricted knowledge obtained from the walkdown.

The assessment of the structural performance is conducted differently for drift sensitive and peak floor acceleration facilities.

Drift sensitive facilities. As a first concern, the degree of freedom (DoF) of the assessed facility have to be reduced, aiming at obtain reduced MDoF or SDoF models. The simplified model should allow, in a very short time, the definition of the capacity curve, expressed in terms of base shear versus top displacement, through a pushover analysis or with simplified manual analyses.

The reduction of the degree of freedom is often straightforward, being a huge percent of industrial facilities characterized by one-storey structure, in which a SDoF non-linear system can be easily identified.

Regarding the realization of the simplified model, the following hypotheses are applied:

- the interaction, static and dynamic as well, between the main structure and equipments, e.g. pipelines, is neglected;
- rigid diaphragm hypothesis is applied at each storey;
- secondary structures are modelled as masses; therefore their stiffness is neglected;
- the storey-mass is modelled uniformly distributed at each floor;
- pipelines and equipments are analysed by means of the sub-structuring method.

In order to obtain results exploitable within the PBEE framework, fragility curves need to be exploited from the numerical model. Clearly, the adoption of IDA analyses would surely provide reliable results, but it requires a suitable selection of accelerograms and the execution of numerous numerical models, the results of which would require an enormous time consuming effort.

In this perspective, the SPO2IDA (Static Pushover over to IDA curves) model (Vamvatsikos and Cornell, 2005) is a valuable method for a rapid estimation of the seismic fragility. In fact, such tool provides a $R - \mu - T$ (strength reduction-Ductility-Period) relation that allow to translate the ADRS (Acceleration displacement response spectrum) capacity curve into IDA curves,

from which the fragility curves can be easily estimated with an IM-based approach, assuming a log-normal shape.

Peak floor acceleration (PFA) sensitive facilities. Regarding PFA sensitive structures, the main problem relies on the construction of floor response spectra (FRS). At the first level, instead of deriving the RFS with numerical or analytical models currently provided in technical literature, the structural performances of the PFA sensitive structures is calculated with a simplified approach. Considering a log-normal shape of the fragility curves, the approach allows the definition of the mean seismic capacity and the standard deviation:

1. given the structural configuration, the maximum horizontal force $F_{H,max}$ the secondary structural element can withstand in the elastic field, is calculated with simplified structural model.
2. The maximum expected horizontal force, evaluated accounting for the amplification due to the height, is calculated adopting the procedure proposed by the German VCI guidelines (Butenweg et al. (2012a) and Butenweg et al. (2012b)):

$$F_{a,max} = 1.6 \cdot \gamma \cdot pga \cdot m_a \quad (4.1)$$

where γ is the importance factor, evaluated in function of the hazardous material stored; m_a is the seismic mass of the structural element; 1.6 account for the seismic amplification and pga represents the peak ground acceleration, correspondent to a 10% probability of exceedance within the nominal life of the structure (see EN1998-1). Imposing the equivalence $F_{H,max} = F_{a,max}$, the pga (representative of the mean seismic intensity) at which the structure yields is defined.

3. the dispersion above the mean seismic intensity is computed with the tabular format proposed within the FEMA 695 (FEMA P-695, 2009). Depending on the knowledge of the details and the structural behaviour, the tabular approach (see table 4.1) provides the expected standard deviation.

4.2.3 Consequences matrix

The aim of the consequences matrix is to reduce the human errors associated with subjective elicitation, providing a simple method for the estimation of the consequences.

Table 4.1: Quality rating of design requirements, and the associated dispersion β , as per FEMA P-695 (2009).

Representation of Collapse Characteristics	Confidence in Basis of Design Requirements		
	High	Medium	Low
High. Index models capture the full range of the archetype design space and structural behavioral effects that contribute to collapse.	Superior $\beta = 0.1$	Good $\beta = 0.2$	Fair $\beta = 0.3$
Medium. Index models are generally comprehensive and representative of the design space and behavioral effects that contribute to collapse.	Good $\beta = 0.2$	Fair $\beta = 0.3$	Poor $\beta = 0.5$
Low. Significant aspects of the design space and/or collapse behavior are not captured in the index models.	Fair $\beta = 0.3$	Poor $\beta = 0.5$	-

The consequences matrix born from the observation that $q(\lambda)$ and $t_{rec}(\lambda)$ are linked with the Resilience (or the loss of resilience) through an hyperbolic functional, as shown in Zobel and Khansa (2014). In fact, recalling equation (4.2) and considering a constant recovery path $\mathcal{P}_{q(\lambda) \rightarrow q_{100\%}}$, it is possible to find in the plane $t_{rec}-q$ a set of hyperbola characterized by the same resilience, see figure 4.5.

$$R = \int q(t) dt \quad LR = \int (1 - q(t)) dt \quad (4.2)$$

Some considerations about the two main variables, $q(\lambda)$ and $t_{rec}(\lambda)$, are required for the estimation of the resilience.

The residual functionality can be easily selected considering that residual functionality associated with each resilience states can be seen as definitely fixed, since they are connected with the life safety of workers. Regarding the recovery time, the problem is different. Firstly, t_{rec} is composed by several aliquotes, which mean value strongly depend on the structural damage, the location of the damage (the less the damaged element is accessible the higher is the required time for its replacement) and the resources available within the Plant to face off and restore the damage. On the contrary, the reconstruction time, being independent from the seismic damage, is a quantity that can be easily estimated by the technical office, on the basis of historical data, or defining the typical operations required for the reconstruction.

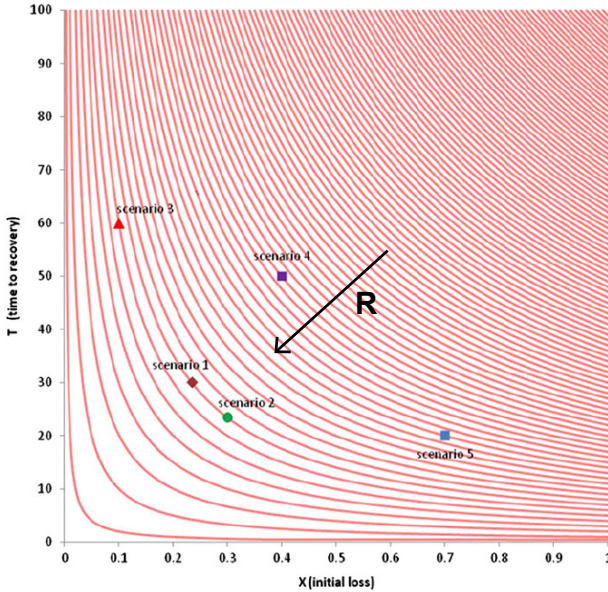


Figure 4.5: Hyperbolic relationships among R - t_{rec} - q .

Considering equally spaced intervals of the recovery time and the residual functionality, it is possible to re-adapt figure 4.5 to a discrete values matrix. Hence, each resilience state can be associated with a specific position within the matrix. The residual functionality is represented as a fraction of the fully functional condition, while the recovery time as a fraction of the total recovery time T_R .

Theoretically, increasing the number of intervals increases the accuracy of the consequence matrix. Anyway, a dense interval spacing might yield to erroneous, or difficult, choice by the technical staff, with the issue of losing its rapidity.

For that reason, five equally spaced intervals are adopted for both the recovery time and the residual functionality. A lexical meaning is associated to each interval to simplify its selection by the technical staff. In particular:

- Intervals of the residual functionality:
 1. full: 80-90%;
 2. high: 60-80%;
 3. moderate: 40-60%;
 4. low: 20-40%;
 5. null/negligible: 0-20%.

- Intervals of the recovery time, represented as a fraction of the total reconstruction time T_R .
 1. rapid: 10-20%;
 2. fast: 20-40%;
 3. moderate: 40-60%;
 4. slow: 60-80%;
 5. complete reconstruction (CR): 80-100%.

Table 4.2: Example of consequence matrix. The values refers to the mean value of the loss of resilience: lower values mean better condition. The values between brackets represent the bounds of each interval.

$t_{rec} / q(t)$	Full	High	Moderate	Low	Null
Rapid	2.25% (1% - 4%)	4.50% (2% - 8%)	7.50% (4% - 12%)	10.50% (6% - 16%)	13.50% (8% - 20%)
Fast	4.50% (2% - 8%)	9.00% (4% - 16%)	15.00% (8% - 24%)	21.00% (12% - 32%)	27.00% (16% - 40%)
Moderate	7.50% (4% - 12%)	15.00% (8% - 24%)	25.00% (16% - 36%)	35.00% (24% - 48%)	45.00% (32% - 60%)
Slow	10.50% (6% - 16%)	21.00% (12% - 32%)	35.00% (24% - 48%)	49.00% (36% - 64%)	63.00% (48% - 80%)
CR	13.50% (8% - 20%)	27.00% (16% - 40%)	45.00% (32% - 60%)	63.00% (48% - 80%)	81.00% (64% - 100%)

The lower bound of the first interval of the recovery time (rapid recovery time) and the upper bound of the first interval of the residual functionality (full residual functionality) are respectively fixed to 10% and 90% to account for, with enough safety margin, the inspection of the facility.

Crossing the two arrays of intervals yields the consequence matrix, shown in table 4.2, in which is represented the loss of resilience for each intervals. Each position of the matrix is constituted by three value: a mean value, obtained adopting for $q(t)$ and t_{rec} the average value of each interval, and the extremal bounds, obtained crossing respectively the lower and the upper shelf of each two array of intervals.

With such a structure, the technical staff of the Plant has different possibilities for the definition of the loss of resilience, associated with each resilience state. If it is confident with the problem, and it's expertise on the problem is pursued through a strong experience in the field, it can decide to provide specific values for the recovery time and the residual function-

ality, within the selected intervals. Otherwise, the technician can decide to select the mean or the upper bound values for t_{rec} , and the mean or the lower bound for $q(t)$, providing a conservative estimation of the loss of resilience.

4.3 Level 2

From the perspective of the complexity of application, the level 2 represents the standard application of the PBEE framework. For such purpose, the analysis of the critical facilities are conducted neglecting the simplification adopted in the first level. Thus, structural vulnerability and the mean value of the immediate consequences are precisely estimated. Obviously, performing a refined assessment of the critical facilities will remove the errors that affect the outcome of the first level: the human error associated with subjective judgments for the estimation of the consequences and the empirical errors associated with the adoption of simplified models.

In the following the assessment of the structural performance and the estimation of the consequences at the second level is defined.

4.3.1 *Assessment of the structural performance*

The structural behaviour towards the seismic action is assessed through complete nonlinear 3D models. In order to provide a coherent assessment method, the following considerations are accounted for:

- definition of the structural configuration (plan configuration, distribution of masses and stiffnesses over the height);
- coupling with non-structural elements;
- presence of drift sensitive or peak floor acceleration (PFA) sensitive non-structural elements or component;

The type of seismic analysis is selected depending the above-mentioned consideration. In particular:

- When the structural configuration is symmetric and the interaction with equipment and adjacent facilities is negligible, non-linear static analyses are privileged over IDA (Incremental Dynamic Analyses) analyses.
- Uncoupled analyses (non-structural element and component not explicitly modelled) are preferred over coupled analyses. Coupled analyses are used when non-structural elements and components might strongly modify the seismic behaviour. Coupled Non-linear static analyses are

used to account for the influence of drift-sensitive non-structural elements. Coupled non-linear time history analyses are performed to account for the influence of PFA-sensitive non-structural elements.

- in the case of PFA sensitive facilities, which configuration does influence the dynamic behaviour of the main structure, the main issue is associated with the definition of floor response spectra. Several simplified procedures are proposed in literature, but they lack in a correct estimation of the effective response spectra, as shown in Pinkawa et al. (2014). Therefore, the following approach is adopted:
 - definition of a set of accelerograms consistent with the requirements of the Eurocode 8; the three principal components must be accounted for, in order to consider the influence of torsional/mixed mode shapes;
 - execution of linear time history analysis on the main structures, adopting a suitable set of real accelerograms, to extract the floor response spectra;
 - execution of a pushover analysis on a sub-structured model representative of the PFA non-structural element;
 - definition of the seismic fragility in the ADRS plane adopting the capacity spectrum method.

4.3.2 Estimation of the consequences

At the second level, the consequences have to be correctly estimated. Thus, as previously mentioned, a strict cooperation among the staff of the technical office and the team of Engineers is compulsory. The following aspects have to be accounted for:

- when the facility remains in the RS_0 and the structural damage is negligible, the time for the inspection can be considered equal to one day. In this case, the assessment of the staff of the technical office is enough to establish the structural safety and declare the facility functional. In the other cases (facility belonging to RS_1 to RS_3), a survey performed by a technical expert is mandatory. Since an external team of experts has to perform the inspection, a minimum time of one week is expected.
- The installation of safety measures foresees the following steps:
 - detachment of the facility from the network, e.g. removing the connection with pipelines or other systems;
 - securing of all the supported equipments and components;

- introduction of temporary safety measures if the bearing capacity of vertical action is compromised.
- Design of the replacement/reconstruction interventions.
- Execution of the replacement/reconstruction. In this stage, typical values required for the realization of structures are accounted for. In particular, the Engineer's team will provide all the informations about the time for the replacement of structural components, including the time required to remove non-structural elements and equipments that may obstruct the operations. The technical office, basing on historical data, provides the expected time required for the substitution of equipment, including the time required to order and receive additional material to external companies.

A much more detailed assessment of the facilities also allows the identification of potential *temporal* redundancies. For instance, let's consider a supported silo, critical for the full operation of the plant. The silo is fully filled every month. The seismic assessment showed a high vulnerability of the silo. To reduce its vulnerability, it is reasonable to reduce the total amount of material beared by the facility, while increasing the frequency of filling. By this mean, the system functionality is not compromised, but the seismic resilience is increased, against minimal modifications of the whole system and additional expenses for the filling procedure.

Furthermore, alternative paths that might be adopted when a facility/-component fails have to be individuated by the technical office, in order to provide suitable solutions to enhance the resilience.

4.4 Level 3

The level 3 aims at providing specific knowledge about the structural behaviour of critical details. If required, hypothesis about the seismic upgrading interventions are performed. By this means, it is established if the improvement of the structural performance can effectively enhance the seismic resilience of the entire Plant.

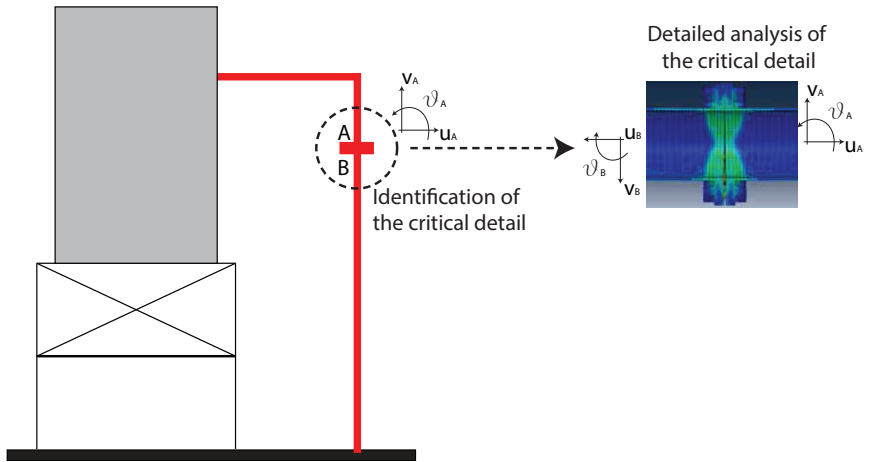


Figure 4.6: Example of detailed assessment by means of the sub-structuring method.

4.4.1 Analysis of the critical details

The critical details individuated at level 2, are investigated in deep at the third level. In particular, detailed finite element models will be adopted in this task. The analyses will be performed adopting the sub-structuring approach, see figure 4.6. The deformation histories, either monotonic or cyclic, will be applied to the investigated component. The following aspects can be taken into account:

- influence of the stiffness of connection on the buckling load of compressed members;
- effective load bearing capacity of steel joint connections, bolted and welded;
- maximum detected crack in steel components, with particular interest to vessel, pressure equipment and pipelines;
- detailed assessment of the collapse mechanism of concrete members.

4.4.1.1 Definition of seismic upgrading procedures

Dealing with the seismic vulnerability, seismic upgrading or retrofit can be attained with local or global interventions. Since the procedure focuses on the resilience assessment, the third level does not aim at providing a detailed

overview of the seismic upgrading solutions, but at defining the seismic resilience enhancement that can be obtained by means of upgrading/retrofit interventions. Therefore, in such step, the interest is to modify the fragility curves, assuming enhanced structural capacity by means of the adoption of common mitigation strategies.

To this aim, it is assumed to modify the vulnerability index I_v , given as the demand over capacity ratio in terms of peak ground acceleration (PGA). The demand is calculated as the expected PGA associated with a 10% PoE calculated within the nominal life. The capacity is represented by the mean value of the fragility curve.

The effect of the upgrading is taken into account, scaling up I_v to the unit; the mean S_a of each limit state is scaled up of the same quantity. The standard deviation remains unvaried. This could represent a strong assumption, since the enhancement of the seismic capacity modifies the collapse mechanism, aiming at a dissipative ones associated with a well known behaviour, and, if necessary, the dynamic properties as well. The first aspect, according to FEMA P-695 (2009), should lead to a lower dispersion in the fragility curve. Regarding the second aspect, the epistemic uncertainty associated with the selection of accelerograms and their record-to-record variability might modify the total dispersion of the fragility curve.

Anyway, it should be considered that, except the case of fragile collapse in the elastic branch, the standard deviation of the actual state's collapse mechanism, being associated with an higher amount of uncertainties (e.g. associated with the material characteristics) should be higher than the retrofitted case (which collapse mechanism is theoretically known, being specifically designed), leading to conservative results.

4.5 Optimization procedure

Industrial plants are frequently characterized by heterogeneous facilities, associated with different level of vulnerability and different consequences in case of damage or collapse. Therefore, they have a different weight on the Resilience of the whole plant.

Within this perspective, the detection of the facilities on which further investigations and, when required, seismic upgrading interventions are necessary, requires a tool enabling to provide a hierarchy among facilities, based on a quantitative metrics representative of the resilience.

To this aim the following aspects are introduced:

- the break even point;
- the resilience indicators;

- the optimal resilience condition.

All the three aspects are mutually connected between each other. The break event point is compulsory for the definition of the optimal resilience condition, necessary for the estimation of the resilience indicators.

4.5.1 Break even point

In microeconomics, the break event point, hereinafter called BEP, is the point at which total cost and total revenue are equal.

This concept can be adapted for the resilience assessment, considering the following plausible hypotheses:

- the fixed costs and variable costs cannot be modified;
- the production volume is constant;
- the BEP, expressed in days of Plant's shut-down, depends on the amount of days at which the production is interrupted for the following reasons:
 - maintenance of the structures and equipment, during which the productions should be shut down to ensure the safety of workers; this value is fixed by the owner of the plant to guarantee the correct functioning of each equipment;
 - the forced shut down of the production due to unforeseen events; as usual, expecting only mechanical failures, such temporal value strongly depend upon the quality of the periodic maintenance; also in this case, the owner provides a threshold value.

Being the only value influenced by the structural vulnerability, the forced shut down due to unforeseen events is set as reference value for the estimation of the BEP and is called hereinafter t_{BEP} . It represents the average number of days of shut down due to unforeseen events per year the Plant can withstand. With such meaning, t_{BEP} can be directly compared with the EAL, obtaining one of the following conditions in which the plant might reside:

- resilience-neutral condition: the analysis of the actual state provides an EAL equivalent to t_{BEP} ;
- resilience-seeking condition: the analysis of the actual state provides an EAL lower than t_{BEP} ;
- resilience-aversion condition: the analysis of the actual state provides an EAL greater than t_{BEP} .

Theoretically, the Plant's owners intend to achieve a zero loss condition, or, in other words, reduce t_{BEP} as much as possible. Such an objective can be achieved in two ways: *a priori*, improving the seismic structural performance of stand-alone facilities, *a posteriori*, improving the response to the emergency conditions.

Therefore, the aim of the owner to lower t_{BEP} as much as reasonably possible has an immediate consequence: depending on the actual state, the resilience-seeking condition is never attained.

4.5.2 Resilience indicators

The aim of the Resilience Indicators (RI) is to provide a classification based on scalar-values of simple interpretation. The Resilience Indicators, from a practical perspective, scales the consequences of each facility to achieve the resilience neutral condition. Thereby, the resilience curve is scaled so that the Plant's EAL is equal to t_{BEP} .

The Resilience Indicators provide a hierarchy among facilities basing on the weight they have on the resilience-neutral condition.

Considering DV_{Req} , the magnitude of the decision variable required to achieve t_{BEP} and DV_{Cur} , the value of the decision variables (t_{rec} , $q(t)$,) in the current state, RI is defined as

$$RI = \frac{DV_{Req}}{DV_{Cur}} \quad (4.3)$$

Achieving the neutral condition requires a modification of the decision variables representative of the loss of resilience. In practice, t_{rec} , $q(t)$ or the effective loss of resilience LR can be adopted.

The parameters that can be modified are $q(t)$ and t_{rec} . The former is directly connected with the safety of workers and, considering its deep connection with current Standards, it makes no sense to modify it. On the other side, t_{rec} is influenced by a variety of parameters, such as the availability of materials, the availability of plausible interventions concepts that might ease the stages from the design to the replacement of the damaged elements.

Recalling the consequences matrix and the intervals in which the recovery time is partitioned, it is possible to define a resilience indicators matrix, as shown in table 4.3. Considering, for the sake of clarity, the case of a single facility which seismic structural damage requires a *moderate* t_{rec} (capacity),

and focusing only on the recovery time⁴ the following situations can be observed:

- $DV_{Req} = DV_{Cur}$: the resilience-neutral condition is achieved and therefore RI is equal to 1;
- $DV_{Req} \leq DV_{Cur}$: in this case the facility resides in the resilience-aversion condition; the resilience-neutral condition is achieved scaling down DV_{Cur} and therefore RI assumes a value lower than 1.
- $DV_{Req} \geq DV_{Cur}$: the facility, residing in the resilience-seeking condition, provides a better resilience performance than the resilience-neutral condition, yielding a RI hypothetically greater than 1.

With such a framework, the lower is RI the higher is the attention required, and detailed assessment is compulsory. Facilities associated with $RI \geq 1$ do not require, theoretically, any further investigations.

Table 4.3: Matrix of the Resilience indicators: on the horizontal direction is reported the capacity (value of the decision variable used for computing the actual state), and on the vertical direction is reported the demand (value of the decision variable required to reach the neutral condition).

	CR	Slow	Moderate	Fast	Rapid
Rapid	0.17	0.21	0.30	0.50	1.00
Fast	0.33	0.43	0.60	1.00	2.00
Moderate	0.56	0.71	1.00	1.67	3.33
Slow	0.78	1.00	1.40	2.33	4.67
CR	1.00	1.29	1.80	3.00	6.00

4.5.2.1 RI for interconnected facilities network

Defining RI encompass the scaling of the consequences to find a solution with an EAL close to the t_{BEP} .

A solution, as simple as trivial, is to scale down the recovery time of each facility with an uniform scale factor SF, as defined in equation (4.4):

⁴ Seen the complexity of its estimation, the residual functionality $q(t)$ is considered as a fixed value.

$$SF = \frac{EAL_{cur}}{t_{BEP}} \quad (4.4)$$

where EAL_{cur} represents the EAL of the plant computed in the current state. By this mean, all the facilities are equally weighted one the Plant resilience, that makes sense if all the involved facilities in the analysis are characterized by equal vulnerability and equal exposure. In common situations, sensitive differences in terms of vulnerability and exposure can be observed among different facilities, because of the heterogeneity and different complexity of each facility. Then, the application of the trivial solution might lead to assignment of high Resilience Indicators to facilities which vulnerability and consequences do not affect the global resilience of the plant.

For the sake of clarity, three examples, characterized by increasing complexity, are shown in the following. In particular, each example is represented by a plant of n units with i facilities (the total amount of facilities in the plant is $m = n \cdot i$), for which different restoration schedule are foreseen:

1. case 1: all the operations are conducted in series;
2. case 2: all the operations are conducted in parallel;
3. case 3: the operation on the facilities within a unit are performed in series, while the unit's operations are conducted in parallel.

Case 1. When the functionality of facilities is restored by means of operations performed in series, the Resilience curve of the whole plant is calculated by the summation of the resilience of each stand-alone facility (figure 4.7). Also the EAL of the Plant is obtained summing the EAL of each facility. The uniform scaling makes sense only if all the facilities have the same weight on the Plant's Resilience. Such a situation happens only when all the facilities have the same EAL, equal to EAL_{plant}/m , being m the number of facilities in the plant.

Anyway, dealing with heterogeneous facilities, aimed at different functions within the Plant, an equally weighted condition is rather difficult to achieve. Therefore, in common configurations, as the one reported in figure 4.7, adopting a uniform scaling leads to a sure misclassification of the importance of the facilities:

- if $EAL_j < EAL_{plant}/m$ the importance of the facility j is overestimated;
- if $EAL_j > EAL_{plant}/m$ the importance of the facility j is underestimated;
- if $EAL_j = EAL_{plant}/m$ the importance of the facility j is correctly estimated.

Case 2. When the restoration schedule foresees only in parallel operations, the Resilience curve is obtained by the envelope of the curves belonging of

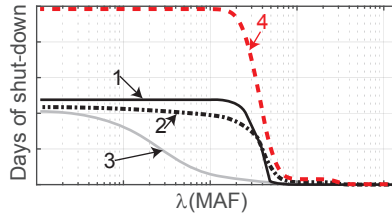


Figure 4.7: Case 1: definition of the EAL of the Plant, in the case of series restoration schedule (a plant constituted of 4 facilities belonging to different units connected in series is considered).

stand-alone facilities. In such a configuration, briefly reported in figure 4.8, the following observations arise:

- not all the facilities' resilience curves envelopes the Plant's resilience curve; is the case of the facility 1, depicted in figure 4.8(a), that does not influence the Resilience curve. In this situation, the uniform scaling would erroneously account for the facility 1 as *risky*.
- Considering the cumulated EAL (see figure 4.8(b)) of the resilience curve, calculated as in the following

$$EAL_{cum}(\lambda) = - \int_{\lambda_{max}}^{\lambda} LR(\lambda) d\lambda \quad (4.5)$$

it appears that the weight of each facility on the total EAL is different and it depends on the range of frequencies in which the resilience curve of the facility envelopes the Plant's resilience curve. Clearly, the lower is such a range of frequencies, the lower is the weight of the EAL of the facility on the Plant's resilience. The case of figure 4.8 shows that the facility 2 provides almost all the Plant's EAL. Therefore, also in this case adopting the uniform scaling appears not convenient.

Case 3. In the case of units restored in parallel, constituted of facilities restored in series, the simplest approach for the definition of the Plant's resilience curve is to calculate before the resilience of each unit, as described in case 1. Then, enveloping the curve of the units provides the Plant's resilience curve.

Such a configuration is affected by all the problems evidenced in case 1 and case 2, and thereby it appear pointless the uniform scaling technique for defining the resilience indicators.

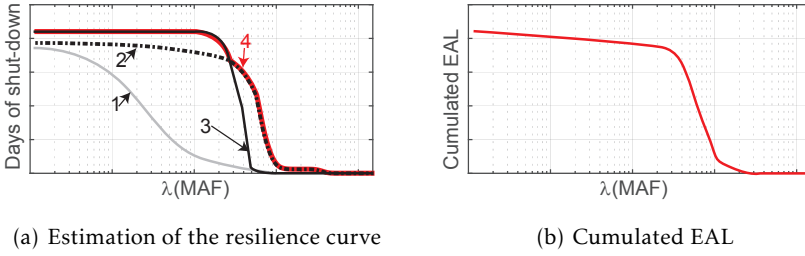


Figure 4.8: Case 2: definition of the a) resilience curve and the b) cumulated EAL of the Plant, in the case of parallel restoration schedule (a plant constituted of 4 facilities belonging to different units connected in series is considered).

It is clear that the definition of resilience indicators of highly interconnected facilities is not straightforward. In fact, depending on the heterogeneous property of facilities, i.e. the seismic vulnerability and the associated consequences, reconstruction schedule and the redundancy at the plant network level, the problem gets more complicated.

Considering that each facility can be associated with different scale factors to achieve the target t_{BEP} , and theoretically infinite set of curves, with different shapes but same EAL, can be identified, as shown in figure 4.9, being the problem unconstrained. Therefore, some additional boundaries and constraints are required and are described in the following.

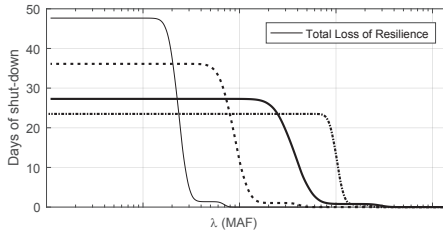


Figure 4.9: Comparison of four different shaped resilience curve, but characterized by the same EAL.

4.5.2.2 Optimal resilience condition

As it has clearly evidenced before, defining RI for interconnected systems requires the identification of an optimal resilience curve. Such a curve rep-

resents an improved situation of the Plant, in terms of resilience, in correspondence of which the t_{BEP} is attained.

In the following, a method for the definition of the optimal curve, that fits with the requirement of the t_{BEP} , is shown. The concept of stochastic dominance, iso-resilience condition are firstly introduced, providing then the method proposed for the definition of the optimal resilience curve.

Stochastic dominance. Stochastic dominance (SD) is a tool commonly adopted in portfolio insurance to compare two portfolios, \mathcal{F} and \mathcal{G} , which loss curve shape, respectively $u(\lambda)$ and $v(\lambda)$ ⁵ is unknown. It is based on the fact that insurer have risk-aversed attitude, meaning that they tend to prefer portfolio that provide greater outcome, and thereby are associated with the lowest losses.

Stochastic dominance is composed of two criteria, that have to be fulfilled simultaneously in order to prove that a portfolio is *better* than another:

- **First-order stochastic dominance criterion (FSD):** \mathcal{F} is preferred over \mathcal{G} if the following relation is satisfied:

$$u(\lambda) \leq v(\lambda) \quad \forall \lambda \quad (4.6)$$

- **Second-order stochastic dominance (SSD):** \mathcal{F} is preferred over \mathcal{G} if the following relation is satisfied:

$$\int u(\lambda) d\lambda \leq \int v(\lambda) d\lambda \quad (4.7)$$

Equation (4.7) explains that a portfolio is preferred over others whether it has the lowest EAL.

A graphical example of the two stochastic dominance criteria is provided in figure 4.10, in which three curves are depicted. Equation (4.8) reports the correlation among the EAL of the three curves:

$$EAL_1 < EAL_2 < EAL_3 \quad (4.8)$$

Adopting the two SD criteria, it can be concluded that:

- curve 1 is better than curve 2, since its shape is always the lowest among the two curves, and has a lower EAL;

⁵ it has been implicitly assumed that the loss curves are function of the seismic mean annual frequency λ .

- curve 2 is better than curve 3 only from the EAL perspective ($EAL_2 < EAL_3$);
- comparing curve 1 and 3, nothing can be said, since FSD is satisfied in favour of curve 3, and SSD is satisfied in favour of curve 1.

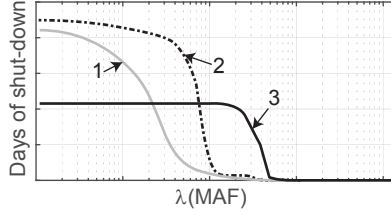


Figure 4.10: Example of three resilience curve for the application of the stochastic dominance criteria (the lognormal scale is adopted for the x axis for a better comparison at low frequencies).

Iso-resilience condition. Considering the Plant in a resilience-seeking condition and aiming at achieving the neutral-resilience one, the iso-resilience represents the condition at which each facility has the same weight on the estimation of the EAL of the Plant, expressed in terms of loss of resilience. Once the iso-resilience condition is attained, the Plant's EAL equals t_{BEP} . A distinction among the reconstruction schedule is required. In fact, the definition of the iso-resilience condition strongly depends upon the adoption of in series or in parallel reconstruction schedule. Considering the case of n facilities on which operations are performed in series, the EAL of each facility associated with the iso-resilience condition assumes the value in equation (4.9):

$$EAL_{iso} = \frac{EAL_{plant}}{n} \quad (4.9)$$

where EAL_{plant} represent the EAL of the entire Plant, expressed in terms of loss of resilience.

In the case of in parallel operations, an additional step is required. In fact, the critical facilities are only the ones which curves envelopes the Plant's resilience curve. Given n the amount of critical facilities, EAL_{iso} is then computed according to equation (4.9). For the purpose of comparison, the EAL of single facilities should be assessed only in the frequencies range in which the facility's curve envelopes the plant's curve.

Constraints on the scaling factors. The recovery time of a facility, as previously defined, is bounded between T_{insp} (time required for the inspection) and T_R (time for the complete reconstruction).

Procedure to define the optimal resilience curve. First of all, the problem requires some simplifications.

For each i^{th} facility, one state function curve $lr_{RS,j}$ is given for each resilience state RS. Each $lr_{RS,j}$ can be associated with an estimation of the EAL, specific for that RS:

$$eal_j = \int lr_{RS,j} d\lambda \quad j = RS_0, RS_1, \dots, RS_{3,b} \quad (4.10)$$

Is also valid the following relation

$$EAL_i = \sum_{j=1}^4 lr_{RS,j} \quad (4.11)$$

being EAL_i the EAL associated with the i^{th} facility.

In order to simplify the problem, instead of considering the recovery time associated with each RS, the averaged recovery time $t_{ave,i}$ over eal_j is considered. It is calculated as in the following:

$$t_{ave,i} = \frac{\sum_{j=1}^4 t_{rec,i,j} \cdot eal_{i,j}}{EAL_i} \quad (4.12)$$

Recapping the aforementioned observations, the following steps are required:

1. identification of the system network and build-up of the block diagram;
2. definition of the restoration schedule;
3. definition of suitable numerical models;
4. perform the structural analysis;
5. define the reconstruction time;
6. identify the consequences by means of the consequences' matrix;
7. identify the critical facilities, their frequency range λ_r in which they envelope the plant's resilience curve and their $EAL_i(\lambda_r)$;
8. define the average recovery time of the actual state
9. define the iso-resilience condition;
10. fix the scale factors boundaries, based on the recovery time.

Given the EAL in the scaled situation, the optimal solution is searched minimizing the SRSS (square root sum of square) of the error between the EAL_{iso} and the scaled EAL. The given solutions must respect the constraints imposed by the stochastic dominance condition. A graphical interpretation of the optimization procedure is given in figure 4.11, in which are shown the range of variability of the EAL (left) and two possible scaled curves (right): the grey one fulfill both the SD criteria and therefore is called *feasible solution*; the black dashed line fulfill only the first SD criterion and therefore is called *unfeasible solution*.

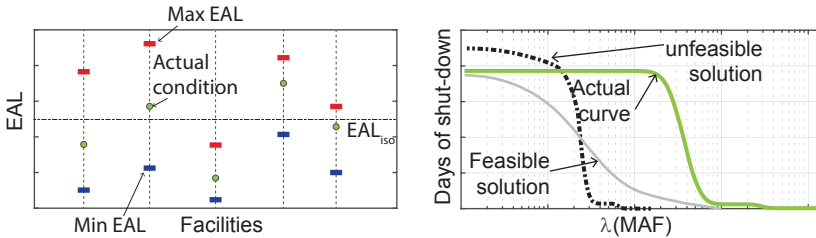


Figure 4.11: Example of iso-resilience condition.

4.5.3 Selection of the algorithm for the optimal resilience condition

It's clear that the definition of the optimal resilience condition requires a optimization algorithm, able to define, among all the valid solutions, the global minima condition, respecting the imposed nonlinear constraints.

As it is conceived, the problem is characterized by

- the presence of lower and upper bounds, i.e. the imposition of thresholds to the EAL of each stand-alone facility;
- the existence of non-linear constraints, e.g. the adoption of the stochastic dominance;
- the possible existence of multiple states at which the SRSS is minimized.

As it is conceived, the problem might be solved by means of an local optimal point, that can be strongly different from the global optimal point. For this reason, local optimization algorithms, that essentially seek for an optimal solution in the neighborhood of a initial starting point, are not suitable

for the problem.

Notwithstanding the major complexity, genetic algorithms represent a global optimization tool that might overcome the aforementioned problems, ensuring the respect of the linear (lower and upper bounds) and nonlinear (stochastic dominance) constraints.

Functioning of Genetic Algorithm (GA). Genetic algorithms are based on Darwin's theory of natural selection. The specific mechanics of the algorithm uses the language of microbiology, and its implementation mimics genetic operations.

The basic idea of the approach is to start with a set of designs, randomly generated using the allowable values for each design variable. Each design is also assigned a fitness value, usually using the cost function for unconstrained problems or the penalty function for constrained problems. From the current set of designs, a subset is selected randomly with a bias allocated to more fit members of the set. Random processes are used to generate new designs using the selected subset of designs. The size of the design set is kept fixed. Since more fit members of the set are used to create new designs, the successive sets of designs have a higher probability of having designs with better fitness values. The process is continued until a stopping criterion is met. In the following paragraphs, some details of implementing these basic steps are presented and explained.

Chapter 5

The Steel Mill

5.1 Introduction

The chapter describes, first, the plant configuration and the facilities of the steel mill case study, supplying all the information required for the application of the resilience assessment procedure, including the definition of the seismic hazard.

The plant is then analysed from the perspective of the production process so to provide a proper scheme of the system network, necessary for the following resilience assessment. On the basis of the plant configuration and the production work-flow, the restoration schedule has been defined.

Finally both the detailed and the multilevel procedures are applied separately for the assessment of the plant resilience. The application of such procedures to the steel mill case study highlighted advantages and disadvantages, properly described at the end of the chapter.

5.2 Description of the case study

5.2.1 *The Plant configuration*

The case study is a portion of a steel mill that produces steel rebars for R.C. constructions and steel rod wires. The production work-flow is composed of three main parts: production of steel lingots, processing of steel lingots and storage of the finished material. Since a part of the steel mill is currently to object of a wide renovation program, the analyses ha been carried out on the

portion associated with the production of steel lingots.

The complete production process of the steel lingots requires the addition of basic material, provided by steel scraps, the addition of alloys in a limited quantity and inter gases for activating the electric arc furnace. Five main units can be highlighted, as shown in figure 5.1: the water treatment (unit 1); the nitrogen-argon storage vessels (unit 2); the main building (unit 3); the dust filter (unit 4) and the additional alloys storage unit (unit 5).

It is important to highlight that the connection among the facilities does not foresees any source of redundancy. This aspect strongly influences the resilience of the whole plant, considering that no alternative paths are foreseen.

5.2.2 Description of the facilities

In the following, a brief description of the facilities that will be analysed in detail is provided.

Main building. The main building is a four bays steel warehouse (see figure 5.2), hosting several heavy cranes for the displacement of lingots and cradle furnaces. It is realized with S235 steel.

The in-bay (X direction) structural behaviour is characterized by cantilever behaviour of the columns; on the perpendicular direction (Y direction), the resistance towards the seismic action is ensured by portal frame bracing systems (see figure 5.2), which disposition is shown in figure 5.3.

The maximum height of the warehouse is close to 26 m, the cranes are placed at height equal to 10.0 m. To withstand the heavy loads transferred by the cranes, main beams are realized by welded sections of height 2.1 m; similarly the columns are realized joining two HEA300 with a plate thick 20 mm and long 1500 mm (see figure 5.4).

The first bay (grids D-E in figure 5.3) is demanded to the production scraps storage, composed of in-ground RC tanks and several cranes aimed at easing the disposition and optimization of the materials. The central bay (grids C-D in figure 5.3) contains the so-called *dog-house* where the cradle furnace is located. It is characterized by refractory perimeter panes to avoid injuries to the workers during the melting phase. The cradle is allowed to slide on a grid of steel beam supported by continuous RC (reinforced concrete) perimeter walls. The elevated temperatures reached during the melting of the steel, enforces the adoption of proper systems to cool down the

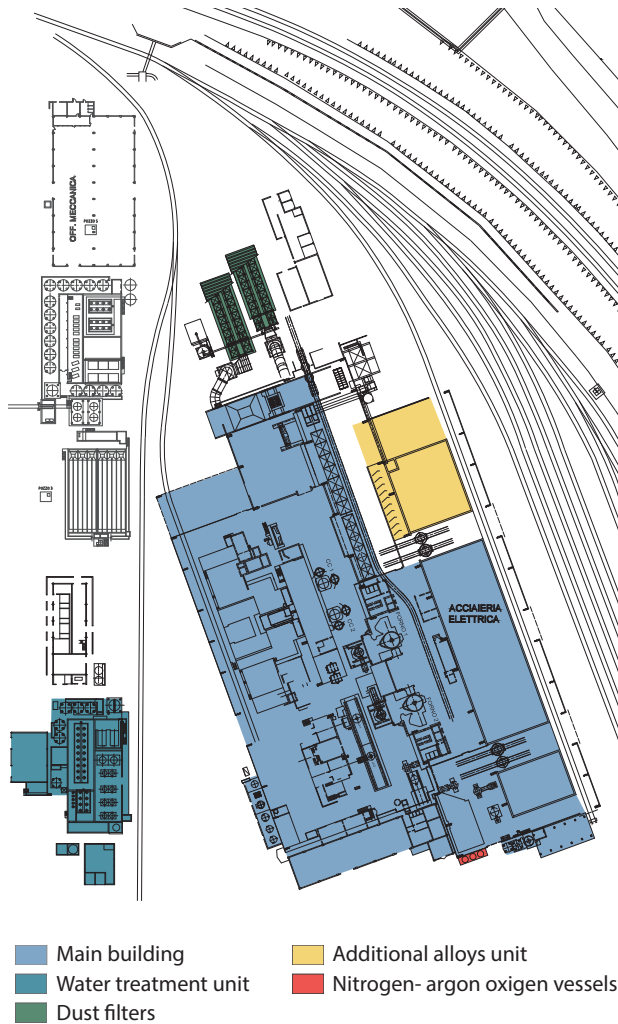


Figure 5.1: Planimetry of the steel mill.

temperature of the supporting beams, to avoid excessive irreversible deformations that may prejudice the functioning of the whole system. All the beams are supported by 80 cm thick RC walls, partially encased on the ground, placed on the perimeter and by internal columns. Its configuration, makes the dog-house not sensitive to the seismic action. The third and fourth bays (grids A-B and B-C in figure 5.3) are demanded to the transportation of

the steel lingots to the successive transformation processes.

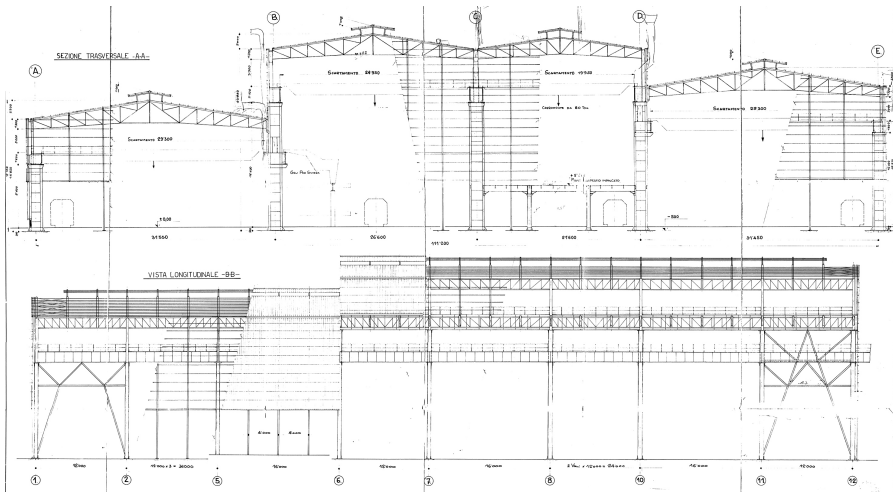


Figure 5.2: Side views of the main building steel warehouse: the spacing among each bay is variable between 16 and 18 meters; top: X direction; bottom: Y direction.

Water treatment. The water treatment unit is composed of three different typologies of structures, connected in series:

1. Two RC MRF (moment resisting frame) structures that support the cooling system, named support 1 and support 2;
2. a RC MRF structure supporting a mud container.
3. a set of silos with the function of sand filters;

All the RC MRF structures have been designed and built during the first years of the 1970. Regarding the materials, a concrete class $C250$, with a characteristic cubic compression resistance, according with the current seismic regulations, equal to 300 kg cm^{-2} ; the steel rebars are realized with FeB44 steel; according to the regulations used during the period of construction (D.M.30/05/1972), this steel type has a characteristic yielding resistance equal to 440 MPa, a maximum tensile resistance equal to 500 MPa and a maximum elongation at fracture close to 12%.

All the facilities belonging to the water treatment unit are connected by means of a network of pipelines, equipped with suitable expansion joints that avoid any kind of coupling.

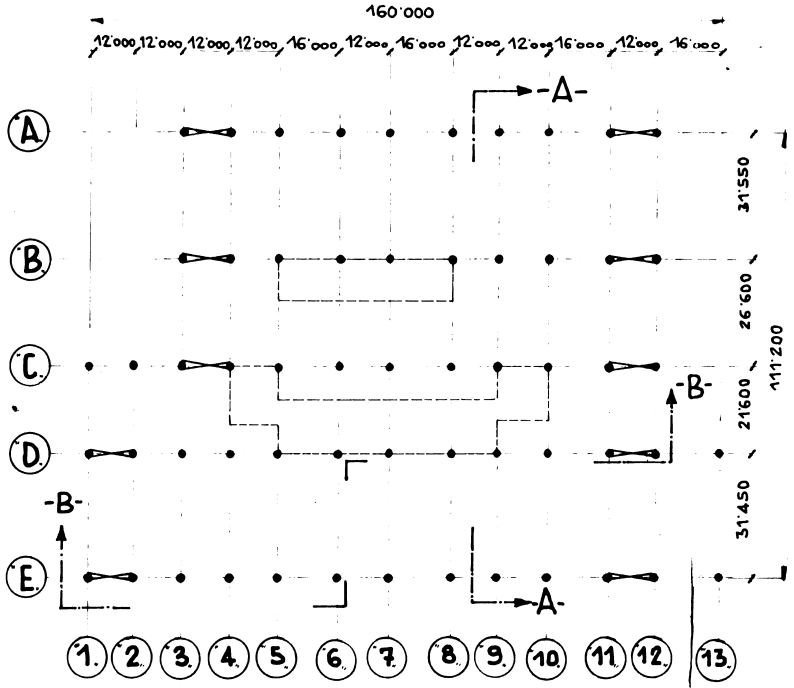


Figure 5.3: Plan configuration of the bracing system.

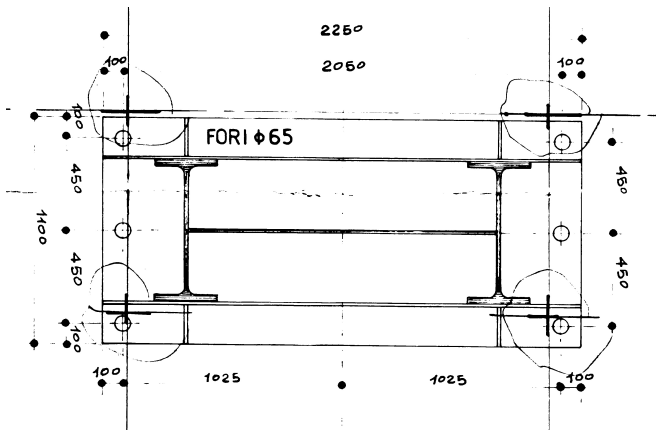
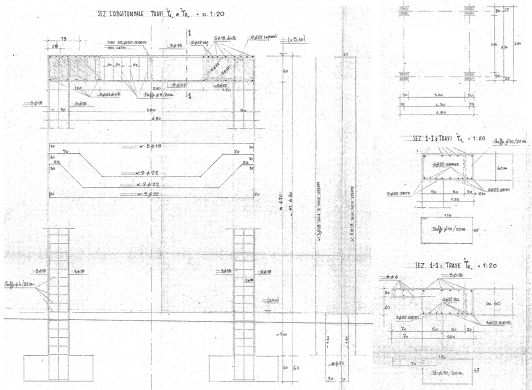


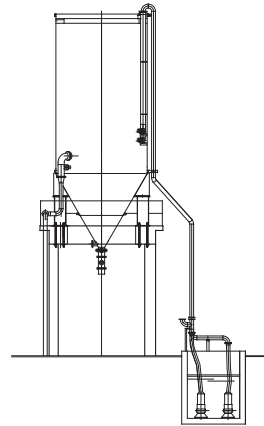
Figure 5.4: Section adopted for the columns.

Regarding the structural geometry, further information are provided in the following.

1. **Mud container** supporting structure (see figures 5.5(a) and 5.5(b)). RC Columns 5 m high have a section of 50x50 cm and 8 ϕ 18 longitudinal rebars, while ϕ 6 stirrups are placed at a constant 20 cm spacing. Beams have a section equal to 120x50 cm, with 5 ϕ 22 longitudinal rebars at the ends placed on top and bottom of the beam; the mid part of the beam has four additional ϕ 22, for a total of 9 ϕ 22 longitudinal rebars at the bottom side. The stirrups are ϕ 8, uniformly spaced of 20 cm over the entire beam. To ensure additional shear resistance, the additional four ϕ 22 are bended at 45° at ends. The structure is beared on four 160x160 cm footings. The connection among the main structure with the mud container is ensured by the four M50 anchor bolts on each beam.



(a) Original drawings of the mud container.



(b) Assembly mud container - support structure

Figure 5.5: Mud container: original drawings and assembly.

2. **Support 1** (figure 5.6) is a two bays simple RC MRF frame while **Support 2** (figure 5.7) is a four bays RC MRF structure. As shown in figures 5.6 and 5.7, the geometry of the structural elements is very similar and therefore they will be described together. Columns of height equal to 500 cm are realized with a 50x50 cm section and 8 ϕ 18 longitudinal rebars, while ϕ 6 stirrups are placed at a constant 20 cm spacing. Three beams typologies are adopted:

- Beams $T1$ and $T2$ have a 50×43 cm section; the ends are symmetrically reinforced with $4\phi 18$ at the top and the bottom; $\phi 8$ stirrups 25 cm uniformly spaced are ensured over the whole beam.
- Beam $T3$ is the perimeter beam on which the load is transferred from the supported cooling towers to the main structure. They have a 40×50 cm section; the ends and the mid section are symmetrically reinforced with $4\phi 16$; open $\phi 8$ stirrups with a constant spacing of 25 cm are ensured over the whole beam. Additional shear resistance is ensured by $2\phi 16$ bended longitudinal beams nearby the corners. This beams have a an additional wing sticking out of approximately 90 cm, on which workers might perform maintenance and inspections.

The support 1 and support 2 are designed to withstand 1800 kg m^{-1} on beams $T2$ and $T3$, 720 kg m^{-1} on beams $T1$, along with the horizontal wind load, assumed equal to 100 kg m^{-2} and 500 kg m^{-2} on the wings of beam $T3$, to account for the maintenance.

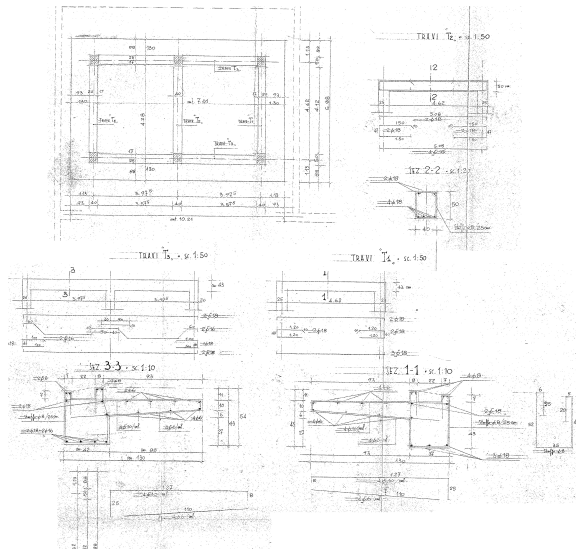


Figure 5.6: Original drawings of the support 1.

3. The silos sand filters (see figure 5.8) are realized with S235 steel. Each silo is supported by four columns, approximately high 150 cm, realized with CHS section 406.8x8 section. The diameter of each silo is equal to 4800 mm and approximately high 6000 mm, including the four columns.

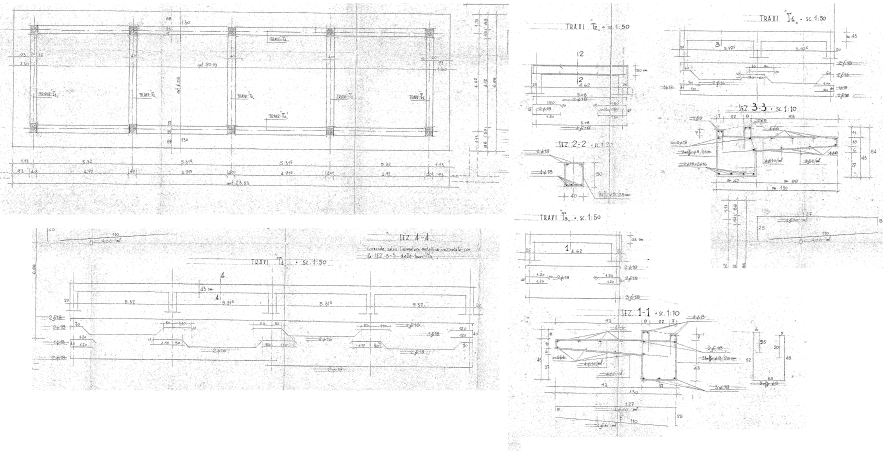


Figure 5.7: Original drawings of the support 2.

The weight of the sand contained in the silos is assumed equal to 20 kNm^{-3} and the silos are always completely filled.

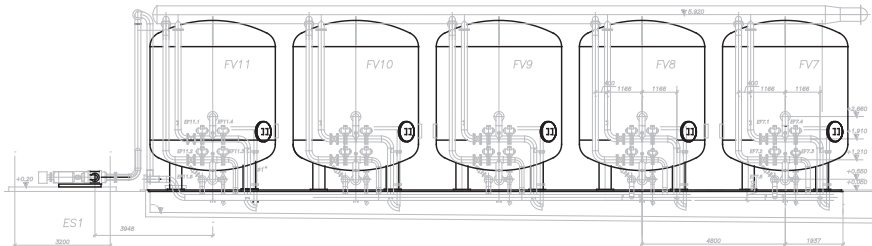


Figure 5.8: Original drawings of the silos.

Storage of additional alloys. The unit for the storage and mobilization of additional alloys is composed of 2 storage silos, respectively referred as **silos 1** and **silos 2**; a **supporting tower** for equipments and 2 belt conveyors, **BC1** and **BC2**, that respectively connect silo 1 to silo 2 and silo 2 to the supporting tower.

From the production process perspective, the five facilities are connected together in series.

The **silos 1** has been realized in the early of the 1980s. It has been designed to withstand only the vertical load. It represents a typical example of industrial facility realized to withstand only the vertical action and has a structural configuration to maximize the functionality and a flexible arrangement of additional equipment. A steel silo, completely realized with steel plates, opportunely stiffened, is used to store coke; it is supported by 8 columns HEA300 9.82 m high. All the seismic resistant structures are realized with S275 steel; two different resisting systems toward the seismic action are adopted in the two directions, as shown in figure 5.10. Along the X direction a MRF system is adopted, while on the Y direction a X bracing system, realized with L110x10, is introduced. To guarantee a satisfactory flexibility regarding the disposition of equipments and the belt conveyors beneath the silo, the X braces are connected from mid height to the top of the columns.

The **silos 2** (figure 5.11), compared to the former one, is relatively new. It has been built in the 2006 following the prescription of the OPCM 3274, 2003, accounting for a non-dissipative behaviour towards the seismic action, therefore the prescription of the capacity design has not been adopted. The steel silo, that stores mainly lime dust, is suspended at above 4.5 m by means of 16 HEA200 columns. The columns are pinned at both the ends and lay on RC supports 67 cm high and a 60x60 cm section. Both the directions are characterized by a concentrically braced system to withstand the seismic action. In the X direction X braces, realized with L110x10 are placed every two bays, while in the Y direction, inverted V braced are adopted, using 2C130x12. In both the directions the braces are bolted to the columns.

The **supporting tower** is a one-bay three storeys CBF structure. At the last storey an heavy pump, that weigh 800 kN, is placed to transfer the additional alloys from the silos 1 and 2 to the cradle furnace. The X bracing system in both the directions is realized with S235 L80x6. The columns are S235 HEA200.

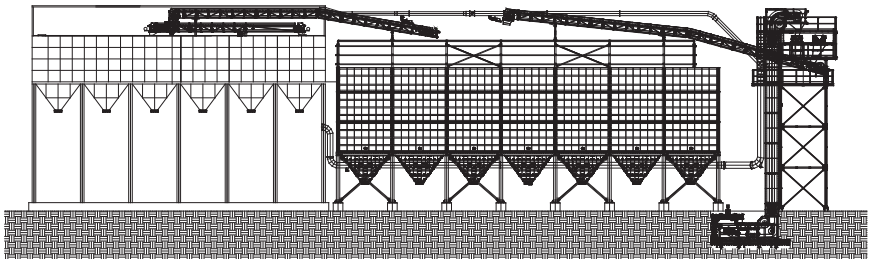


Figure 5.9: Shop drawings of the additional alloys unit.

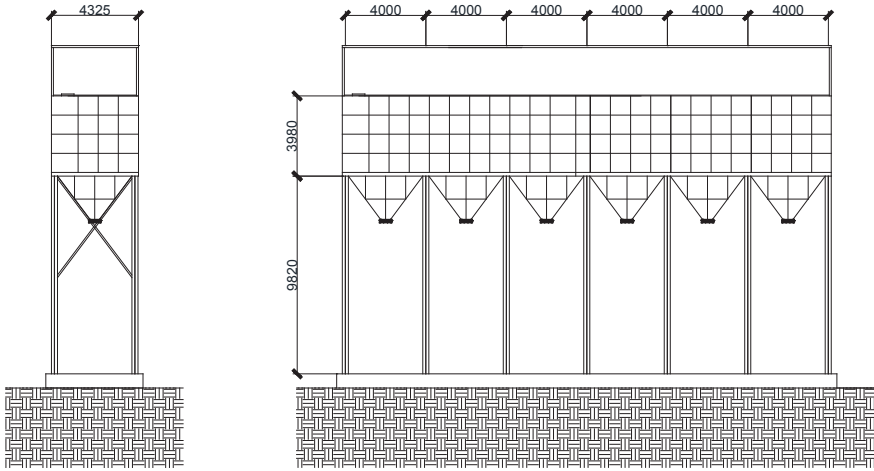


Figure 5.10: Schematic drawings of the silo 1.

The BC1 and the BC2 respectively connect the silo 1 to the silo 2 and the silo 2 to the supporting tower. Its main function is to store the material within the silos. They are characterized by a three dimensional lattice structure, realized with S235 L40x4 welded together, supported by four steel columns, two per each ends, realized with UPN100 for the BC1 and HEA100 for the the BC2. The columns are pinned to the silos and fixed, at the top ends, to the lattice structure.

Storage of nitrogen, argon and oxygen. This unit is constituted by four pressure vessels (see figure 5.12) with a total capacity of 30.000 liters, 2.78 m diameters and total height equal to 8.75 m. The working pressure is set to

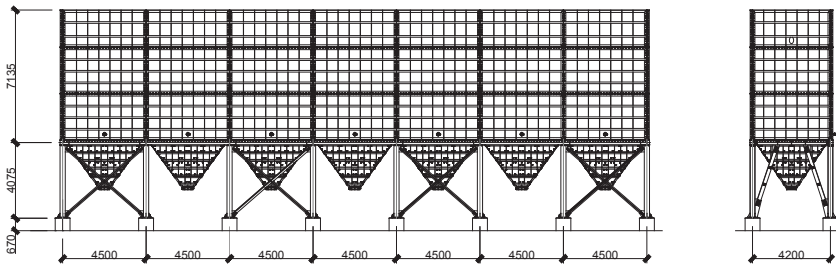


Figure 5.11: Silo 2 (additional alloys storage)

14 bars, while the maximum pressure the vessels can withstand is equal to 18 bars. The weight of the empty vessel is set to 135 kN, while the full vessel weigh 480 kN. Each silos is withstood by three HEA300 columns high approximately 80 cm. The connection of the columns with the bottom of the vessel is welded, providing a thicker plate around the connection to avoid possible punching phenomena. The columns are anchored to the ground with a bolted connection that ensure a pinned behaviour.

The four vessels are connected between with the cradle furnace by means of a system of pipelines DN32, realized with S235. The pipelines are placed above the ground; expansion joints are not adopted to decouple the pipelines behaviour from the one of the vessels.

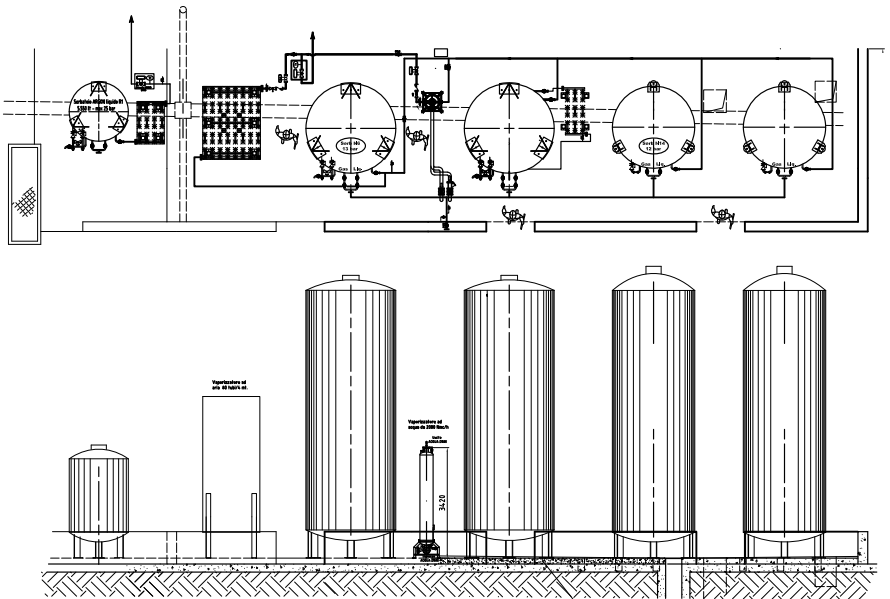


Figure 5.12: Overview of the four silos and disposition of the pipelines

5.3 Seismic hazard

The Steel Mill does not contain hazardous materials neither produces hazardous wastes, therefore, according to the EN1998-1 (EN 1998-1, 2005), an importance factor γ equal to one can be adopted. From the perspective of the Italian regulations, such a choice yields to consider a nominal life of 50 years.

For the purpose of the analyses, the Plant is considered located in a high seismicity area in Turkey (long: 27.067457°, lat: 38.846017°), graphically shown in figure 5.13. By means of the REASSES2 (Iervolino et al., 2016) software, a seismic hazard analysis has been performed, defining the uniform hazard spectra (UHS) and the hazard curve as well. In particular, it has been considered a soil type D, defined according to the EN1998-1 (EN 1998-1, 2005), characterized by a $V_s = 360$ m/s.

Considering a 10% PoE in 50 years and a soil type A, the PGA at the site is equal to 0.321 g. Introducing the soil amplification, the PGA results to be 0.379 g. The seismic hazard curve, determined with the REASSES2 software, is reported in figure 5.14, while the UHS in figure 5.15.

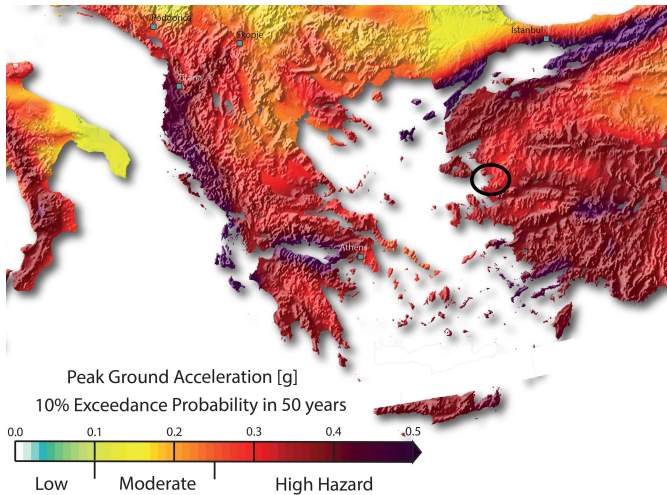


Figure 5.13: Seismic hazard map and identification of the area of interest in which the Steel Mill is considered.

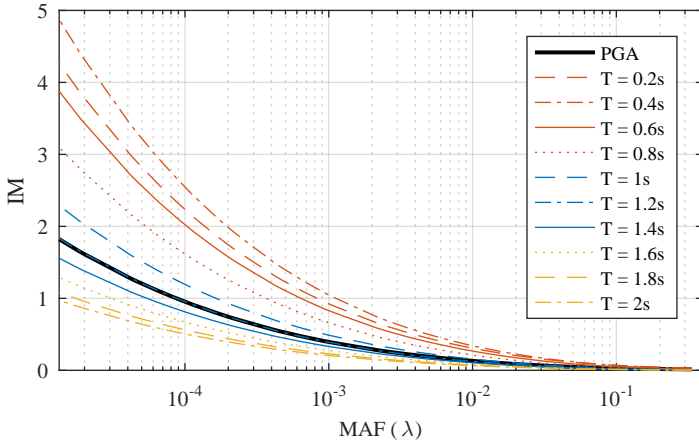


Figure 5.14: Seismic hazard curve considered for the steel mill.

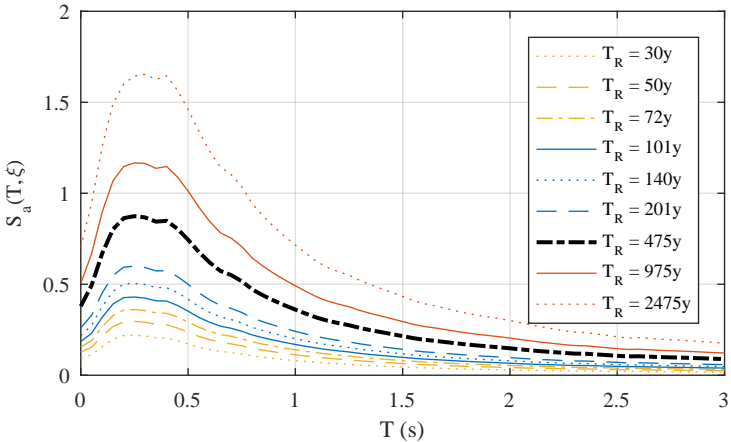


Figure 5.15: Uniform hazard spectra considered for the steel mill.

5.4 Resilience Assessment

In this section, the seismic resilience of the Steel Mill is assessed first by means of the detailed procedure and then by the multilevel procedures. The application of the multilevel procedure was carried out in the hypothesis of having not applied the detailed procedures, so that the results obtained are not influenced by the first procedure. Before the application of bot the

procedures, a brief description of common aspects, the system network configuration and the restoration schedule, is provided.

5.4.1 System network

As already stated in the description of the plant, the facilities are connected between themselves in series. To properly work, the production process requires the full functionality of the whole stand-alone units and facilities and the reduction of the functionality of a single facility leads to a reduced global productivity. Therefore, the block diagram representative of the system network can be represented by a set of units in *series* (see figure 5.16): the failure of one component (unit or facility) leads to the failure of the whole system. Moreover, being the additional alloys and the water treatment units composed by several sub-units, and being such sub-units connected in series, also the failure of a single sub-unit can lead to the global interruption of activities.

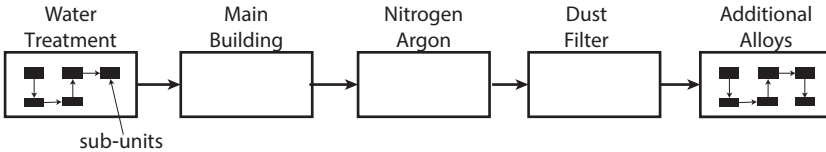


Figure 5.16: Block diagram of the steel mill.

In this situation, the *minimum cut set* is represented by one facility that mostly reduces the plant's resilience. Considering a system network, the loss of resilience depends upon the vulnerability, the consequences, the connection among facilities and the restoration plan. Such values define the weight of each facility on the plant's resilience. It could be that one facility with a high vulnerability is associated with low consequences, being less influencing at the global level than a facility with medium vulnerability and medium consequences. Therefore, the execution of the analyses is required to establish which is effectively the *minimum cut set* of the system.

5.4.2 *Restoration schedule*

The main problem associated with the definition of the restoration schedule, intended as the set of operations to restore the operational condition after a seismic event and their temporal organization, is associated with the interferences among simultaneous operations. Due to the restricted amount of available information, it is assumed that simultaneous operations on neighbor facilities should be executed with a reduced efficiency, to ensure the safety of workers during all the replacement/reconstruction interventions.

On the basis of these observations, the proposed *optimal restoration schedule* can be, rationally, defined as in the following.

First, the operations are performed in parallel on different units. Regarding the facilities belonging to the additional alloys unit, defined in the 5.2.2.

- the operations performed on silo 1, silo 2 and the supporting tower are performed in parallel considering an efficiency equal to 50% due to interferences;
- the operations performed on belt conveyors are performed in series: when the operations on silo 1 and silo 2 are concluded, the operations on BC 1 and then on BC 2 are performed;

For what concern the facilities belonging to the water treatment, the operations on support 1, support 2 and the mud container are executed in parallel, considering an efficiency equal to 50%; the sand filters are replaced when the works on the RC facilities are concluded.

5.4.3 *Detailed Resilience assessment*

The detailed resilience assessment, as described in details in chapter 3, is developed carrying out the vulnerability assessment by means of detailed analyses, defining the consequences in a detailed way and, in conclusion, estimating the resilience and facilities prioritization.

5.4.3.1 **Vulnerability assessment**

The vulnerability assessment, considering the rather simple configuration of the facilities, is carried out through the execution of nonlinear static analyses. The fragility curves are directly derived from the capacity curve by means of the SPO2FRAG tool (Baltzopoulos et al., 2017), realized in the matlab environment. The toolbox transforms, with the SPO2IDA tool (Vamvatsikos and

Cornell, 2005), the pushover curve in IDA curve, from which the fragility is derived. A detailed definition of the procedure is given in sect. 4.2.2.

As stated in sect. 3.3.5, the application of the detailed procedure requires an exhaustive knowledge of the facilities. In this thesis, being impossible the execution of extensive in-field surveys and destructive and non-destructive tests to characterize the mechanical parameters of structural materials, the mean values proposed in current scientific literature, briefly summarized in table 5.1, are adopted for the execution of the analyses.

Table 5.1: Characteristics and mean values of the structural materials used for the vulnerability assessment of the two case studies.

	f_{yk} [MPa]	f_{ym} [MPa]	σ [MPa]	Distribution	Reference
Steel					
S235	235	328	15.9	Gaussian	Badalassi et al. (2017)
S275	275	361	22.9	Gaussian	Badalassi et al. (2017)
Concrete					
C250	25	16.38	8.59	Gaussian	Masi et al. (2014)
Rebars					
Aq42	230	322.34	26.59	Gaussian	Verderame et al. (2011)

All the information on the modelling, the analyses and the results of the vulnerability assessment carried out on the facilities of the steel mill are summarized in the following and reported in appendix A.

The structural check have been performed for the DL, SD and NC limit states; the results are summarized in table 5.2 and are expressed in terms of mean value of the spectral acceleration calculated at the fundamental period T_1 ($S_a(T_1)$), standard deviation (β) and the vulnerability index (I_v), representative of the ratio between the seismic capacity that trigger the collapse and the seismic demand, calculated for a 10% PoE in 50 years. From table 5.2, it can be easily recognized the particular heterogeneity of the facilities. Different collapse mechanism are expected, such as shear failure for RC columns, brace buckling or the maximum plastic rotation of columns for steel structures. Moreover, some facilities, such as the silo 1, the sand filters and the mud container, are highly vulnerable towards the seismic action, i.e. low I_v ; others, such as the belt conveyor 2 and the nitrogen argon vessels, requires a seismic action twice the design value to attain the collapse.

Table 5.2: Summary of the results of the detailed vulnerability assessment.

	Facility Mechanism	$S_a(T_1)$	β	I_v
Mud container	DL	shear failure	0.21	0.09
	SD	shear failure	0.21	0.09
	NC	shear failure	0.21	0.09 0.24
Support 1	DL	shear failure	0.53	0.07
	SD	shear failure	0.53	0.07
	NC	shear failure	0.53	0.07 0.61
Support 2	DL	shear failure	0.27	0
	SD	shear failure	0.27	0
	NC	shear failure	0.27	0 0.32
Sand filters	DL	Sliding failure	0.08	0
	SD	Sliding failure	0.08	0
	NC	Sliding failure	0.08	0 0.1
Silo 1	DL	Brace buckling	0.01	0
	SD	Connection failure	0.02	0
	NC	Connection failure	0.03	0.02 0.02
Silo 2	DL	Brace buckling	0.04	0.09
	SD	Connection failure	0.22	0.09
	NC	Connection failure	0.22	0.09 0.36
Supporting tower	DL	Brace buckling	0.28	0
	SD	Brace plastic deformation	0.86	0.2
	NC	Brace fracture	0.88	0.21 2.36
BC1	DL	column plastic deformation	0.01	0.1
	SD	column plastic deformation	0.01	0.1
	NC	column plastic deformation	0.02	0.1 0.26
BC2	DL	column plastic deformation	0.24	0.09
	SD	column plastic deformation	1.27	0.2
	NC	column plastic deformation	1.56	0.26 1.85
Nitrogen argon	DL	column plastic deformation	0.5	0.09
	SD	column plastic deformation	1.56	0.2
	NC	column plastic deformation	2.12	0.26 2.5
Dust filters	DL	column plastic deformation	0.02	0.15
	SD	column plastic deformation	0.36	0.27
	NC	column plastic deformation	0.47	0.32 1.05
Main building	DL	Brace buckling	0.27	0.12
	SD	Brace plastic deformation	0.8	0.16
	NC	Brace fracture	1.07	0.23 2.29

5.4.3.2 Definition of the consequences

The consequences have been estimated, for each resilience state (see sect. 3.3.1.1), according to sect. 3.2.1.3. The recovery time depends upon different quantities, reported in the following for the sake of clarity:

- time for the inspection (T_{insp}): after a seismic event, T_{insp} represents the time required, by a team of experts, to detect the entity of the damage on a structure; it is assumed that the survey can be executed by the Plant's technical office in 1 day, when the null-to-limited damage is expected; otherwise, when the damage is not negligible an expert team opinion is required, prolonging the time for the inspection.
- time for the introduction of safety measures (T_{sm}): time interval required to introduce temporary measures, aimed at ensuring the safety of workers in the surrounding and avoiding the aggravation of the damaged condition of the facility;
- time for the design (T_D): time interval required to design the operations to restore the structural integrity according the actual standards (e.g the Eurocodes);
- time for the replacement (T_{rep}): time interval required to actualize the operations to restore the structural integrity, replacing or intervening on the damaged elements;
- time for demolition and reconstruction (T_R): time interval required to demolish and rebuild the whole facility to its previous undamaged condition, when the retrofit through local/global interventions is unfeasible from an economic perspective.

The residual functionality is a fixed value defined to ensure the safety of workers and avoid the aggravation of the damaged condition.

Table 3.3, reported in this chapter in table 5.3 to ease the comprehension, shows hoe to calculate the consequences, t_{rec} and $q(t)$, for each resilience state.

In the following, the detailed estimation of the consequences for each facility is reported. The consequences are briefly summarized in table 5.4.

Water treatment: mud container. The vulnerability analyses highlighted that the mud container supporting structure collapses in the elastic field due to a weak shear resistance in the columns.

For the RS_0 the recovery time is equal to 1 day and the residual functionality is full. For all the other resilience states, the residual functionality drops to zero. The recovery time is set as the sum of the inspection (7 days), the time for the demolition, design and reconstruction of the facility.

The demolition is assumed equal to 3 working days and foresees the removal

Table 5.3: Quantitative meaning of the Resilience states in terms of recovery time and residual functionality

Resilience state	Recovery time	residual functionality
RS₀	$T_{RS_0} = T_{insp}$	$q_0(t) = \text{full}$
RS₁	$T_{RS_1} = T_{insp} + T_d + T_{rep}$	$q_1(t) = \text{partial (50 - 80 \%)}$
RS₂	$T_{RS_2} = T_{insp} + T_d + T_{rep}$	$q_2(t) = \text{partial (20 - 50 \%)}$
RS_{3,a}	$T_{RS_{3,a}} = T_{insp} + T_d + T_{rep}$	$q_{3,a}(t) = \text{null (0 \%)}$
RS_{3,b}	$T_{RS_{3,b}} = T_{insp} + T_R$	$q_{3,b}(t) = \text{null (0 \%)}$

Table 5.4: Summary of the estimated consequences (days) for each resilience state: the number out of brackets represents the recovery time, the number between brackets represents the residual functionality.

Facility	T_R	RS₀	RS₁	RS₂	RS_{3,a}	RS_{3,b}
Mud Container	85	0 (100%)	-	-	-	85 (0)
Support 1	85	0 (100%)	-	-	-	85 (0)
Support 2	85	0 (100%)	-	-	-	85 (0)
Sand filters	50	0 (100%)	-	-	-	50 (0)
Dust filter	137	0 (100%)	9 (80%)	21 (50%)	-	137 (0)
Silo 1	102	0 (100%)	9 (50%)	-	-	102 (0)
Silo 2	112	0 (100%)	9 (80%)	-	-	112 (0)
BC 1	63	0 (100%)	9 (80%)	21 (50%)	-	63 (0)
Supporting tower	53	0 (100%)	9 (80%)	21 (50%)	-	53 (0)
Main building	492	0 (100%)	50 (80%)	72 (50%)	-	492 (0)
Nitrogen-argon	56	0 (100%)	14 (80%)	18 (50%)	-	56 (0)

of the supported mud container, the demolition and removal of the RC supporting structure and the demolition of the foundation structure.

In order to perform the design of the new structure, the main informations should be gathered by the technical office who will entrust the professional job to an external office. The external office will provide a design of the facility in compliance with the actual regulations, e.g. the EN 1998-1 (2005) or the Norme Tecniche per le Costruzioni (2008), along with all the shop drawings for its realization. For this purpose a total of 5 weeks are considered for the design process.

The reconstruction requires a minimum of 40 days. The following flowchart is expected:

- casting of foundations, columns and beams, with 7 days of interval between each stage;
- disposition of the empty mud-container and connection with the pipelines;
- restoration of the full functionality 28 days after the casting of the beams.

Water treatment: support 1 and support 2. The structural performance of the two supports of the cooling towers is the same of the mud container supporting structure. Considering also that the size and function within the production process are comparable, the recovery times and the residual functionality for the different resilience states are assumed equal to the mud container supports.

Water treatment: sand filters. Sand filters fails due to a lack of ground anchoring. Being supported on RC columns 60 cm high, it is assumed that once the seismic action overcomes the friction force, the vessel falls from the columns and therefore they need to be completely replaced. In this condition the residual functionality is null. For the estimation of the recovery time, the following operations to restore the full functionality have to be accounted for:

- removal of the pipelines;
- design and commitment of the of the sand filters in compliance with the actual standards.
- replacement of the old filters with new ones.
- connection of the new filters with pipelines and restoration of the functionality.

To this aim, considering also that a total of 20 filters are used, 50 days are expected to performed the above-mentioned scheduled operations.

Dust filter. Regarding the dust filter, different considerations have to be applied considering the two principal directions, X and Y. In fact, along the X direction, the structure resisting towards the horizontal action (bracing system) is different to the vertical load bearing systems. On the contrary, along the Y direction, the vertical and horizontal loads resisting systems are the same.

Along the X direction, for each resilience state the following operations are expected:

- RS₀:** one day of inspection time is expected, during which the functionality is dropped to the 80% to ensure the safety of workers.
- RS₁:** the braces in tension achieved the elastic limit and the compressed members buckled. The time for the inspection is assumed equal to 7 days. Therefore, considering also what provided by the FEMA P-58 Hamburger et al. (2012), 5 days for the design of the intervention, plus three additional days for the replacement of the buckled braces. A total recovery time of 15 days is expected. The residual functionality is dropped down to 80% to ensure the safety of workers during the inspection and during the replacement operations¹.
- RS₂:** the braces in tension develop plastic deformations, and sensitive residual deformations are expected. Thereby, to restore the functionality the plastically deformed braces have to be replaced and the undeformed condition of the whole facility must be restored. For the last requirement, a contrast structure, to which anchor hydraulic jackets, must be used to *force* the deformed structure in its original position. To this aim, considering the design and the execution of the operations, 3 weeks are expected. The residual functionality must be dropped at the 50% of the full load. In such a way, the vertical load, imposed by iron dusts and particles aspirated by the system during the melting process, is strongly lowered, increasing the structural safety towards the seismic actions.
- RS_{3,b}:** the collapse is reached when all the braces achieve the maximum plastic elongation. In this condition, the facility and the supported equipment must be completely reconstructed. The inspection time is set to 7 days. The time for the demolition is expected equal to 5 days. Design time is assumed equal to 90 days, since the steel structure and all the supported needs to be revised, to make them compliant with actual construction standards. Furthermore, a strict collaboration between the structural engineer and the equipment producer is required to optimize the design process. The reconstruction is expected to be completed in one month, being industrial facilities usually modular. When the RS_{3,b} is attained, the residual functionality is dropped to zero.

¹ It should be considered that the functionality is strongly reduced during the operations. In the other time intervals the functionality can be considered full.

Along the Y direction, the seismic performance is ruled by the plastic rotation of columns. The same strategies used along the X to restore the functionality are adopted.

Additional alloys: silo 1. As previously described, the silo 1 is strongly sensitive to $P - \Delta$ effects. The seismic performance is governed by the plastic rotations of the columns in correspondence of the base connection. In the following the recovery times and residual functionality for each resilience state is outlined.

RS₀: one day for the inspection is expected, during which the functionality is dropped to the 80% to ensure the safety of workers.

RS₁: the columns reach the elastic limit rotation at the base. The time for the inspection is assumed equal to 7 days. Therefore, considering also what provided by the FEMA P-58 Hamburger et al. (2012), 5 days for the design of the intervention, plus three additional day for the replacement of the buckled braces. A total recovery time of 15 days is expected. The residual functionality is dropped down to 50% to ensure the safety of workers during the inspection and during the replacement operations. It should be noted that the decision to drop the functionality to 50% is a consequence of the sensitivity to the $P - \Delta$ effects.

RS₂: the columns develops DL plastic rotations and sensitive residual deformations are expected. Thereby, to restore the functionality the plastically deformed braces have to be replaced and the and the undeformed condition of the whole facility must be restored. For the last requirement, a contrast structure, to which anchor hydraulic jackets, must be used to *force* the deformed structure in its original position. To this aim, considering the design and the execution of the operations, 3 weeks are expected. The residual functionality must be dropped at the 50% of the full load. In such a way, the vertical load, imposed by iron dusts and particles aspirated by the system during the melting process, is strongly lowered, increasing the structural safety towards the seismic actions.

RS_{3,b}: the collapse is reached when the sensitivity factor θ to the $P - \Delta$ effects exceeds 0.3. In this condition, the facility and the supported equipment must be completely reconstructed. The inspection time is set to 7 days. The time for the demolition is expected equal to 5 days. The design is assumed equal to 90 days, since the steel structure and all the supported needs to be revised, to make them compliant with actual construction standards. Furthermore, a strict collaboration between the structural engineer and the equipment producer is required to optimize the design process. The reconstruction is expected to be completed in one month, being industrial facilities usually modular. When the RS_{3,b} is attained, the residual functionality is dropped to zero.

Additional alloys: silo 2. Immediately after the buckling of the compressed braces, the column-to-brace connections fails. Hence, after the RS_0 , the silo 2 can attain the RS_1 and $RS_{3,b}$ resilience states. The following values are considered:

RS_0 : the inspection is performed in one day, while the design and replacement is expected to be executed respectively in 5 and 3 days. The functionality is full.

RS_1 : the recovery time is the same of RS_0 , but the residual functionality is lowered to the 80% of the full capacity.

$RS_{3,b}$: the inspection requires the evaluation of a technical expert; for that reason seven days are foreseen. The demolition is expected to be executed in 20 days, because of the interference with other facilities (silo 1 and the belt conveyors of the additional alloys unit). The design is expected to be concluded within 35 days and the reconstruction in 50 days, considering also the placement of the belt conveyors.

Additional alloys: supporting tower. The supporting tower is characterized by a modular steel facility. The following values are considered:

RS_0 : the inspection is performed in one day, while the design and replacement is expected to be executed respectively in 5 and 3 days. The functionality is full.

RS_1 : the functionality is dropped to the 80%, while the recovery time is set to 9 days: 1 day of inspection, 5 days for the design of the intervention and 3 days for the replacement of the buckled and plastically deformed braces.

RS_2 : the residual functionality is lowered to 50% and the recovery time is equal to 22 days: 7 days for the inspection, 3 days for the safety measures, 5 days for the design of the interventions and approximately one week for the replacement.

$RS_{3,b}$: the residual functionality is null. The recovery time is set to 61 days: 7 days of inspection, 5 days for the demolition, 2 weeks for the design of the new structure and 35 days for the construction.

Additional alloys: belt conveyors Regarding the belt conveyors, the recovery time and residual functionality for each resilience state is reported in the following:

RS_0 : the inspection is performed in one day. The functionality is full.

RS_1 : the functionality is dropped to the 80%, while the recovery time is set to 9 days: 1 day of inspection, 5 days for the design of the intervention and 3 days for the replacement of the buckled and plastically deformed braces.

- RS₂** : the residual functionality is lowered to 50% and the recovery time is equal to 22 days: 7 days for the inspection, 3 days for the safety measures, 5 days for the design of the interventions and approximately one week for the replacement.
- RS_{3,b}**: the residual functionality is null. The recovery time is set to 53 days: 7 days of inspection, 5 days for the demolition, 3 weeks for the design of the new structure and 20 days for the construction (including the delivery of the assembled equipments).

Main Building. Regarding the main building steel warehouse, the recovery time and residual functionality for each resilience state is reported in the following:

- RS₀** : the inspection is performed in one day; the functionality is full.
- RS₁** : the functionality is dropped to the 80%, while the recovery time is set to 41 days: 7 days for the inspections, 15 days for the design of the interventions and 25 days for the replacement of columns or the buckled and plastically deformed braces.
- RS₂** : the residual functionality is lowered to 50% and the recovery time is equal to 50 days: 7 days for the inspection, 2 days for the safety measures, 15 days for the design of the interventions and approximately 25 days for the replacement.
- RS_{3,b}**: the residual functionality is null. The recovery time is set to 492 days: 7 days of inspection, 60 days for the demolition, 60 days for the design of the new structure and one year for the construction (including the delivery and assembly of equipments).

Nitrogen - Argon vessels

- RS₀** : the inspection is performed in one day; the functionality is full.
- RS₁** : the functionality is dropped to the 80%, while the recovery time is set to 11 days: 1 days for the inspections, 3 days for the safety measures and 8 days for the replacement of damaged columns.
- RS₂** : the residual functionality is lowered to 50% and the recovery time is equal to 18 days: 7 days for the inspection, 3 days for the safety measures and 8 days for the replacement of damaged columns.
- RS_{3,b}**: the residual functionality is null. The recovery time is set to 56 days: 7 days of inspection, 7 days for the demolition, 21 days for the design of the new structure and for the construction (including the delivery of the assembled equipments).

5.4.3.3 Estimation of the resilience and facilities prioritization

The detailed resilience assessment is performed in two steps: the first aims at drawing the resilience curve, its confidence bounds and an estimation of the EAL curve, basing the analyses on the PBEE framework; the second is a sensitivity-based assessment aimed at identify the weight of each facility on the resilience of the whole plant.

In order to perform the PBEE Resilience assessment, a Monte Carlo analysis has been set up to account for the variability of the recovery time. In particular, a sample size of 10^5 elements and a Sobol Sampling strategy have been adopted.

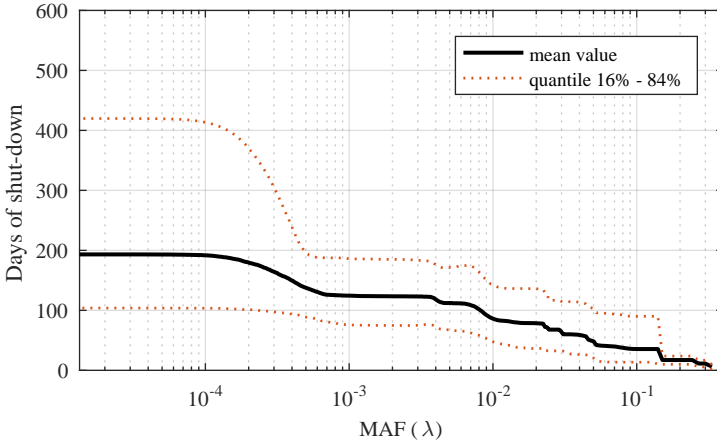
Step 1. The results of the detailed vulnerability assessment and the precise estimation of the consequences, gathered as explained in sect. 3.3 and sect. 3.3.5, yields the resilience curve shown in figure 5.17. It should be noted that, globally, the plant evidence an high vulnerability. In fact, the expected days of shut-down result substantial at MAF higher than 10^{-2} , for which, according to actual regulations, no seismic damage should be expected for new buildings. The mean value of the EAL is equal to 8.46 days/year, while the C.O.V. is equal to 0.36.

Observing figure 5.18 it can be observed that at MAF lower than that 10^{-2} 1/years, the cumulated EAL (EAL_{cum}), computed as in eq. (5.1), remains almost constant. This fact is a direct consequence of the high vulnerability of the considered facilities, achieving the collapse or near collapse, and therefore the highest loss of resilience (of stand-alone facility) at rather high MAF of the seismic action.

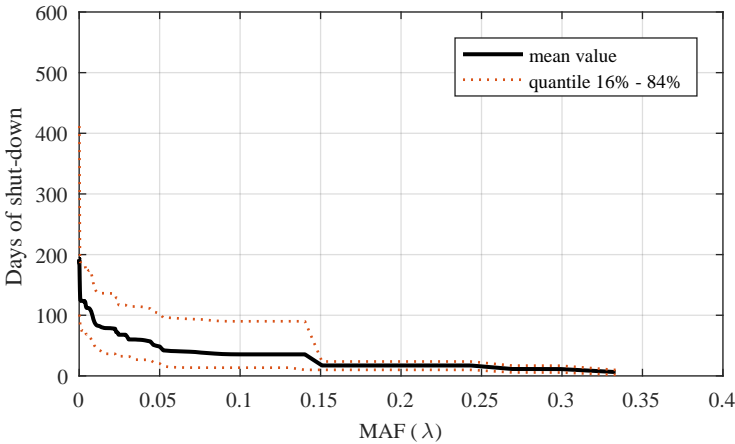
$$EAL_{cum} = - \int_{\lambda_{max}}^{\lambda} LR(\lambda) d\lambda \quad (5.1)$$

Being the recovery times considered as lognormally distributed, the EAL can be suitably fitted with a lognormal distribution, as shown in figure 5.19.

Step 2. The second step, focuses on the definition of facilities prioritization, through a linearized method, based on the standard regression coefficient (SRC) method. The results of the sensitivity analysis are reported in figure 5.20, irrecalling table 5.5 for what concerns the ID number of each facility. It can be observed that the only few facilities that influences the plant EAL are the dust filter and the additional alloys units. As it can be observed, for the



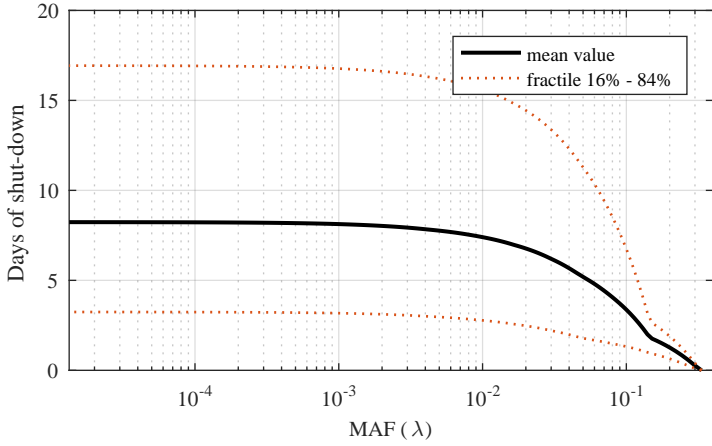
(a) lognormal plot



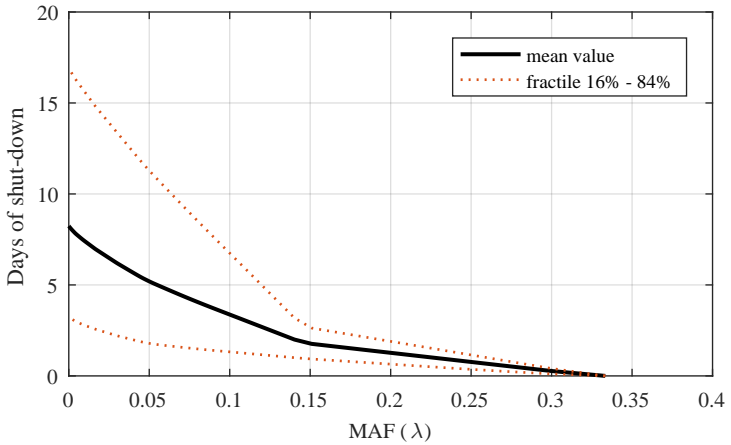
(b) standard plot

Figure 5.17: Resilience curve: a) lognormal plot, B) standard plot.

additional alloys unit, the most critical facilities are represented by the BC1 and the silo 1, that are also the most vulnerable ones.



(a) Lognormal plot



(b) Standard plot

Figure 5.18: Cumulated EAL: a) lognormal plot, B) standard plot.

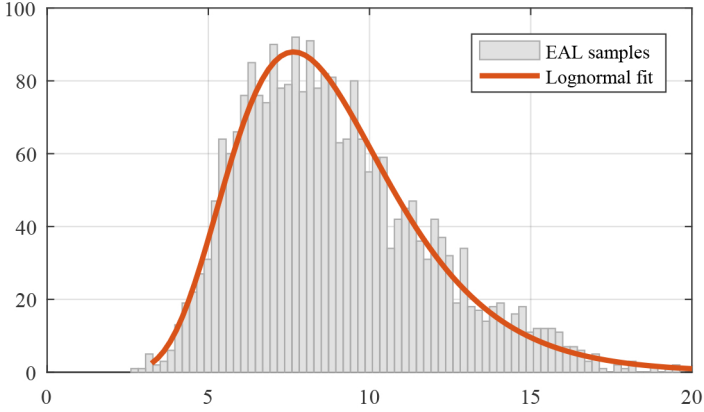


Figure 5.19: Distribution of the EAL and its lognormal fit.

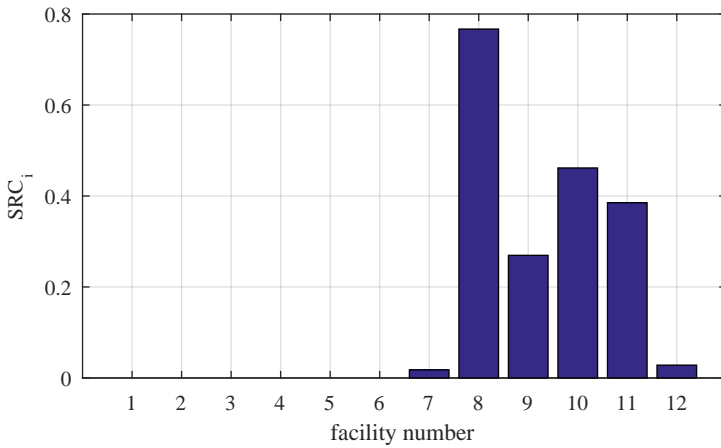


Figure 5.20: SRC parameters for the steel mill (for the numbering see table 5.5).

Table 5.5: ID of the facilities belonging to the steel mill

1	2	3	4	5	6
WT - mud container	WT - sand filters	WT - support 1	WT - support 2	Dust filter	Nitrogen Argon vessels
7	8	9	10	11	12
Main building	AA - BC1	AA - BC2	AA - silo 1	AA - silo 2	AA - supp. tower

WT = Water Treatment unit

AA = Additional Alloys unit

5.4.4 Application of the multilevel procedure

According to what developed in 4, the application of the multilevel procedure on the steel mill is carried out through the following steps:

- identification of the break-even point (t_{BEP}), representative of the maximum days of shut-down the Plant can withstand without making economic losses;
- planning of the walk down, aimed at the attainment of a minimum knowledge level to perform simplified structural analyses;
- vulnerability assessment;
- application of the three levels procedure and identification of the critical structures.

It should be noted that the procedure can requires more than three steps. In fact, it is possible that the facilities that results critical at the first level, could appear to be less critical at the second level. Therefore, a further application of the second level could be indispensable.

In the following, the above-mentioned aspects are described in detail.

5.4.4.1 Break even point

After a collaboration with the technical office of the steel mill, it was decided to fix the break even point to 5 days. For the sake of clarity, it is remembered that the t_{BEP} considers only the shut-down of the plant as a consequence of unforeseen events. This value is usually selected by the owner and depends upon several conditions, such as the productivity of the plant, the additional fines when the deadlines delivery are not respected, and so on.

5.4.4.2 Walk-down

The walk-down aims at providing a limited knowledge of the facilities, to make possible the execution of vulnerability analyses by means of simplified numerical models. The walk-down should validate the information provided by technical documentation (reports and drawings) when available; otherwise it should focus on the definition of the structural configuration and most important details.

The aspects of major concern when the walk-down is carried out on the steel mill are summarized in the following.

Water treatment. Three RC structures (Support of the mud container and two supports for the cooling towers) and steel vessels (sand filters).

- *RC concrete elements:* the columns should be surveyed, in order to verify that at least the number of longitudinal rebars and the average spacing of stirrups fits with the technical drawings.
- *sand filters:* survey of the base connection, aimed at verifying the presence of anchor bolts, and the columns to vessel connection, to verify the presence of thicker plates to avoid punching mechanisms.

Additional alloys. This unit is composed of steel structures. The focus must be posed on.

- *Silo 1:* check the size of the columns, the braces and the thickness of column-to-brace plate connection.
- *Silo 2:* check the size of braces and the configuration of the bolted connections.
- *Support tower:* check the size of braces and the configuration of the bolted connections.
- *Belt conveyors:* check the supporting columns and the connections with the silos.

Dust filter. The surveys must focus on the size of the columns, the braces and the thickness of column-to-brace plate connection.

Main building. The main building is a steel industrial warehouse. Particular attention has to be placed on the following aspects:

- the consistency of the section of columns and braces
- the consistency of the columns to ground and column to brace connections;
- the particular configuration of the columns to crane beam joints. It is necessary to verify that the crane beams do not stiffen the structure and therefore can be involved in the seismic collapse mechanism.

Nitrogen-argon. The surveys have to focus on the base plate connection and the presence of suitable anchor bolts, the size of the supporting columns and the presence of a thicker plate at the column-to-vessel connection, required to reduce possible punching problems.

5.4.4.3 Application of the procedure

For the application of the procedure, different types of structural analyses, depending on the applied level, are necessary to define the vulnerability assessment. For this reason, the different analyses, that can be performed at the first and second level of the procedure, are reported in the following.

Vulnerability assessment. The results of the simplified analyses are reported in appendix A. In particular, the following assumptions are given for each level of the multilevel procedure:

- For the *level 1* simplified models are applied. The limited knowledge level, gained through the analysis of the technical documentation and the walk-down on the plant, imposes the adoption of the characteristic values of the mechanical parameters of structural materials, reported in 5.1. For what concern the modelling approach, being all the facilities rather simple, the same models of the detailed assessment are used, except for the vessels. For such facilities a simplified model that account only for the post-elastic behaviour of the supporting columns is adopted.
- For the *level 2* the same models used for the detailed vulnerability assessment are used, reported in sect. 5.4.3.1. In this case, supposing the achievement of an exhaustive knowledge level, the mean values of the mechanical parameters are used (see table 5.1).

The results of the vulnerability assessment performed for the level 1, using a simplified approach, are summarized in table 5.6. The results are particularly heterogeneous. Some facilities highlighted an high seismic vulnerability, others show a negligible to null vulnerability. The results for the level 2, i.e. adopting a detailed vulnerability assessment approach, are reported in table 5.2.

For the sake of clarity, the procedure is composed of three levels:

- the first one focuses on simplified modelling, which results are reported in 5.6 of the seismic behaviour and of the consequences; the resilience indicators (RI) provide a preliminary hierarchy of the facilities, useful for the prioritization of the facilities to establish which one should be assessed in the next iteration;
- the second level refines the informations obtained from the first level, improving the knowledge of the structural behaviour and/or the consequences; the RI are updated and a refined hierarchy among facilities is obtained; for what concern the vulnerability analyses, the results of the detailed assessment, reported in 5.2, are used.

- the third level focuses on highly refined analyses of the critical details and on the identification of the effective benefits that retrofit interventions might provide.

The possible solutions to apply the second and third levels are reported in table 5.7.

For the purpose of the analysis of the steel mill, four steps are applied to achieve the t_{BEP} . In particular:

- the first step focuses on the application of the first level;
- the second and third steps focus on the application of the second level;
- the fourth step focuses on the application of the third level.

5.4.4.4 Application of the first level

At the first step, the first level is applied. In order to apply the first level, the following steps are compulsory:

- development of the simplified structural models on the basis of the information obtained from the *walk-down* of the facilities;
- estimation of the consequences;
- definition of the plant resilience;
- definition of the Resilience Indicators.

The analyses are performed with numerical models as much as possible simplified. Since the informations about the facilities are scarce and derived mainly studying the technical drawings, possibly confirmed by means of fast in-field surveys (walk-down), the characteristics values of the mechanical parameters of structural materials, adopted by the Regulations adopted during the design of the facilities, are used.

The simplified estimation of the consequences is performed by means of the consequences matrix. The following quantities are required:

- estimation of the total reconstruction time;
- estimation of the recovery time for each resilience state, as a percent of the complete reconstruction time;
- estimation of the residual functionality.

Table 5.8 summarizes the estimated consequences. It should be noted that reconstruction time is close to the values obtained with the detailed assessment.

From the first analysis it results that the estimated EAL is equal to 10.65. The water treatment (unit 1), the dust filter (unit 4) and the additional alloys unit (unit 5) are the critical ones, as shown in figure 5.21. From the figure it can be easily observed that almost all the resilience curve is *dominated* by the additional alloys unit (*unit 5*) and only partially by the water treatment unit (*unit 1*) at low MAFs. The high vulnerability of some facilities yields to high loss of resilience values already at rather high MAF.

In order to achieve the t_{BEP} , the uniform and optimized scaling procedures are shown, in order to provide a practical comparison of the two methods. Figure 5.21 shows a comparison between three conditions:

1. the **available condition**, representative of the current resilience condition;
2. the resilience condition that should be achieved to fit the t_{BEP} requirements, adopting a **uniform scaling** on the facilities belonging to the critical units;
3. the resilience condition that should be achieved to fit the t_{BEP} requirements, adopting an **optimized scaling**.

For the sake of clarity, it should be remembered that, for the condition 2 and 3, the non critical units are scaled up, until they reach the upper shelf bound (maximum EAL) or they intersect the resilience curve.

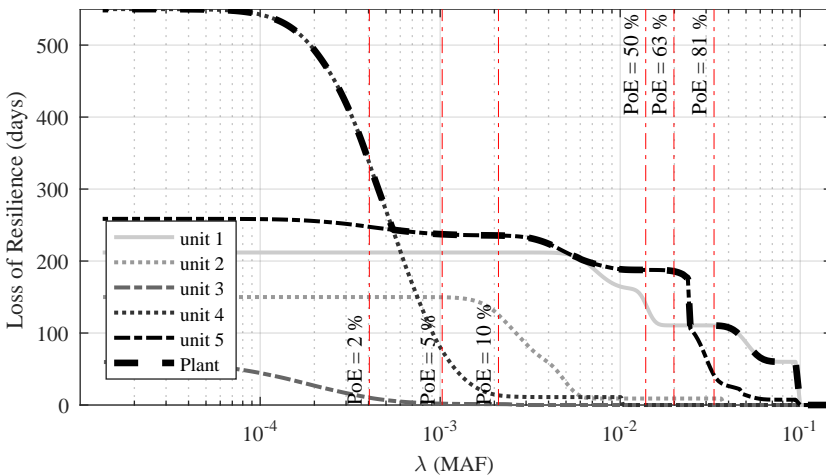


Figure 5.21: Resilience curve of the five units and the whole Plant.

Resilience indicators. Table 5.9 reports the resilience indicators. The lower the resilience indexes the higher is the attention the facility requires.

The plant configuration of the Steel Mill represents a clear example in which the uniform scaling fails. In fact, adopting the uniform scaling is not possible to distinguish different levels of importance within the same unit, and all the critical units have an RI equal to 0.28.

The optimized scaling solves this problem providing a wider range of RI intervals. Within the same critical unit, a distinction among facilities, depending on their vulnerability and associated consequences, can be now provided. In fact, still looking at table 5.9, the following aspects can be observed:

- for the unit 1, support 1 and 2 are less critical and, therefore, are associated with an higher RI than the mud container and, in particular, the sand filters;
- the dust filter influences the resilience curve at very low MAF, therefore it has RI equal to the unit, being it's influence negligible;
- for the unit 5, the supporting tower results the less vulnerable structure; as a consequence, its resilience index is higher than the other structures.

Looking at the shape of the three resilience curves (available, uniform scaling and optimized), briefly reported in figure 5.23, the uniform scaling solution provides a constant scaling over the entire MAF range. On the contrary, the optimized scaling, provides a greater reduction in correspondence of higher MAFs, while at lower MAFs (lower than 10^{-3}) the scaled curve is close to the original one. That means that the facilities characterized by an high vulnerability index and great consequences need a strong modification, in terms of post-earthquake interventions (reduction of consequences in the preparedness situation) or in the terms of vulnerability reduction (pre-earthquake mitigation intervention) to achieve an EAL equal, or lower, to t_{BEP} (resilience-neutral or resilience-seeking condition). Clearly, achieve the same result scaling only at lower MAFs, means that the improvement of the interventions techniques and the reduction of the application time are so high that they results to be unfeasible. In fact, looking at figure 5.22, the cumulated EAL has a steep variation at higher MAF, remaining almost constant at the lower frequencies.

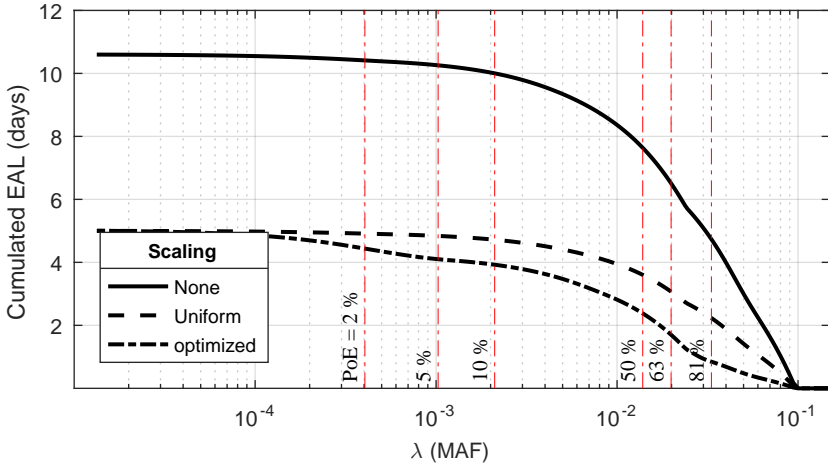


Figure 5.22: Comparison of the cumulated EAL obtained from the three resilience curves.

Table 5.6: Summary of the results of the simplified vulnerability assessment.

	Facility Mechanism	$S_a(T_1)$	β	I_v	
Mud container	DL	shear failure	0.17	0.1	
	SD	shear failure (elastic field)	0.17	0.1	
	NC	shear failure (elastic field)	0.17	0.1	
Support 1	NC	shear failure (elastic field)	0.17	0.1	0.19
	DL	shear failure (elastic field)	0.44	0.1	
	SD	shear failure (elastic field)	0.44	0.1	
	NC	shear failure (elastic field)	0.44	0.1	0.56
Support 2	DL	shear failure (elastic field)	0.23	0	
	SD	shear failure (elastic field)	0.23	0	
	NC	shear failure (elastic field)	0.23	0	0.27
Sand filters	DL	Sliding failure	0.08	0	
	SD	Sliding failure	0.08	0	
	NC	Sliding failure	0.08	0	0.1
Silo 1	DL	Brace buckling	0.01	0.1	
	SD	Connection failure	0.02	0.1	
	NC	Connection failure	0.03	0.1	0.27
Silo 2	DL	Brace buckling	0.03	0	
	SD	Connection failure	0.2	0	
	NC	Connection failure	0.2	0	0.31
Supporting tower	DL	Brace buckling	0.28	0	
	SD	Brace plastic deformation	0.5	0.2	
	NC	Brace fracture	0.5	0.21	2.06
BC1	DL	column plastic deformation	0.01	0	
	SD	column plastic deformation	0.01	0.01	
	NC	column plastic deformation	0.01	0.07	0.02
BC2	DL	column plastic deformation	0.06	0.01	
	SD	column plastic deformation	0.36	0.07	
	NC	column plastic deformation	0.49	0.14	0.79
Nitrogen argon	DL	column plastic deformation	0.06	0.01	
	SD	column plastic deformation	0.36	0.07	
	NC	column plastic deformation	0.49	0.14	0.79
Dust filters	DL	column plastic deformation	0.02	0.05	
	SD	column plastic deformation	0.33	0.33	
	NC	column plastic deformation	0.43	0.34	0.98
Main building	DL	Brace buckling	0.27	0	
	SD	Brace plastic deformation	0.7	0.16	
	NC	Brace fracture	0.95	0.21	2.02

Table 5.7: Possible strategies for the application of level 2 and 3 of the multilevel procedure.

Facility	Level 2	Level 3
Mud container	Increase the knowledge level	Seismic retrofit, increasing the shear resistance of the columns
Sand Filters	Increase the knowledge level	Seismic upgrading, increasing the shear resistance of the columns
Support 1	Increase the knowledge level	Seismic upgrading, increasing the shear resistance of the columns
Support 2	Increase the knowledge level	Seismic upgrading, increasing the shear resistance of the columns
Dust Filter	Increase the knowledge level	Seismic upgrading
Nitrogen-Argon	Increase the knowledge level/detailed modelling of the facility	Detailed modelling of the collapse mechanism (e.g. crack propagation)
Main building	Increase the knowledge level/detailed modelling of the facility	
BC1	Increase the knowledge level/detailed modelling of the facility and the supporting structure (silo 1 and 2)	Seismic retrofit
BC2	Increase the knowledge level/detailed modelling of the facility and the supporting structure (silo 2 and supporting tower)	Seismic retrofit
Silo 1	Reduce weight	Seismic Retrofit
Silo 2	Reduce weight	Detailed Assessment of the connection/Seismic retrofit
Supporting tower	Increase the knowledge level	Detailed assessment of the supported weight/Seismic retrofit

Table 5.8: Consequences estimated in a simplified way a fraction of the total reconstruction time; the values reported between brackets refers to the residual functionality.

Facility	T_R	Consequence Matrix			
		RS_0	RS_1	RS_2	RS_3
Silo 1	120	0.0 (100%)	0.1 (50%)	1.0 (25%)	1.0 (0.0%)
Silo 2	120	0.0 (100%)	0.1 (80%)	1.0 (50%)	1.0 (0.0%)
Support tower	60	0.0 (100%)	0.1 (80%)	0.5 (50%)	1.0 (0.0%)
Belt conveyor	50	0.0 (100%)	0.2 (80%)	0.2 (50%)	1.0 (0.0%)
Mud container	90	0.0 (100%)	0.2 (80%)	0.2 (50%)	1.0 (0.0%)
Support 1	90	0.0 (100%)	1.0 (0.0%)	1.0 (0.0%)	1.0 (0.0%)
Support 2	90	0.0 (100%)	1.0 (0.0%)	1.0 (0.0%)	1.0 (0.0%)
Sand Filters	60	0.0 (100%)	1.0 (0.0%)	1.0 (0.0%)	1.0 (0.0%)
Dust filter	150	0.0 (100%)	0.1 (80%)	0.25 (50%)	1.0 (0.0%)
Main Building	365	0.0 (100%)	0.1 (80%)	0.25 (50%)	1.0 (0.0%)
Nitrogen-Argon	60	0.0 (100%)	0.1 (80%)	0.5 (50%)	1.0 (0.0%)

Table 5.9: Definition of the resilience indicators (RI) adopting the optimized scaling procedure. The colored filling refers to the importance level: the lower is the resilience indicator the more the structure is critical. The number between brackets represents the ranking based on the R.I.

Facility n°	Optimized scaling RI
Silo 1	0.15
Sand Filters	0.16
Silo 2	0.18
BC1	0.19
Mud container	0.45
BC2	0.46
Dust filter	1
Support 1	1
Nitrogen-argon	1.22
Main building	1.57
Support 2	1.0
Supporting tower	1.0

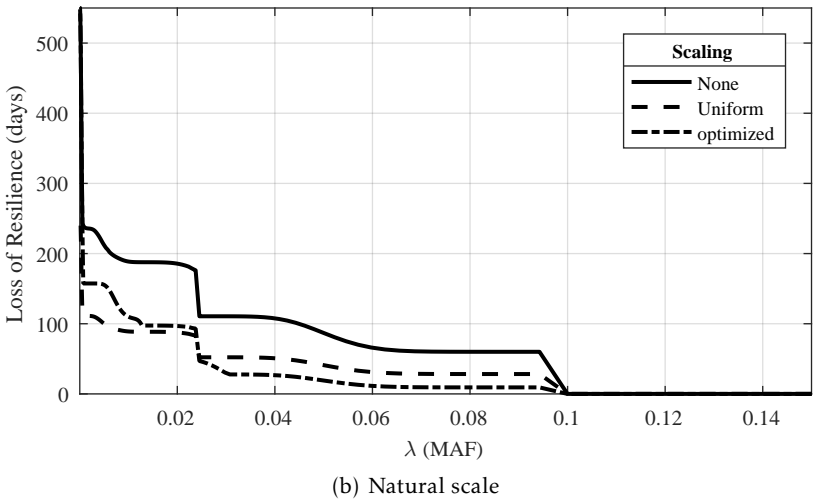
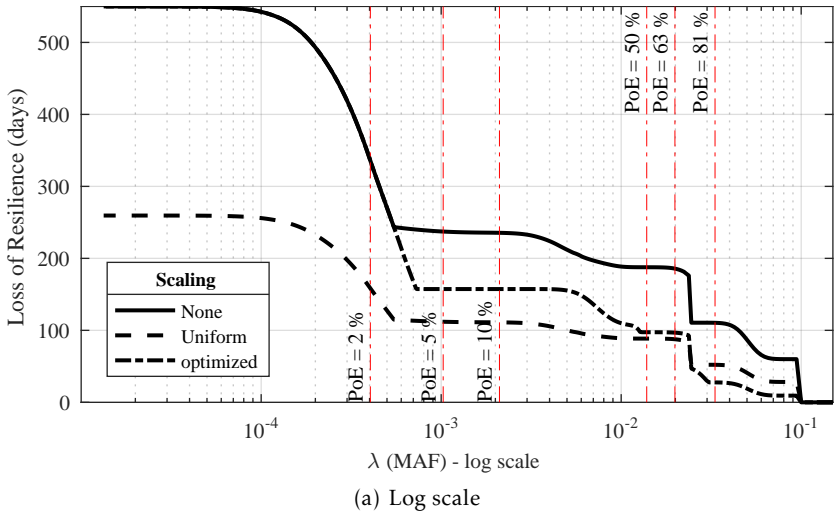


Figure 5.23: Resilience curves obtained from the three scaling conditions.

5.4.4.5 Application of the second level.

Since from the level 1 to the first application of the second level the RI are changed, the second level is applied through two iterations.

First iteration. For the application of the second level at the first iteration, all the critical facilities with a RI lower than 0.2 have been considered as critical. According to what is provided in table 5.7, the knowledge level of the *critical* facilities is improved by means of the following in deep assessments:

- the silo 1 requires more detailed investigation to achieve as high as possible knowledge level. Since it has been designed to withstand only the vertical loads, the silo 1 can be empty, completely or partially, modifying the filling procedures of the silo 2: instead of filling all the silo 2 with limestone, the two external container can be partially filled with coke, without modifying the total seismic mass. By this means the vulnerability of the silo 1 is reduced.
- For the silo 1, only detailed investigations can be performed to improve the knowledge of the facility.
- for what concern the BC1, the modification of the structural behaviour, changing the effective seismic mass of the silo 1, can be enough to improve the performance of the BC1 itself.
- For what concern the sand filters, further investigations can be performed in order to check if the anchor bolts are effectively present. Furthermore, the improvement of the structural capacity considering a suitable ground anchoring can be applied.
- The consequences associated with the four critical facilities are precisely assessed.

The proposed interventions to the silo 1, changing the masses, should lead to a reduction of the vulnerability, enhancing the resilience associated with the facility and therefore, being the system network a set of in series structures, of the Plant itself. At the end of the first iteration, the EAL is reduced from 10.65 to 9.55, and the prioritization based on the RI is reported in table 5.10. Being the $t_{BEP} = 5 \text{ days}$ not achieved, a further step is required.

Second iteration. At the end of the first iteration on the second level, the mud container, the silo 1 and the sand filters are associated with the lower RI. Therefore, at the third iteration, still focusing on the second level, since the silo 1 has been already assessed, only the mud container, the sand filters and support 2 are deeply assessed. The following operations can be performed to refine the results:

Table 5.10: Resilience indicators after the second step.

Facility	RI
Mud container	0.17
Sand Filters	0.17
Silo 1	0.29
Support 2	0.33
Main building	0.97
Support 1	1
BC1	1
BC2	1
Silos S11-S17	1
Supporting tower	1
Azoto Argon Ossigeno	1.03
Dust Filters	1.14

- improve the knowledge level of the structures by means of detailed in situ surveys, foreseeing also tests for the mechanical characterization of the structural materials;
- sharp the recovery times, by means of detailed assessment.

At this iteration, the resilience is lowered at 8.04 days. Since also in this case, the $t_{BEP}5\text{ days}$ is not achieved, a fourth iteration is required.

5.4.4.6 Application of the third level

At the fourth iteration, the third level is applied, in order to assess if localized retrofit solutions can be enough to achieve the break-even point.

Considering the first three facilities with the lowest RI (see table 5.11), the mud container (RI = 0.15), silo 1 (RI = 0.29) and the sand filters (RI = 0.56), need further assessment.

For what concern the sand filters it is supposed to reduce the vulnerability, providing a suitable ground anchoring. For the silo 1 and the mud container, it is supposed to improve the seismic performance by only increasing the mean value of the vulnerability index, assessed for the NC limit states, to

Table 5.11: Resilience indicators after the third iteration.

Facility	RI
Mud container	0.15
Existing Silos	0.29
Sand Filters	0.56
Warehouse	0.91
BC1	1.03
BC2	1.03
Silos S11-S17	1.03
Torretta	1.03
Nitrogen-argon	1.03
Support 2	1.06
Support 1	1.06
Dust Filters	1.25

the unit. The dispersion β remains fixed. For instance a solution for the mud container should be the introduction of steel or RC jacketing of the columns to increase the shear resistance.

The fragility curves of the RC mud container are updated as in the following:

- the shear failure is considered solved, increasing the shear capacity;
- the fragility curves are calculated considering the attainment of the limit states associated with the plastic/ductile flexural behaviour of the columns;
- the mean of the fragility curve at the NC limit states is translated in order to achieve a vulnerability index equal to the unit. The DL and SD fragility curves are translated of the same quantity.

By this means it is obtained an EAL equal to 3.63, evidencing the necessity to plan retrofit intervention to increase the plant resilience, reducing the mean annual shut-down days (EAL).

5.5 Concluding considerations

In this chapter the seismic resilience of the steel mill case study is assessed, in order to establish the validity of the proposed multilevel procedure. To this aim, both the detailed resilience assessment and the multilevel procedures are applied on the case study to assess the resilience.

The Steel Mill is composed of several one-storey facilities (R.C. and steel braced frames or MRF) that support heavy equipments, such as storage silos or cooling towers. All the facilities are connected in series; therefore, when one fails or reduces its functionality, the whole Plant is affected.

For what concern the detailed assessment of the seismic resilience, refined model have been realized. In such a way it was possible to precisely detail the structural performance towards the seismic action. Seen the particularly simple configuration of the facilities, fragility curves has been derived from the results of non-linear static analyses. For what concern the consequences, specific analyses have been performed to obtain a detailed estimation at each resilience state.

The detailed assessment evidenced that the facilities belonging to the additional alloys and the dust filter unit are the most influencing on the Plant's Resilience. This is mainly due to the high seismic vulnerability of such facilities and the associated consequences.

For what concern the multilevel procedure, seen the particular simplicity of the structural configuration of facilities, the simplified numerical models of supporting structures (RC and steel) are technically equivalent to the detailed ones. Regarding the vessels, some simplifications have been introduced, focusing only on the post-elastic behaviour of the supporting columns.

The multilevel procedure has been applied in four steps:

- the first step focuses on the on a simplified analysis of the plant;
- the second and third steps provide a deep insight of the critical units, for what concern the structural performance, considering an increased knowledge level obtained through specific in-field surveys, and the consequences estimation;
- the fourth step provides an evaluation of the effectiveness of the application of retrofit strategies.

The increase of the knowledge of the structural performance and the consequences is evidenced in table 5.12 and in figure 5.24: the EAL strongly reduces from one step to the next and the resilience curve tends to fit the

one obtained with the detailed assessment. At the third step, at which an exhaustive knowledge level associated with all the critical units is achieved, the resilience curve and the EAL are in very good agreement with the detailed procedure. The resilience curve, obtained at the third step, in the MAF range $[10^{-3}, 10^{-2}] 1/\text{years}$ results lower than the resilience curve obtained from the the detailed assessment are due to the fact that the silo 1, at the second step, is partially empty to reduce its seismic vulnerability.

The fourth level, introducing some hypotheses for the seismic upgrading of facilities, yields an EAL and, therefore, a resilience curve, significantly lower than the detailed procedure, as shown respectively in table 5.12 and figure 5.24. In particular, it is evidenced that the t_{BEP} can be achieved only by improving the seismic capacity of the critical facilities.

Table 5.12: EAL, expressed in days of shut down, obtained from the application of the detailed approach and the multilevel procedure.

Procedure	1 st step	2 nd step	3 rd step	4 th step
Detailed procedure	8.46*			
Multilevel procedure	10.65	9.55	8.04	3.63

*C.O.V = 0.36

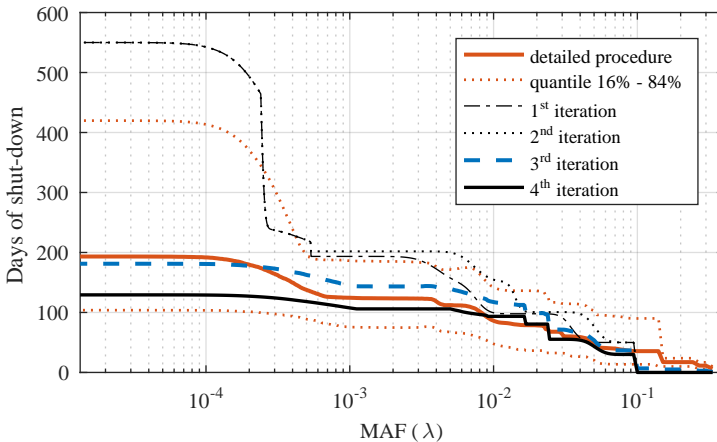


Figure 5.24: Comparison of the resilience curves obtained applying the detailed and the multilevel procedures.

The multilevel procedure achieves satisfactory results in terms of Plant's Resilience with very few steps, allowing an optimization of the in-field surveys and complex assessment, aimed at characterizing the structural performance and the consequences as well.

The application of the first and second level provided a prioritization of facilities, based on the Resilience Indicators; it is in good agreement with the one extracted from the sensitivity analysis performed on the detailed approach, evidencing the effectiveness of the optimization procedure. The multilevel procedure does not catch the dust filter as critical because such unit influences the loss of resilience at very low MAF.

Chapter 6

The chemical Plant

6.1 Introduction

The chapter describes, first, the plant configuration and the facilities of the chemical plant case study, supplying all the information required for the application of the resilience assessment procedure, including the definition of the seismic hazard.

The plant is then analysed from the perspective of the production process so to provide a proper scheme of the system network, necessary for the following resilience assessment. On the basis of the plant configuration and the production work-flow, the restoration schedule has been defined.

Finally both the detailed and the multilevel procedures are applied separately for the assessment of the plant resilience. The application of such procedures to the chemical plant case study highlighted advantages and disadvantages, properly described at the end of the chapter.

6.2 Description of the chemical plant

6.2.1 *The Plant configuration*

The chemical plant is composed of 21 vessels, supported by a three storey steel structure. Some of them get through two or all the storeys and are supported by suitable chevron beams, other are simply supported by columns leaning directly on the floor. Those vessels are connected through a pipe network in series and contains process units for the chemical transformations. The production process is continuous all the day. Interruptions of the pro-

duction are expected only for planned maintenance of the equipments.

6.2.2 Description of the facilities

The main structure and the vessels are described in detail in the following. All the facilities are realized with S235 steel.

The main structure. The main structure is a three storey steel facility (2 storeys of height 6 m and the last of height 5 m), with braces systems differently arranged to withstand the horizontal loads. The structural configuration is compact, covering approximately 20x18 meters in plan, and is organized in three bays along the X direction and two bays along the Y one. The columns are HEM 280, to withstand heavy concentrated loads and allow enough freedom for the modification of the applied loads. Along the X direction all the three main frames are braced, while along the Y direction only the external frames are braced (see figures from 6.4 to 6.10). In floor planes bracing systems are provided at the first and second storeys, adopting single L70x7, but not along all the two principal direction, as clearly shown in figures in 6.2 and 6.3.

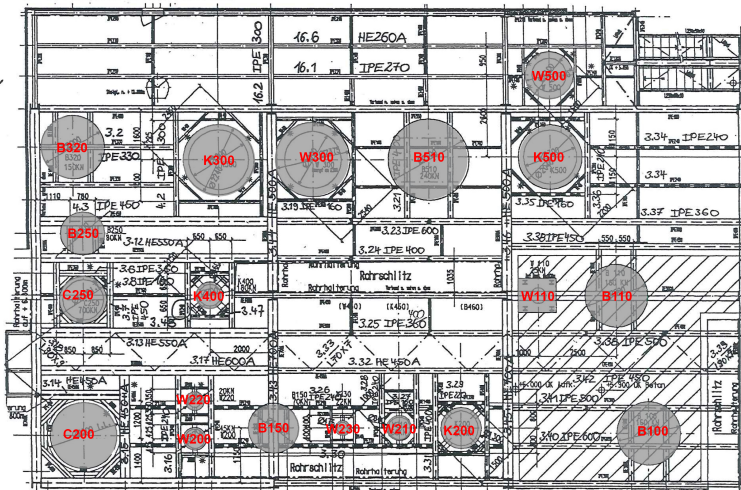


Figure 6.1: First storey (+6.00 m).

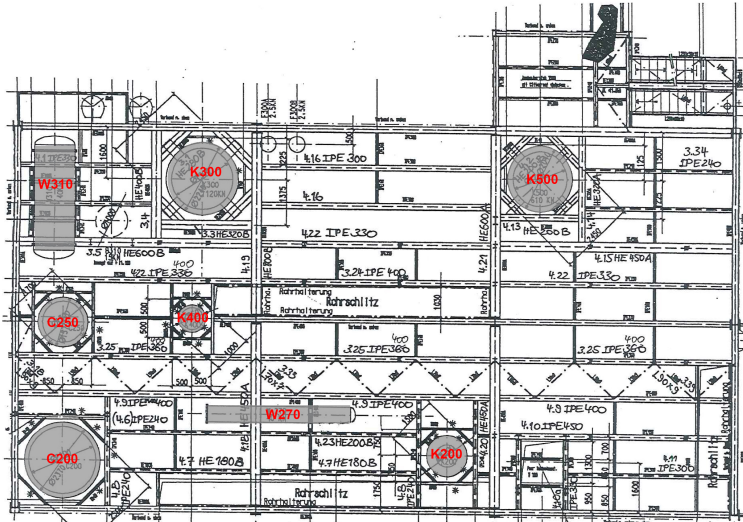


Figure 6.2: Second storey (+12.00 m).

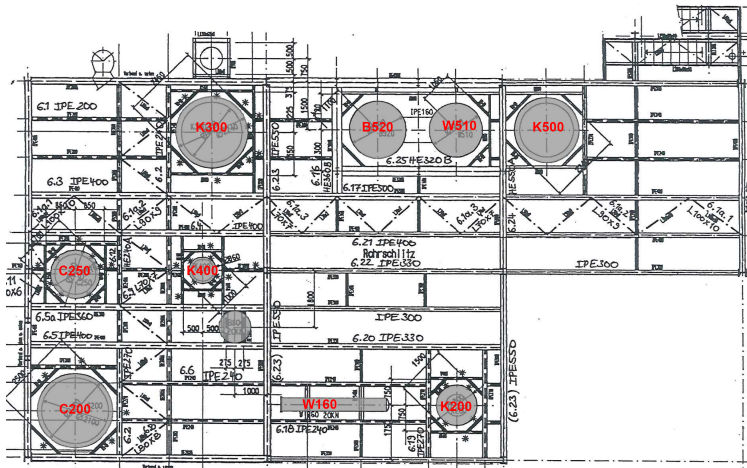


Figure 6.3: Third storey (+17.00 m).

According with the technical reports, the structure has been designed to withstand the vertical loads and wind action, evaluated as a uniform horizontal pressure equal to 1 kNm^{-1} . During the design stage, the vertical load has been considered equal to 5 kNm^{-2} in addition to the load transferred by vessels and process towers. This was a typical approach adopted for the design of industrial facilities, as it provides a great flexibility to the owner to

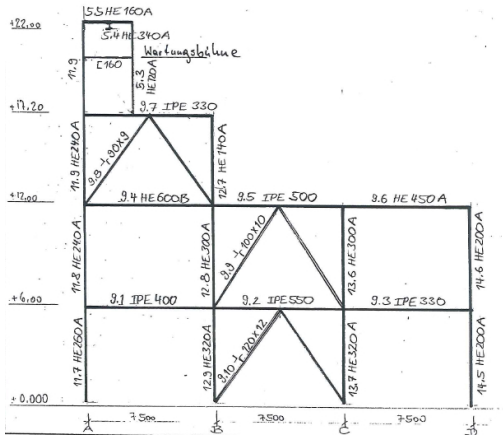


Figure 6.6: Side view: alignment 3.

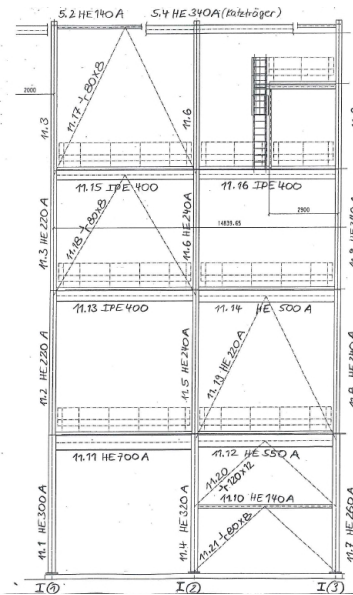


Figure 6.7: Side view: alignment A.

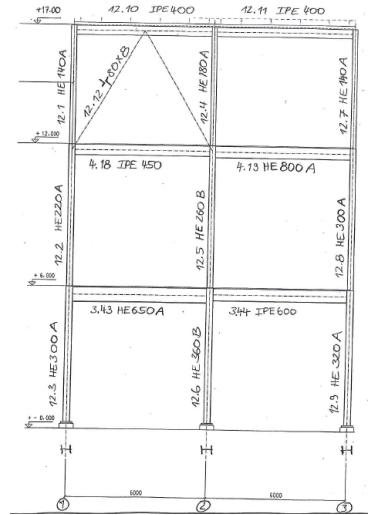


Figure 6.8: Side view: alignment B.

The Supported vessels. A total of 21 process towers and vessels are supported at the three storeys. Each vessel has a cylindrical shape and is characterized by different anchoring configurations: some are supported by simple steel columns, others are directly anchored to the main beams by means of

ical infrastructure.

It is evident that for a *critical infrastructure* site, a suitable characterization of the seismic hazard is necessary. One of the aspect of highest concern is represented by the definition of the importance factor γ_1 (EN 1998-1, 2005), as clearly stated in Koller and Kölz (2014).

For that reason, the importance factor has been selected according to the Italian regulations (Norme Tecniche per le Costruzioni, 2008), that suggests the adoption for critical facilities of a γ_1 factor equal to 2. As consequence, a nominal life of 100 years, and the return period associated with a 10% PoE within the nominal life is equal to 975 years.

For the purpose of the analyses, the plant is assumed located in a high seismicity area in Turkey (long: 22.9513000°, lat: 37.9400700°), which location is evidenced in figure 6.11.

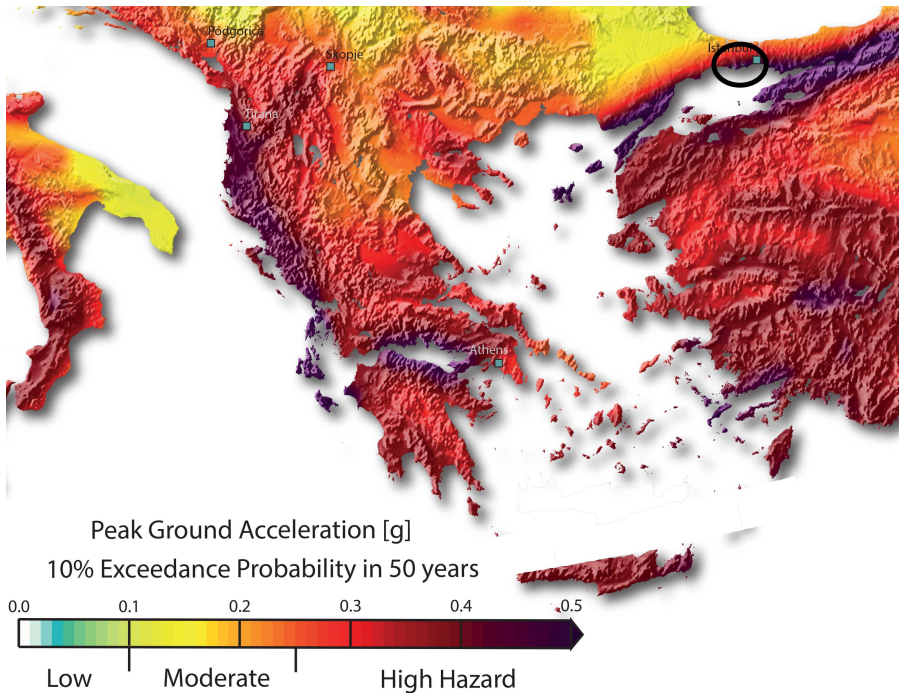


Figure 6.11: Seismic hazard map and identification of the area of interest in which the chemical plant is considered.

Figures 6.12 and 6.13 report the uniform hazard spectra and the seismic hazard curves associated with some relevant spectral pseudo-accelerations.

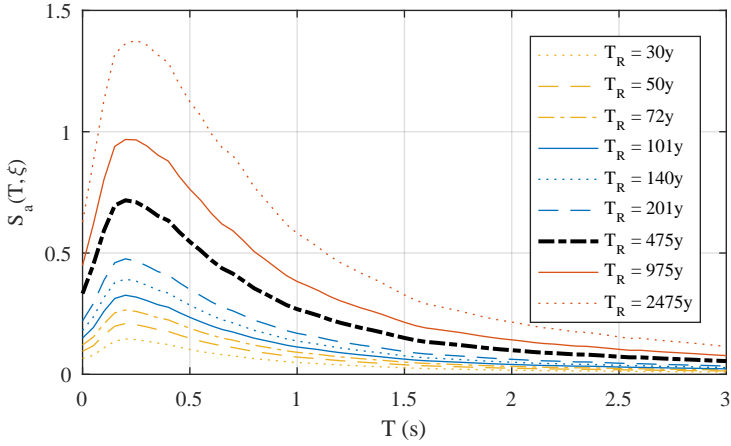


Figure 6.12: Uniform hazard spectra for the chemical plant.

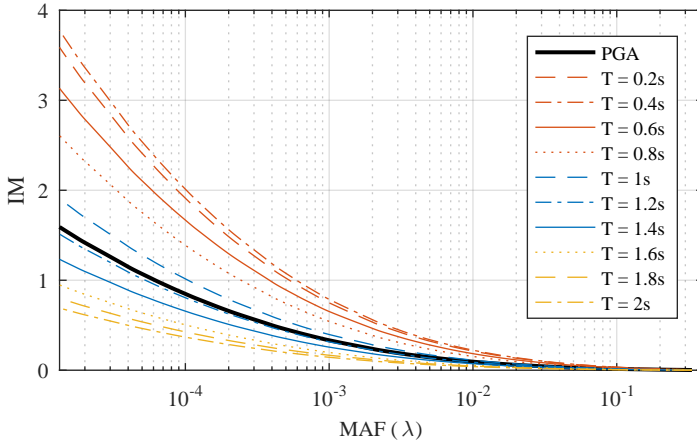


Figure 6.13: Hazard curve for the chemical plant.

Table 6.1: Main characteristics of the vessels and additional informations about the main structure-to-vessels connections. The bolts are equal for each bolted connection of the vessel.

Vessel	D [mm]	P [kN]	Restraints	Storey Restraints ^{*,**}	Connections
B100	2200	160	4 IPE270, h = 560 mm	1st (V+H)	2M14
B110	2500	160	4 IPE270, h = 560 mm	1st (V+H)	2M14
B150	1600	70	4 HEA280, h = 560 mm	1st (V+H)	2M14
B250	1400	80	4 HEA280, h = 560 mm	1st (V+H)	2M14
B310	1000	40	4 HEA280, h = 1580 mm	2nd (V+H)	2M14
B320	200	150	4 IPE270, h = 560 mm	1st (V+H)	2M14
B510	2600	240	4 HEA280, h = 560 mm	1st (V+H)	2M14
C200	2600	1200	anchor to the main beams	1st (V+H), 2nd and 3rd (H)	2M20
C250	1500	700	anchor to the main beams	1st (V+H), 2nd and 3rd (H)	2M20
K200	1300	200	anchor to the main beams	1st (V+H), 2nd (H)	2M20
K210	840	60	anchor to the main beams	1st (V+H)	2M14
K300	1900	120	anchor to the main beams	1st and 3rd (H), 2nd (V+H)	2M20
K400	1200	180	anchor to the main beams	1st (V+H), 2nd (H)	2M20
K500	2040	610	anchor to the main beams	1st and 3rd (H), 2nd (V+H)	2M20
V500	1440	100	anchor to the main beams	1st (V+H)	2M14
W200	720	45	anchor to the main beams	2nd (V+H)	2M14
W220	475	22	anchor to the main beams	1st (V+H)	2M14
W230	680	20	anchor to the main beams	1st (V+H)	2M14
W270	700	40	4 HEA280, h = 1580 mm	2nd (V+H)	2M14
W300	2375	160	anchor to the main beams	1st (V+H)	2M14
W510	2000	190	anchor to the main beams (beneath the floor)	3rd (V+H)	2M14

*V = vertical restraint;

**H = vertical restraint;

6.4 Resilience assessment

In this section, the seismic resilience of the Chemical Plant is assessed by means of the detailed and the multilevel procedures. Before the application of both the procedures, a brief description of common aspects, the system network configuration and the restoration schedule, is provided.

6.4.1 System network

As already stated in the description of the plant, the facilities are connected between themselves in series. To properly work, the production process requires the full functionality of the whole stand-alone units and facilities and the reduction of the functionality of a single facility leads to a reduced global productivity. Therefore, the block diagram representative of the system network can be represented by a set of units in *series* (see figure 6.14): the failure of one component (main structure or vessels) leads to the failure of the whole system.

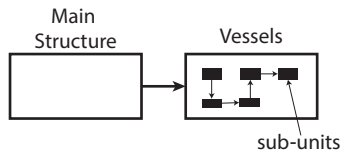


Figure 6.14: Block diagram of the chemical plant.

In this situation, the *minimum cut set* is represented by one facility that mostly reduces the plant's resilience. Considering a system network, the loss of resilience depends upon the vulnerability, the consequences, the connection among facilities and the restoration plan. Such values define the weight of each facility on the plant's resilience. It could be that one facility with a high vulnerability is associated with low consequences, being less influencing at the global level than a facility with medium vulnerability and medium consequences. Therefore, the execution of the analyses is required to establish which is effectively the *minimum cut set* of the system.

6.4.2 Restoration schedule

The Plant is characterized by a very compact in-plan configuration, that makes difficult the possibility to perform the restoration of each facility/-component simultaneously. This is due to interferences at the operation level, between storeys and adjacent facilities as well. Considering the lack of information, the *optimal restoration plan* was individuated through seven homogeneous macro-areas, defined for different storeys. Within each macro-area, summarized in table 6.2, in *series* operations has to be performed; among each macro-areas, in *parallel* operations can be performed.

Since each vessel that extends through more than one floor belong to a specific macro-area, the interferences among different storeys has been neglected. The same has been done for what concerns the interferences of operations performed at the same storey.

Table 6.2: Macro-areas used for the definition of the *optimal restoration schedule*

Macro-area	Facility/vessel
1	Main supporting structure
2	B250, B320, C200, C250, K300, K400, W200
3	B510, K200, K210, W230, W300
4	B100, B110, B150, K500, V500, W220
5	B310
6	W270
7	W510

6.4.3 Detailed Resilience assessment

The detailed resilience assessment, according to what reported in the chapter 3, is developed through the following three steps:

- vulnerability assessment by means of detailed analyses;
- definition of the consequences;
- estimation of the resilience and facilities prioritization ;

6.4.3.1 Vulnerability assessment

The vulnerability assessment is carried out by means of IDA analyses. In particular, it is considered that an exhaustive knowledge level is achieved for all the facilities. Therefore the mean value of the mechanical parameters of structural materials can be adopted. For the purpose of the analyses, being particularly difficult the execution of the detailed in-field surveys accompanied by extensive non-destructive and destructive tests for the characterization of materials, the parameters reported in table 5.1 in the previous chapter are adopted, and summarized here in table 6.3 for sake of clarity.

Table 6.3: Characteristics and mean values of the structural materials used for the vulnerability assessment of the two case studies.

	f_{yk} [MPa]	f_{ym} [MPa]	σ [MPa]	Distribution Reference	
Steel					
S235	235	328	15.9	Gaussian	Badalassi et al. (2017)

In the following, the definition of the numerical model and the results of the analyses are reported.

Detailed numerical model

Due to its particular configuration, the chemical plant is composed of drift sensitive and peak floor acceleration sensitive components:

- the main supporting structure is drift sensitive; its vulnerability is mainly associated with the plastic deformation of bracing systems and the attainment of sensitive values of interstorey drifts;
- the vessels placed on a single storey are peak floor acceleration sensitives;
- the vessels that get through two or three levels are drift sensitive.

Current literature can provide reliable analytical formulations for the evaluation of the floor response spectra, given an elastic structure. The evolution in the plastic field of the main structure might involve a strong modification of the peak floor response spectra, due to the period elongation of the damaged structure. This might lead to a strong modification of the ratio between the dynamic properties of the main structure and the supported element. Hence, a coupled analysis seems to be reasonable.

Chosen the coupled approach, remains the selection of the analysis method, linear or non-linear, static or dynamic. Obviously static analyses are not suitable to catch the dynamic amplification of the seismic action over the height. Moreover, linear dynamic analyses, when differences in the order of magnitude of the properties of structural and nonstructural components are present, often make numerical models ill-suited to the traditional modal analysis procedure. Step-by-step analyses are then more suitable and frequently used to catch such a kind of phenomena.

Therefore, in order to take into account, in a specific and detailed way, the interaction between the main structure and the supported elements and their mutual plastic deformations that might modify the dynamic properties, an Incremental non linear Dynamic Analysis (IDA) approach is chosen.

The selected model to represent the inelastic behaviour of the structural elements and the selection of the accelerograms is described in detail.

Inelastic modelling of structural members. The numerical model should be able to account for the post elastic behaviour of concentrically braced frames and the plastic deformation of the supported elements, which performance is mainly ruled by the flexural behaviour.

For such kind of structures, being the plastic deformation of the compressed element a key factor for the correct assessment of the post-elastic performances, several approaches are proposed in literature.

The most reliable and refined model is given by Uriz (2008), and its further improvements (Chao et al., 2008). The authors, adopting the open source software OpenSees (Mazzoni et al., 2006), proposed an approach based on the explicit modelling of the braces with their initial imperfection through a suitable number of force-based fiber elements. In such a way, the numerical model is able to catch the buckling and post-buckling phenomena, governed by a mixed axial flexural behaviour of the compressed braces.

In parallel to this modelling approach, the American regulations FEMA 356 (FEMA 356, 2000), and the most recent FEMA P-58 (Hamburger et al., 2012), proposes a lumped plasticity approach. As shown in figure 6.15, the force displacement in the post elastic field has two different branches, one in compression, with a softening branch representative of the post-buckling behaviour, and one in tension, with a elastic plastic behaviour. Both the sides drop at a displacement eight times the yielding displacement to simulate the fracture.

The model proposed by Uriz (2008) clearly represent the most precise solution. Anyway, the experience evidenced that such modelling approach ap-

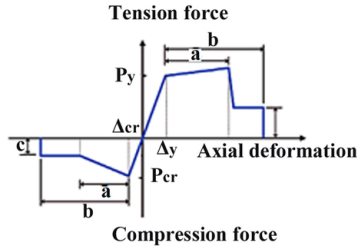


Figure 6.15: Tension and compression behaviour adopted for the lumped plasticity approach proposed by the FEMA 356 (2000) and Hamburger et al. (2012).

plied to large numerical models might cause several numerical instabilities and difficulties to achieve the convergence at very high plastic deformations. Therefore, all the braces are modelled adopting the FEMA 356 (2000) approach, that provide enough accuracy in the results and higher numerical stability.

All the vessels' chevron beams and supporting columns are modelled with lumped plastic hinges, representative of the nonlinear flexural behaviour of steel elements, as suggested by the FEMA 356 (2000).

Selection of accelerograms. Different solutions for the selection of accelerograms are available in literature. Real, artificial or synthetic accelerograms might be adopted. A detailed description about the several solutions available for the selection and scaling of accelerograms sets is provided in Venture (2011).

As suggested by several feedback in the technical literature, the adoption of real accelerograms seems to be most suitable. One of the main issues associated with the adoption of real accelerograms is related to the strong dispersion of results, in particular when the structure develop important plastic deformations close to the collapse situation.

The natural accelerograms can be selected referencing to different response spectra, chosen as a target. Among all, the most used are the uniform hazard spectrum (UHS) and the conditional mean spectrum (CMS) (Baker, 2010). It has been observed that records consistent with the UHS, although its shape is consistent with almost all the worldwide regulations, tends to provide excessive failure probability results. On the other side, the CMS, maximizing the pseudo-spectral acceleration in correspondence of only one period (or, at least a narrow band of periods), yields apparently to more realistic results in terms of failure probability. Anyway, it is not code consistent. Therefore, for the purpose of the following analyses, the UHS, being code consistent and providing results on the safe side is adopted.

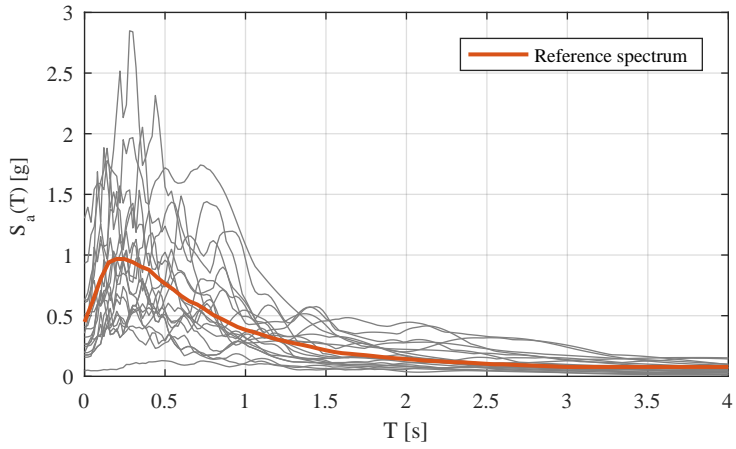
The following selection and scaling procedure is adopted:

1. definition, for the specific site, of the reference response spectrum of the horizontal directions adopting the UHS, defined for 10% PoE within 100 years;
2. the reference response spectrum for the vertical action is derived from the horizontal UHS adopting the vertical-to-horizontal response spectra correlation provided by Bommer et al. (2011);
3. selection of a suite of 20 unscaled real records, adopting the algorithm proposed by Baker and Lee (2017); the matching is exploited through the following sub-steps:
 - definition of the geometric mean of the horizontal response spectra;
 - detection of a suite of unscaled accelerograms, which means response spectrum shape fits the reference UHS;
4. spectral matching, by means of the SeismoMATCH software (Abrahamson, 2006), of the two horizontals and vertical records, to fit the mean response spectra with the reference one;
5. definition of scaling factors in order to match the mean PGA with the PGAs calculated at the following probabilities of exceedance (PoE): [3%, 5%, 10%, 22%, 30%, 39%, 50%, 63%, 81%]. Additional scale factors, to reach a PGA equal to [2, 3, 4, 5]g, are defined to achieve the collapse of the main structure and all the supporting elements of the vessels, in order to correctly build the fragility curves.

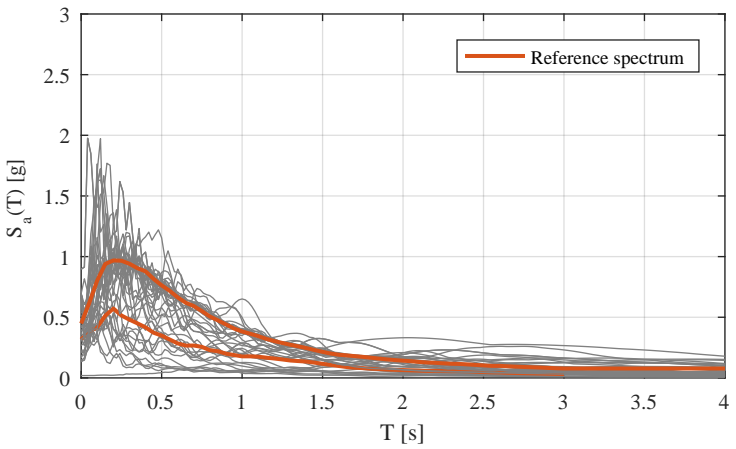
Figures from 6.16 to 6.18 show the response spectra, unscaled and matched, for the three principal directions, defined as east-west (E-W), north-south (N-S) and vertical (V). The list of the selected accelerograms is shown in table 6.4.

Table 6.4: List of the 20 real accelerograms selected from the PEER database.

Earthquake num.	ID	Earthquake Name	Magnitude	Mechanism	Rrup (km)	Vs30 (m/sec)	Year
1	549	Chalfant Valley-02	6.19	strike slip	17.17	303.47	1986
2	1489	Chi-Chi Taiwan	7.62	Reverse Oblique	3.76	487.27	1999
3	1487	Chi-Chi Taiwan	7.62	Reverse Oblique	35	520.37	1999
4	1549	Chi-Chi Taiwan	7.62	Reverse Oblique	1.83	511.18	1999
5	2734	Chi-Chi Taiwan-04	6.2	strike slip	6.2	553.43	1999
6	1793	Hector Mine	7.13	strike slip	196.78	525.79	1999
7	178	Imperial Valley-06	6.53	strike slip	12.85	162.94	1979
8	164	Imperial Valley-06	6.53	strike slip	15.19	471.53	1979
9	1118	Kobe Japan	6.9	strike slip	31.69	312	1995
10	779	Loma Prieta	6.93	Reverse Oblique	3.88	594.83	1989
11	741	Loma Prieta	6.93	Reverse Oblique	10.72	476.54	1989
12	1633	Manjil Iran	7.37	strike slip	12.55	723.95	1990
13	451	Morgan Hill	6.19	strike slip	0.53	561.43	1984
14	496	Nahanni Canada	6.76	Reverse	4.93	605.04	1985
15	963	Northridge-01	6.69	Reverse	20.72	450.28	1994
16	959	Northridge-01	6.69	Reverse	14.7	267.49	1994
17	1004	Northridge-01	6.69	Reverse	8.44	380.06	1994
18	540	N. Palm Springs	6.06	Reverse Oblique	6.04	425.02	1986
19	1510	Chi-Chi Taiwan	7.62	Reverse Oblique	0.89	573.02	1999
20	68	San Fernando	6.61	Reverse	22.77	316.46	1971

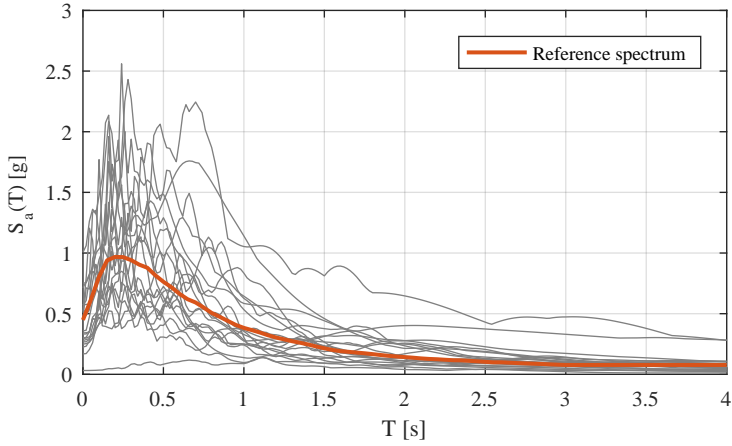


(a) E-W unscaled

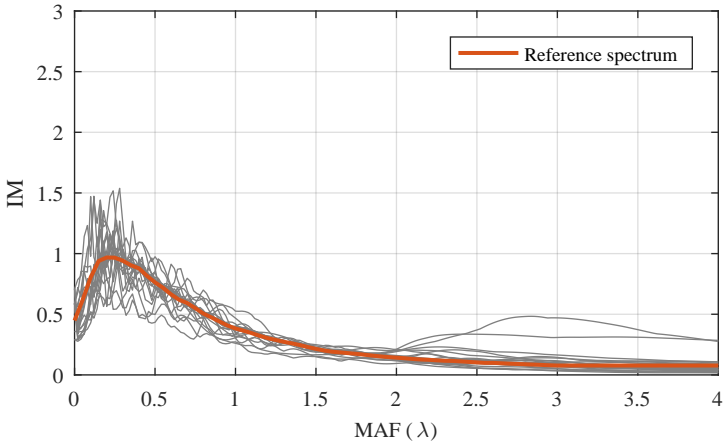


(b) E-W matched

Figure 6.16: Response spectra, unscaled and matched for the E-W direction.



(a) N-S unscaled



(b) N-S matched

Figure 6.17: Response spectra, unscaled and matched for the N-S direction.

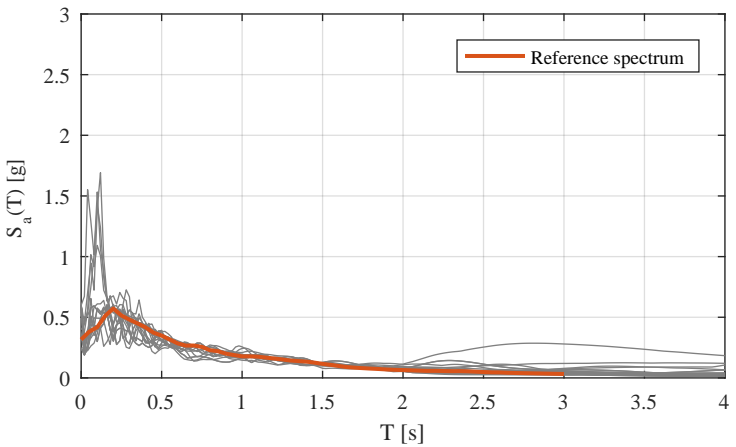
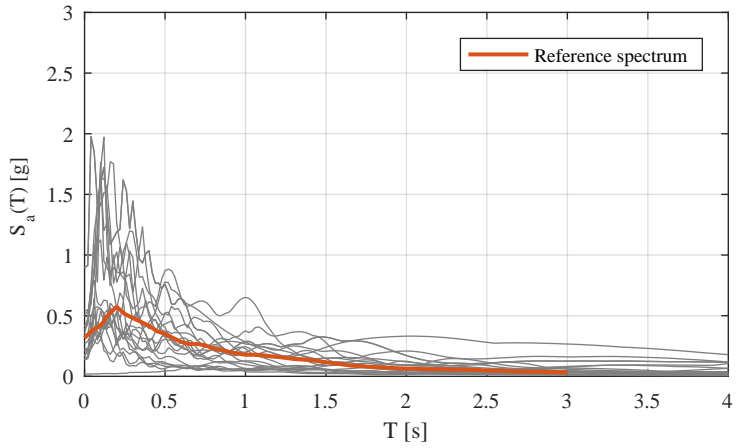


Figure 6.18: Response spectra, unscaled and matched for the vertical direction.

Results of the vulnerability assessment

The Incremental Dynamic Analyses (IDAs) were executed applying simultaneously the three components (2 horizontal and 1 vertical) of each ground motion and using 13 scale factors. A total of 520 nonlinear time-history analyses were carried out (20 ground motions \times 13 scale factors \times 2 directional combinations) (see Morelli et al. and Faggella et al. (2013)).

Definition of fragility curves The fragility curves are computed from the IDA curves, adopting an EDP based approach (Iervolino, 2017). In particular the method proposed by Baker (Baker, 2010) are adopted in this study. For the computation, the reference period with which calculate the IM $S_a(T_1^*)$ is the geometric mean of the two fundamental periods (associated with a translational mode shape that involve the maximum participating mass) along the two directions. Considering $T_{1,x} = 0.53$ s and $T_{1,y} = 0.70$ s, the reference IM is equal to 0.65g.

The seismic vulnerability of the main structure is ruled by the post elastic behaviour of the diagonal braces. The plastic behaviour concentrates at the first storey, yielding a soft-storey collapse behaviour. As shown in table 6.5, the facility results correctly designed to withstand the design seismic action. The vulnerability of the vessels is associated whit the post elastic behaviour of the supports, either beams and columns, and the failure resistance to combined axial-shear loads of the bolted connection systems. Looking at tables 6.6 and 6.7, it can be clearly observed that also almost all the vessels are correctly designed to withstand the seismic action. Few vessels are characterized by a vulnerability index lower that the unit, while the B250 vessel highlighted to be highly vulnerable towards the seismic action.

Table 6.5: Vulnerability index and parameters for the definition of the fragility curves of the main structure.

Detailed assessment				
Mechanism	$S_a(T_1)$	β	I_v	
DL Brace yielding	0.20	0.02	-	
SD Brace plastic deformation	0.52	0.10	-	
NC Brace collapse	0.71	0.25	1.09	

$S_a(T_1) = \exp(\eta)$

Table 6.6: Mean value and standard deviation for the estimation of the fragility curves of the vessels, adopting the detailed assessment (T_1 refers to the period of the principal modal shape of the peak floor acceleration sensitive components).

Vessel	$S_a(T_1)$ DL	$S_a(T_1)$ SD	$S_a(T_1)$ NC	β DL	β SD	β NC
B100	1.16	1.16	1.16	0.20	0.22	0.24
B110	1.07	1.07	1.07	0.21	0.19	0.18
B150	0.75	0.75	0.75	0.33	0.33	0.33
B250	0.23	0.23	0.23	0.21	0.21	0.21
B310	0.65	0.65	0.65	0.45	0.45	0.45
B320	0.63	0.83	1.05	0.34	0.21	0.25
B510	0.65	0.65	0.65	0.25	0.25	0.25
C200	0.65	0.65	0.65	0.21	0.21	0.21
C250	1.52	1.52	1.52	0.23	0.23	0.23
K200	0.65	0.65	0.65	0.37	0.37	0.37
K210	2.29	2.32	2.40	0.05	0.06	0.07
K300	0.55	0.55	0.55	0.14	0.14	0.14
K400	0.65	0.65	0.65	0.25	0.25	0.25
K500	0.54	0.54	0.54	0.04	0.04	0.04
V500	0.58	0.58	0.58	0.04	0.04	0.04
W200	0.48	0.48	0.48	0.35	0.35	0.35
W220	1.70	1.97	2.12	0.28	0.29	0.35
W230	0.60	0.60	0.60	0.06	0.06	0.06
W270	0.65	0.65	0.65	0.06	0.06	0.06
W300	0.65	0.65	0.65	0.04	0.04	0.04
W510	1.05	1.31	1.58	0.39	0.34	0.32

Table 6.7: Vulnerability indexes and failure mechanism of the vessels.

Vessel	I_v	Mechanism DL	Mechanism SD	Mechanism NC
B100	1.93	connection	connection	connection
B110	1.78	connection	connection	connection
B150	1.28	connection	connection	connection
B250	1.38	connection	connection	connection
B310	1.08	connection	connection	connection
B320	1.05	connection	connection	connection
B510	1.08	connection	connection	connection
C200	1.08	connection	connection	connection
C250	2.53	connection	connection	connection
K200	1.08	connection	connection	connection
K210	3.82	connection	connection	connection
K300	0.92	connection	connection	connection
K400	1.08	connection	connection	connection
K500	0.90	connection	connection	connection
V500	0.97	connection	connection	connection
W200	0.80	connection	connection	connection
W220	2.83	supporting element	supporting element	supporting element
W230	1.00	connection	connection	connection
W270	1.08	connection	connection	connection
W300	1.08	connection	connection	connection
W510	1.75	connection	connection	connection

6.4.3.2 Definition of the consequences

The consequences have been estimated, for each resilience state (see sect. 3.3.1.1), according to sect. 3.2.1.3. The recovery time depends upon different quantities, reported in the following for the sake of clarity:

- time for the inspection (T_{insp}): after a seismic event, T_{insp} represents the time required, by a team of experts, to detect the entity of the damage on a structure; it is assumed that the survey can be executed by the Plant's technical office in 1 day, when the null-to-limited damage is expected; otherwise, when the damage is not negligible an expert team opinion is required, prolonging the time for the inspection.
- time for the introduction of safety measures (T_{sm}): time interval required to introduce temporary measures, aimed at ensuring the safety of workers in the surrounding and avoiding the aggravation of the damaged condition of the facility;
- time for the design (T_D): time interval required to design the operations to restore the structural integrity according the actual standards (e.g the Eurocodes);
- time for the replacement (T_{rep}): time interval required to actualize the operations to restore the structural integrity, replacing or operating on the damaged elements;
- time for demolition and reconstruction (T_R): time interval required to demolish and rebuild the whole facility to its previous undamaged condition, when the retrofit through local/global interventions is unfeasible from an economic perspective.

The residual functionality is a fixed value defined to ensure the safety of workers and avoid the aggravation of the damage condition.

Table 3.3, reported in this chapter in table 6.8 to ease the comprehension, shows the calculation of the consequences, t_{rec} and $q(t)$, for each resilience state.

In the following, the residual functionality and the recovery time for each resilience state is reported for the main supporting structure and for the vessels. Due to the lack of information, has been assumed that vessels, being constituted by a very similar geometry of the supports, are characterized by the same $q(t)$ and t_{rec} .

Main building. Regarding the main building, the operations are derived on the basis of the results of the vulnerability analysis.

The following operations are expected for each resilience state:

Table 6.8: Quantitative meaning of the Resilience states in terms of recovery time and residual functionality

Resilience state	Recovery time	Residual functionality
RS₀	$T_{RS_0} = T_{insp}$	$q_0(t) = \text{full}$
RS₁	$T_{RS_1} = T_{insp} + T_d + T_{rep}$	$q_1(t) = \text{partial (50 - 80 \%)}$
RS₂	$T_{RS_2} = T_{insp} + T_d + T_{rep}$	$q_2(t) = \text{partial (20 - 50 \%)}$
RS_{3,a}	$T_{RS_{3,a}} = T_{insp} + T_d + T_{rep}$	$q_{3,a}(t) = \text{null (0 \%)}$
RS_{3,b}	$T_{RS_{3,b}} = T_{insp} + T_R$	$q_{3,b}(t) = \text{null (0 \%)}$

RS₀: one day of inspection time is expected, during which the functionality is dropped to the 80% of the full functionality to ensure the safety of workers.

RS₁: the braces in tension achieved the elastic limit and the compressed members buckled. The time for the inspection is assumed equal to 7 days. Therefore, considering also what provided by the FEMA P-58 (Hamburger et al., 2012), 5 days for the design of the intervention, plus three additional day for the replacement of the buckled braces. A total recovery time of 15 days is expected. The residual functionality is dropped down to 80% to ensure the safety of workers during the inspection and during the replacement operations¹.

RS₂: the braces in tension develop plastic deformations, and sensitive residual deformations are expected. Thereby, to restore the functionality the plastically deformed braces have to be replaced and the and the undeformed condition of the whole facility must be restored. For the last requirement, a contrast structure, to which anchor hydraulic jackets, must be used to *force* the deformed structure in its original position. To this aim, considering the design and the execution of the operations, 3 weeks are expected. The residual functionality must be dropped at the 50% of the full load. In such a way, the vertical load, imposed by iron dusts and particles aspirated by the system during the melting process, is strongly lowered, increasing the structural safety towards the seismic actions.

RS_{3,b}: the collapse is reached when all the braces achieve the maximum plastic elongation. In this condition, the facility and the supported equipment must be completely reconstructed. The inspection time is set to 7

¹ It should be considered that the functionality is strongly reduced during the operations. In the other time intervals the functionality can be considered full.

days. The time for the demolition is expected equal to 5 days. Design time is assumed equal to 90 days, since the steel structure and all the supported needs to be revised, to make them compliant with actual construction standards. Furthermore, a strict collaboration between the structural engineer and the equipment producer is required to optimize the design process. The reconstruction is expected to be completed in 50 days (total time required equal 152 days), being industrial facilities usually modular. When the $RS_{3,b}$ is attained, the residual functionality is dropped to zero.

The supported Vessels. Regarding the supported vessels, the main source of vulnerability is associated with the plastic deformation of the supporting elements, either beams or columns, or the failure of the bolted connections. The following values of $q(t)$ and t_{rec} are used for each resilience state.

RS₀: one day of inspection time is expected, during which the functionality is dropped to the 80% to ensure the safety of workers.

RS₁: the supported element reached the plastic deformation, associated with the DL limit state. The following recovery times are expected:

- 7 days for the inspection;
- 3 days to design the intervention;
- 2 days to introduce safety measure, to shut-off the pipelines from the vessels and to, if required, temporarily remove the supported vessels;
- 3 days to execute the intervention.

Thus, a total recovery time of 15 days is expected. The residual functionality is dropped down to 80% to ensure the safety of workers during the inspection and during the replacement operations.

RS₂: the supported element reached the plastic deformation, associated with the SD limit state. The following recovery times are expected:

- 7 days for the inspection;
- 5 days to design the intervention;
- 3 days to introduce safety measure, to shut-off the pipelines from the vessels and to temporarily remove the supported vessels, if necessary;
- 5 days to execute the intervention.

Thus, a total recovery time of 20 days is expected. The residual functionality is dropped down to 50% to ensure the safety of workers during the inspection and during the replacement operations.

RS_{3,b}: the collapse is reached when at least the 50% of the supporting elements reaches the NC limit states when one of the connections fails. In this condition, the supporting elements and the vessel as well must be completely reconstructed. The inspection time is set to 7 days. The time for the demolition is expected equal to 7 days, including the introduction

of safety measures and the removal of connected pipelines. The time for the design is assumed equal to 15 days, since the supporting steel structure and the vessel's structural capacity need to be revised, to make them compliant with actual construction standards. The reconstruction is expected to be completed in one month, being industrial facilities usually modular. Hence, a total recovery time close to two months is expected. When the $RS_{3,b}$ is attained, the residual functionality is dropped to zero.

6.4.3.3 Estimation of the resilience and facilities prioritization

In this paragraph, the results obtained from the application of the detailed PBEE-based resilience assessment of the chemical plant are provided. As it has been done for the steel mill, a Monte Carlo analysis is adopted to account for the uncertainty associated with the recovery time. Therefore, also in this case, a stochastic population of 10^5 samples, generated adopting a Sobol sampling strategy has been adopted.

Also for the Chemical Plant, the PBEE resilience assessment is performed in two steps:

1. the first aims at drawing the resilience curve, its confidence bounds and an estimation of the EAL curve;
2. the second is a sensitivity-based assessment aimed at identify the weight of each facility on the resilience of the whole plant.

Step 1. The resilience curve is shown in figure 6.19. The confidence bounds, also in this case, are calculated adopting the parametric bootstrap resampling method. The mean value of the EAL is equal to 0.53 days and the C.O.V. is equal to 1.15 (see figure 6.20).

It can be observed that the Residual functionality starts at MAF values close to 10^{-2} , as a direct consequence of the low vulnerability index of the assessed facilities.

Step 2. The second step, focuses on the definition of a hierarchy among facilities, through a linearized method, based on the SRC method. The results of the sensitivity analysis are reported in figure 6.21. It can be observed that the only few facilities that influence the EAL are reported in figure 6.21. It's worth noting that, being the vessels associated with the same consequences, the facilities prioritization is ruled by the vulnerability index.

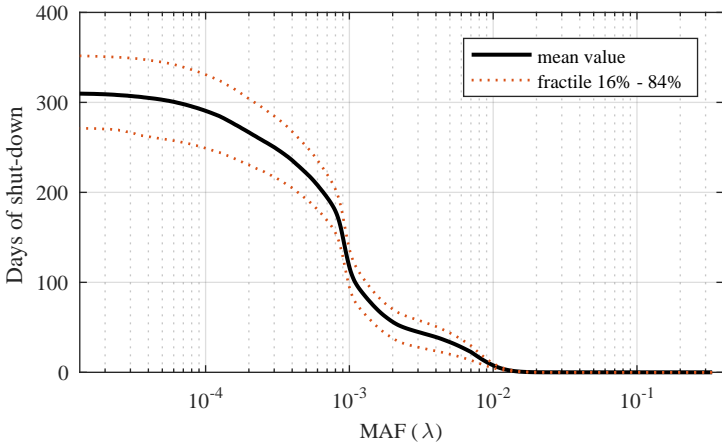


Figure 6.19: Resilience curve and its confidence bounds (16% and 84% quantile) used for the estimation of the EAL of the chemical plant.

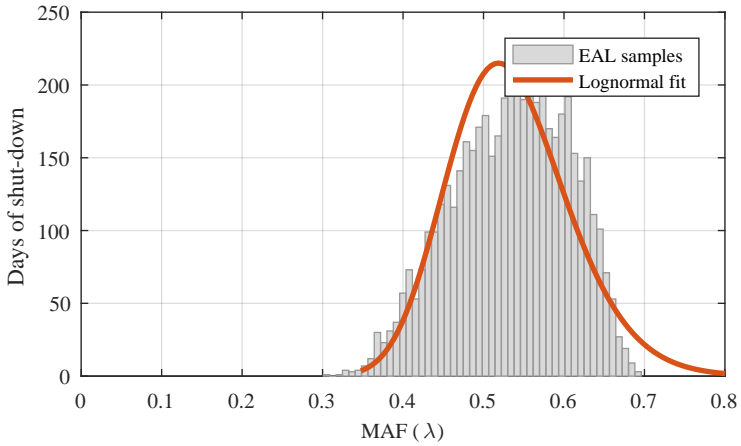


Figure 6.20: Distribution of the EAL and lognormal fit for the chemical plant.

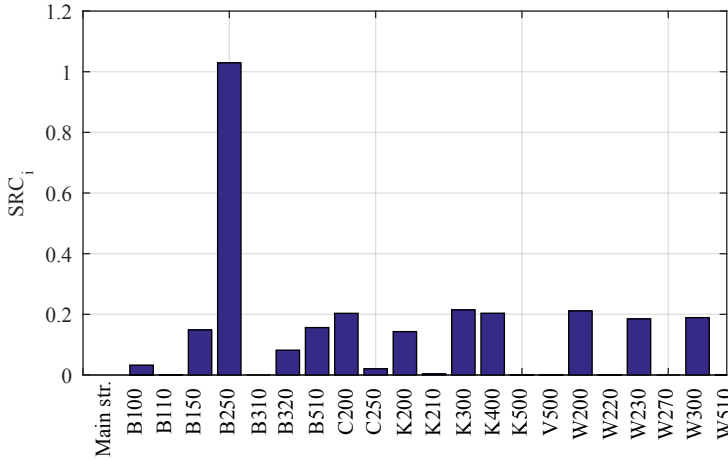


Figure 6.21: Distribution of the SRC ranks over the main building and the 21 vessels.

6.4.4 Application of the multilevel procedure

The application of the procedure to the steel mill is developed through the following steps:

- identification of the break-even point (t_{BEP}), representative of the maximum days of shut-down the Plant can withstand without making economic losses;
- planning of the walk down, aimed at the attainment of a minimum knowledge level to perform simplified structural analyses;
- vulnerability assessment by means of simplified numerical models;
- application of the three levels procedure and identification of the critical structures.

It should be noted that the procedure can requires more than the three iterations. In fact, it is possible that the facilities that results critical at the first level, could appear to be less critical at the second level. Therefore, further steps could be indispensable to achieve at least the resilience-neutral condition, i.e. when $EAL = t_{BEP}$.

In the following, the above-mentioned aspects are described in detail.

6.4.4.1 Break even point

For the calculation of the break even point, it is necessary considering the fact that the chemical plant serves a set of other neighbor industries; thus, the failure, either partial or full, might lead to enormous economic losses. Thereby, according with the technical office of the plant, the t_{BEP} is fixed to one day.

6.4.4.2 The walk-down

The details that should be inspected during the walk-down are reported in the following for the main structure and the vessels as well.

Main structure. It's worth noting that the available technical drawings are realized considering different possible sections for the braces, constituted by the same gross section area. In this way, the designer of the equipments has an higher chance to select the sections that interfere as less as possible with the networks of pipelines and the positioning of vessels. Therefore, the preliminary walk-down of the main structure must be focused on the section of the braces and on the typology of their end connections.

Vessels. The main aspects that worth to be checked are the supporting elements and their system of connections. In fact, being in the past usual to design such components towards only the gravitational loads, the connections result down-sized or even absents.

6.4.4.3 Application of the procedure

Before the description of the results obtained applying the multilevel procedure, the model adopted, simplified and detailed as well, and the results of the vulnerability assessment are briefly summarized and commented in the following.

Vulnerability assessment In this paragraph, the vulnerability analysis performed for the first and the second levels are briefly summarized for both the main structure and the vessels.

- For the main structure:
 - *Level 1*: the storey is modelled as an infinitely rigid diaphragm and the storey masses are uniformly distributed over each storey;

pushover analysis is performed, and the fragility curve is derived with the SPO2IDA tool (Vamvatsikos and Cornell, 2005). The results of the simplified assessment, reported in table 6.9, show a negligible vulnerability of the main structure at the first level. Moreover, the adopted simplifications lead to an estimation of the vulnerability lower than the detailed procedure.

- *Level 2*: the stiffness of each floor is correctly modelled and the seismic masses, associated with the weight of each vessel, are applied in the correct position of the vessels. Such a model is used for the execution of linear time history analyses, aimed at deriving the floor response spectra.
- For the vessels:
 - *Level 1*: the estimation of the seismic amplification is calculated with the VCI guidelines (Butenweg et al. (2012a) and Butenweg et al. (2012b)). A linear model is used to define the expected seismic intensity that might trigger the collapse. The dispersion β , required to build the fragility curve, is assumed equal to 0.5, according to FEMA P-695 (2009). The results of the vulnerability assessment are provided in table 6.10, in which is shown an high vulnerability of the vessels, mainly due to the approach of the VCI guidelines (Butenweg et al. (2012a) and Butenweg et al. (2012b)), which provides peak floor acceleration higher than the values effectively expected.
 - *Level 2*: the performance-based seismic analysis is adopted, as previously described in sect. 4.3.1: the vessel support is modelled, considering all the sources of non-linearity, geometric and related with material as well, and a pushover analysis is performed. The seismic performance is derived from the ADRS spectrum, adopting the floor response spectra. The results of the vulnerability assessment of the vessels analysed at the second level are reported in table 6.11, while the response spectra are graphically depicted in figures from 6.22 to 6.23.

For the critical facilities, the results analyses at the second and third levels can be improved with the following considerations.

- Regarding the *main structure*, the knowledge level might be increased by means of
 - additional detailed surveys, also performing destructive or non-destructive tests for the characterization of materials,

- refining the model, considering the effective structural behaviour of each storey, adopting, for instance, more precise and reliable analysis methods, e.g. IDA analyses (level 3).
- detailing the estimation of the consequences.
- Regarding the vessels, either the peak floor sensitive and the drift sensitive ones, the reliability of the results can be achieved, from one iteration to the next:
 - defining, in a more precise way, the peak floor acceleration spectra (level 2);
 - performing more detailed analyses of the post-elastic behaviour of the structural system (level 2);
 - performing coupled nonlinear dynamic analyses (level 3), whether both the main structure and several vessels require detailed assessment;
 - detailing the estimation of the consequences.

Also in this case, being impossible for the scope of the thesis to execute detailed in-filed surveys and the experimental campaign for the characterization of materials, the characteristics and mean values, reported in table 6.3, are adopted.

For the purpose of the analysis of the case study, three steps are applied to achieve the t_{BEP} . In particular:

- the first step focuses on the application of the first level;
- the second and third steps focus on the application of the second level, in order to seek at attaining the break even point.

The third level is never applied, because the resilience-seeking condition is attained at the third step.

The results of the vulnerability analyses, with simplified (first level) and detailed (second level) approaches, regarding both the main structure and the vessels are briefly summarized in the following, in order to provide more space on the estimation of the resilience at the various level of the procedure.

6.4.4.4 Application of the first level

At the first step, the first level is applied. In order to apply the first level, two steps are mandatory: the definition of a simplified model and the estimation

Table 6.9: Vulnerability index and parameters for the definition of the fragility curves of the main structure; T_1 is equal to 0.23 s

Simplified assessment				
X direction				
Mechanism	$S_a(T_1)$	β	I_v	
DL Brace yielding	0.11	0.14	-	
SD Brace plastic deformation	0.19	0.27	-	
NC Brace collapse	0.25	0.31	1.34	
Y direction				
Mechanism	$S_a(T_1)$	β	I_v	
DL Brace yielding	0.20	0.02	-	
SD Brace plastic deformation	0.52	0.10	-	
NC Brace collapse	0.71	0.25	1.76	

$S_a(T_1) = \exp(\eta)$

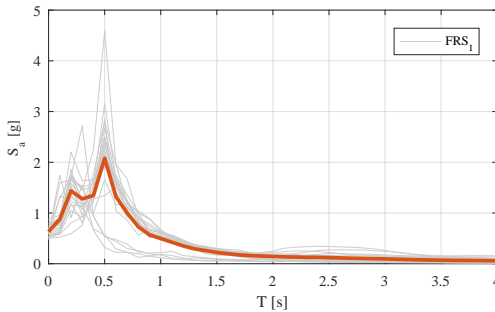


Figure 6.22: Floor response spectra at the first level, along the x direction.

of the consequences with the consequences matrices.

As previously stated in the vulnerability assessment section (sect. 6.4.4.3), the following models are used. The main building vulnerability is assessed through a nonlinear analysis performed on a simplified model. For what concern the vessels, simplified numerical models have been used to derive the maximum horizontal force that yield the first element or trigger the failure of a bolted connection. Considering the vertical amplification, as suggested by the VCI guidelines (Butenweg et al. (2012a) and Butenweg et al. (2012b)) for simplified analyses, the horizontal force is tramuted as a PGA. The dispersion β is implicitly considered by means of the tabular format proposed

Table 6.10: Results of the simplified vulnerability analysis of the vessels, performed for the first level.

Vessel	max a_g	β
B100	0.27	0.50
B110	0.34	0.50
B150	0.99	0.50
B250	0.86	0.50
B310	1.73	0.50
B320	0.15	0.50
B510	0.29	0.50
C200	0.20	0.50
C250	0.33	0.50
K200	1.17	0.50
K210	1.20	0.50
K300	1.95	0.50
K400	1.30	0.50
K500	0.38	0.50
V500	0.17	0.50
W200	1.53	0.50
W220	3.14	0.50
W230	3.45	0.50
W270	1.05	0.50
W300	0.13	0.50
W510	0.10	0.50

Table 6.11: Results of the vulnerability assessment of the vessels investigated with detailed procedure at the second level.

Vessel	DL		SD		NC	
	Sa	beta	Sa	beta	Sa	beta
B150	0.55	0.37	0.55	0.37	0.55	0.37
B250	0.19	0.26	0.19	0.26	0.19	0.26
C250	1.25	0.26	1.25	0.26	1.25	0.26
K200	0.59	0.28	0.59	0.28	0.59	0.28
K300	0.45	0.15	0.45	0.15	0.45	0.15

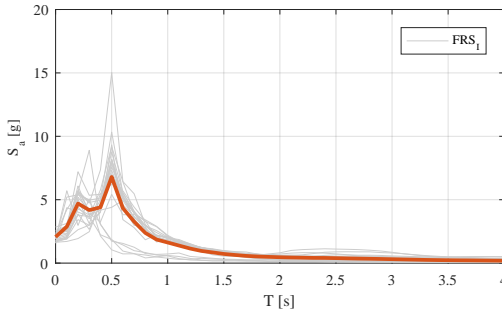


Figure 6.23: Floor response spectra at the second level, along the x direction.

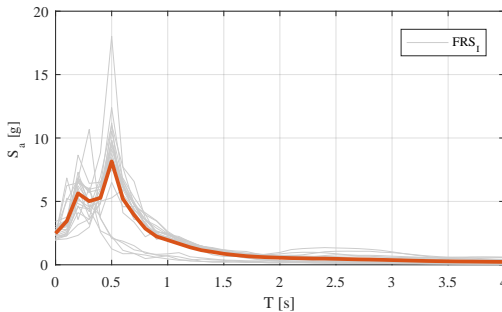


Figure 6.24: Floor response spectra at the third level, along the x direction.

Table 6.12: Consequences estimated with a the simplified approach of the consequence matrices.

Facility	T_R	RS0	RS1	RS2	RS3b
Main structure	200	0 (1.0)	0.1 (0.8)	0.5 (0.5)	1.0 (0.0)
Vessels	60	-	-	-	1 (1.0)

by the FEMA P-695 (2009).

The consequences are estimated by means of the consequences matrix approach and are reported in table 6.12. It should be noted that the consequences of the vessels are estimated only for the $RS_{3,b}$, being the fragility curves available for the collapse limit states.

At the first step, the EAL is equal to 1.49 days. The resilience curve is provided in figure 6.25.

In order to achieve the t_{BEP} , the Resilience indicators obtained applying the optimization procedure are shown in table 6.13.

Table 6.13: RI of the chemical plant after the application of the first step.

Facilities	RI
K300	0.35
B250	0.35
C200	0.42
C250	0.48
K400	0.51
W510	0.59
B150	0.71
K200	0.71
W230	0.71
W300	0.71
B510	0.71
K210	0.71
B320	0.76
W200	0.76
B100	0.77
B110	0.77
K500	0.77
V500	0.77
W220	0.77
B310	1.0
W270	1.0
Main Structure	1.05

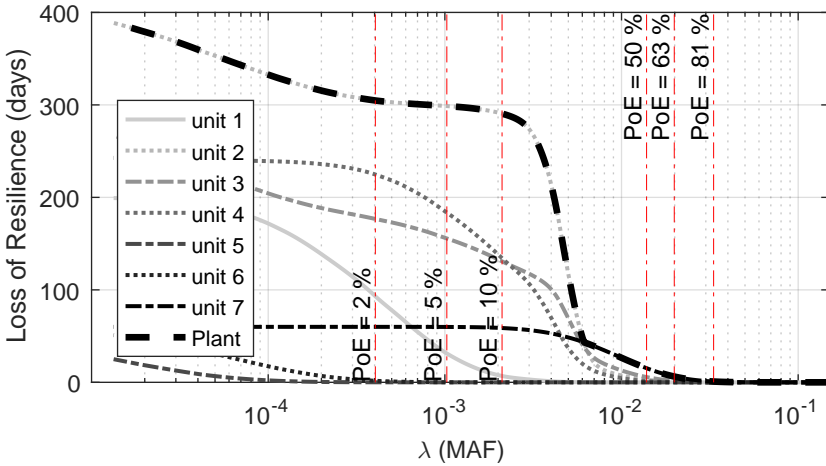


Figure 6.25: Resilience curve of the plant at the first level (units are the macro-areas reported in table 6.2).

6.4.4.5 Application of the second level

The second level is applied in two iterations, corresponding to the second and third step of the procedure.

First iteration. At the first iteration, the four facilities with RI lower than 0.5 are considered. Therefore, being the vessels the critical elements, the procedures, described in detail in 6.4.4.3, are performed and briefly hereinafter recalled:

- the peak floor acceleration spectra are derived by means of a time history linear analysis, performed on the main structure adopting the accelerograms selected in sect. 6.4.3.1 and reported in table 6.4;
- the critical vessels (K300, B250, C200, C250, K400), considering the achievement of an exhaustive knowledge, are analysed in detail through a non linear static analysis.

At the end of the first iteration, the EAL results improved and is equal to 1.12 days. Although really close, it does not match the requirements of the break even point. Hence, a further step is required. For this purpose the RI are newly calculated and reported in table 6.14.

Second iteration. At the second iteration, the second level is applied on the two critical facilities **B150** and **K200**, both characterized by a RI equal to 0.87

Table 6.14: RI of the chemical plant evaluated at the second step (first iteration on the second level) of the multilevel procedure.

Facilities	RI
B150	0.87
K200	0.87
K300	0.89
B250	0.89
C200	0.89
C250	0.89
K400	0.89
W510	0.89
W300	0.89
B510	0.89
K210	0.89
B320	0.89
W200	0.89
B100	0.95
B110	0.95
K500	0.95
V500	0.95
W220	0.95
W230	1
B310	1
W270	1
Main Structure	1.41

(see table 6.14). The execution of the refined analyses, belonging to the second level, on the two critical facilities, yields to a reduction of the EAL to 0.93 days. Thus, the resilience-seeking condition is achieved ($EAL < t_{BEP}$) and the application of the multilevel procedure can be considered concluded.

6.5 Concluding considerations

In this chapter the seismic resilience of the chemical plant case study is assessed applying the detailed and the multilevel procedures. The comparison of the results allows the validation of the reliability of the multilevel procedure.

The plant is characterized by a very compact structure that support a conspicuous number of vessels. The main structure has a bracing system along both the principal directions and is drift sensitive, while almost all the vessels are sensitive to the peak floor acceleration. This lead to the definition of different numerical models, for both the simplified and the refined analyses. Obviously, the application of the simplified procedures, i.e. the first level of the procedure, required the introduction of strong hypotheses on the structural behaviour of the facilities.

For what concern the detailed resilience assessment, the particular configuration of the main structure, without a rigid diaphragm floor, with a non-uniform distribution of the stiffness and masses over the height and the presence of vessels sensitive to peak floor accelerations, practically imposed the adoption of nonlinear dynamic analyses. Thus, IDA protocol was applied. Regarding the estimation of the consequences, detailed analyses has been performed for all the facilities and all the resilience states.

The detailed resilience assessment evidenced that the plant is not sensitive to seismic events with a frequency higher than $\approx 9 \cdot 10^{-1} 1/\text{years}$, as it can be clearly observed in figure 6.26. An EAL equal to 0.53 days is expected (see table 6.15), evidencing an high resilience of the Plant towards the seismic action. At low probability seismic events ($MAF \leq 10^{-4} 1/\text{years}$) the expected days of shutdown are always lower than one year.

The facilities prioritization evidenced that the main structure does not influence the resilience. Therefore the problem is mainly governed by the seismic behaviour of the vessels.

Table 6.15: EAL, expressed in days of shut down, obtained from the application of the detailed approach and the multilevel procedure.

Procedure	1 st step	2 nd step	3 rd step
Detailed procedure	0.53*		
Multilevel procedure	1.49	1.12	0.93

*C.O.V = 1.15

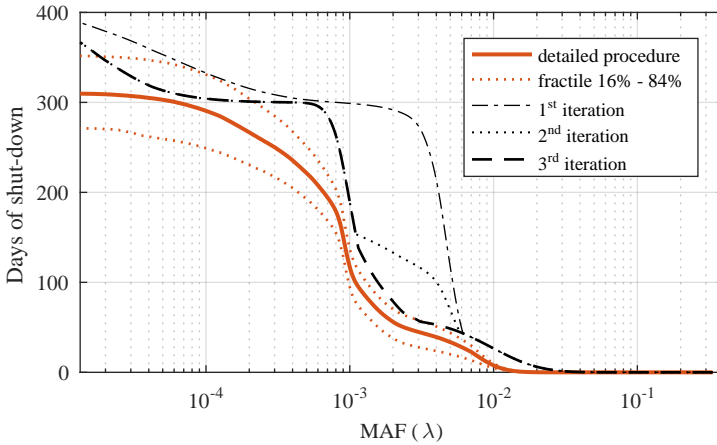


Figure 6.26: Comparison of the resilience curves obtained applying the detailed and the multilevel procedures.

Regarding the multilevel procedure, several simplifications are introduced to perform simplified structural analyses of the main building and the vessels at the first level. In particular, to perform a non-linear static analysis of the main structure, the presence of a rigid floor diaphragm and uniformly distributed masses at each floor has been considered. The vessels have been analysed considering only the SD limit state. The analyses are performed in a simplified fashion to account for the peak floor accelerations.

Considering that the facilities observed as critical at the first step, once detailed assessments are carried out, might not result critical in the second step, the multilevel procedure has been applied with the following steps:

- the first step focused on a simplified analysis at the plant level, applying the first level;
- the second and third steps provided a deep insight of the critical units, applying only the second level;

At third step the resilience-seeking condition has been attained; therefore, the hypothesis about possible upgrading solutions to enhance the resilience wasn't necessary.

The first step gave rather high values of the EAL (1.49 day), if compared with the detailed analyses. In fact, the resilience curve resulted greater than the one obtained with the detailed assessment, in particular for MAF lower than 10^{-2} 1/years. The problem is governed by the relatively high seismic vulnerability exhibited by the vessels, when analysed by means of the simplified method.

Specializing the knowledge associated with the structural performance and the consequences of the critical vessels, at the second and third steps, the resilience curve tends to fit the upper bound of the resilience curve obtained from the detailed assessment. The first level provided an estimation of the loss of resilience much higher than the detailed procedure, because the procedure for the estimation of the maximum floor amplification, proposed by the German VCI guidelines (Butenweg et al. (2012a) and Butenweg et al. (2012b)), always gives results on the safe side.

With only three steps, the multilevel procedure achieve satisfactory results, fitting the results of the detailed procedure. The differences in terms of EAL, for each step of the procedure are reported in table 6.15.

The good fitting of the multilevel procedure with the detailed procedure was achieved confining the detailed analyses on few vessels, gaining time and economic expenses (e.g. associated with specific in-situ surveys and experimental campaign for the characterization of structural material).

It's worth noting that the IDA analyses of the detailed procedure, required approximately 5 days of cpu-time for their execution. The structural analyses of the multilevel procedure, few pushover analyses, executed on the main structure and the vessels assessed at the second and third levels, and 20 linear time history analyses, to derive the peak floor acceleration spectra, required few hours on the same computer.

The Resilience indicators obtained at the second step are in very good agreement with the SRC sensitivity-based prioritization performed at the end of the detailed assessment, evidencing the goodness of the optimization procedure.

A comparison among the floor response spectra (FRS) extracted with the linear time history analysis and the one obtained with the EN 1998-1 (2005) (see equation (6.1)), should also be carried out, to establish if the calculation of FRS by means of linear time history analyses could provide real benefits.

$$S_a = a_g \cdot S \cdot \left(\frac{3(1+z/H)}{1+(1-T_a/T_1)^2} - 0.5 \right) \quad (6.1)$$

For the calculation of the FRS according to the Eurocode 8 (EN 1998-1, 2005), the ratio z/H (z = position of the non-structural element considered, H = height of the construction), the ratio T_a/T_1 (T_a = fundamental period of vibration of the non-structural element, T_1 = fundamental period of vibration of the structure) and the peak ground acceleration a_g are required. In the analysed case, the approach proposed by the Eurocode 8 is in good agreement with the FRS of the first storey (see figure 6.27), while tends to un-

derestimate the FRS, from two to four times, at the second and third storeys, as respectively shown in figures 6.28 and 6.29.

Hence, considering the fact that the aim is to obtain detailed results of the vessels' structural performances, expressed in terms of fragility curves, the approach proposed by the Eurocode 8 seems to be not particularly suitable for two reasons:

- the FRS proposed by the Eurocode 8 are not usually on the safe side; therefore the seismic intensity that trigger the collapse of the supported vessels is underestimated;
- the approach proposed by the Eurocode 8 provides any information about the dispersion β to build the fragility curves; the adoption of the empirical tabular approach proposed by the FEMA P-695 (2009), could solve this issue, but does not provide any further improvement from the level 1 of the multilevel procedure.

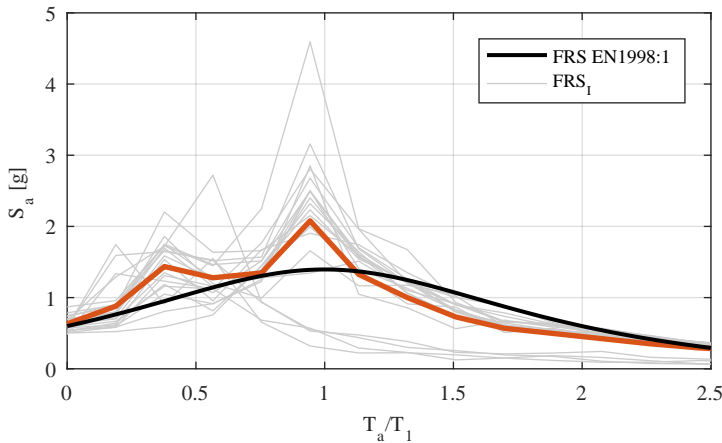


Figure 6.27: Floor response spectra of the first floor: detailed calculation versus EN1998:1 ($z/H = 0.350$).

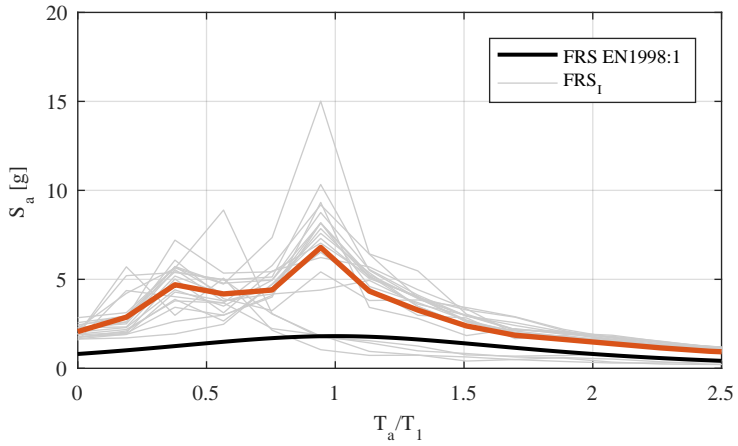


Figure 6.28: Floor response spectra of the second floor: detailed calculation versus EN1998:1 ($z/H = 0.70$).

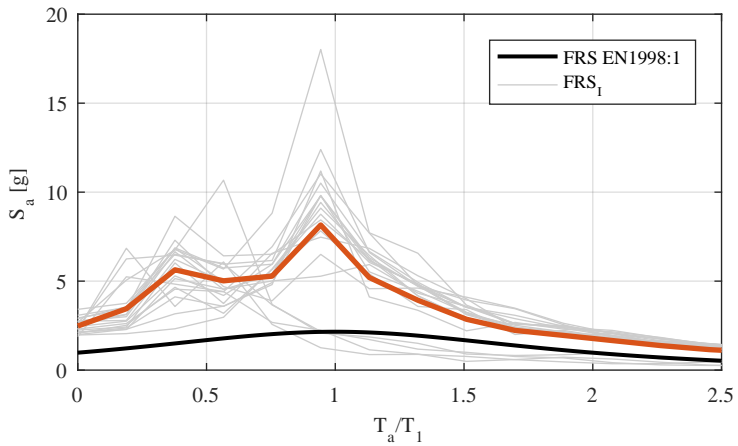


Figure 6.29: Floor response spectra of the third floor: detailed calculation versus EN1998:1 ($z/H = 1$).

Chapter 7

Conclusions and further developments

7.1 General conclusions

In the present work the problem of the seismic resilience of industrial plants, characterized by a sensitive interconnection among single facilities, has been investigated.

Initially, a brief introduction about the typical problem the industrial facilities are affected after a seismic event was provided.

Notwithstanding the importance of the resilience for the assessment of industrial facilities, a lack in the current scientific literature was observed. It was evidenced that risk analysis is frequently adopted, providing several methodologies, qualitative and quantitative as well.

The scope of the thesis is twofold: to develop a detailed procedure for the resilience assessment and the facilities prioritization; to develop a multilevel procedure for the resilience assessment of industrial plants, aimed at optimizing the analyses. Adopting a three level framework the idea is to provide an overview of the Resilience at the first level, specializing the results in the following levels, by means of detailed analyses executed only on few critical facilities.

Two case studies, a steel mill and a chemical plant, representative of actual industrial realities were carefully selected to apply and validate the proposed multilevel procedure. Since the detailed procedure is conceived to provide a very detailed information about the seismic resilience of the Plant, its results were used as a reference to validate the goodness and reliability of the multilevel procedure. The two case studies were chosen in order to analyse different situations.

The Steel Mill, indeed, is characterized by a huge in plan extension, and is

characterized by drift sensitive facilities, easily to schematize as single degree of freedom structures. On the contrary, the Chemical Plant has a compact configuration that develops over the height; it is characterized by a main braced building that support several vessels, mainly sensitive to peak floor acceleration.

Since the two plant have a strong different configuration, specific methods for the structural analyses, simplified and detailed as well, have been used.

The detailed procedure was applied adopting non-linear static analyses on the facilities of the steel mill, while for the chemical plant a IDA protocol was carried out on the entire plant, modelling all the supported vessels.

The detailed assessment showed very different results for the steel mill and the chemical plant. The former was characterized by an Expected Annual Losses (EAL), representative of the attended days of shutdown per year, equal to 8.46 days, 1.5 times higher than the break even point (t_{BEP} , the maximum shutdown time due to unforeseen events to avoid economic losses). The chemical plant, evidenced an EAL equal to 0.53 days, almost half of the t_{BEP} imposed by the owner (1 day).

For the steel mill, the facilities prioritization highlighted that the resilience is governed by few facilities, characterized by an high vulnerability and elevated consequences.

On the contrary, the resilience of the chemical plant was governed by several vessels, shown by the facilities prioritization. The main structure does not influence the seismic resilience of the plant.

The multilevel procedure is applied through several steps. In fact, after the application of the first level, the facilities outlined as critical, could not be equally critical after the refinement of the results at the second level. Such step can be applied several times to reach the convergence in terms of the detected critical facilities. Therefore, the achievement of the t_{BEP} might requires several steps.

For what concern the **steel mill** four steps were required to achieve an EAL lower than the t_{BEP} . The first step focused on the application of the first level. The second and the third steps focused on the application on the second level, refining the information about the vulnerability and the estimation of the consequences of the critical facilities. Since at the end of the third level, the neutral-resilience condition, i.e. $EAL \equiv t_{BEP}$ was not achieved, the fourth level provided the effectiveness of the seismic upgrading of the most critical facilities.

The **chemical plant** case study required three steps. Also in this case the first step focused on the application of the first level, characterized by as fast as simple procedures for the estimation of the vulnerability and the conse-

quences as well. The second and third steps detailed the information associated with the vessels, achieving at the end and EAL of 0.84 days, lower than the t_{BEP} .

As it is evidenced in figures 7.1 and 7.2, the application of the multilevel procedure fits gradually the results of the detailed assessment (red line). In fact, increasing gradually the knowledge of the critical facilities, the results are improved, reducing the error associated with the first *rough* level.

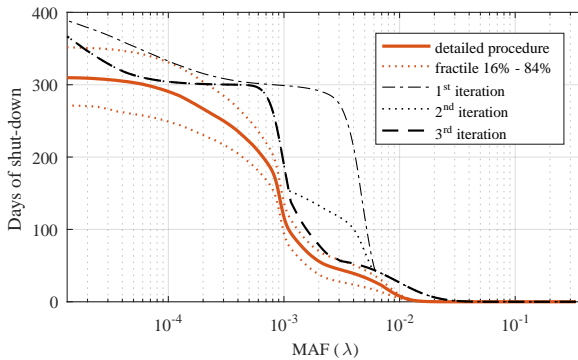


Figure 7.1: comparison of the resilience curves obtained applying the detailed and the multilevel procedures - chemical plant.

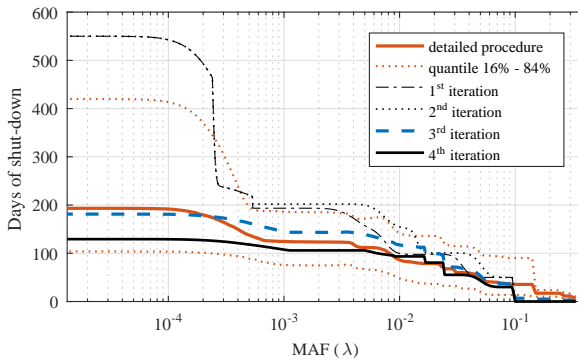


Figure 7.2: comparison of the resilience curves obtained applying the detailed and the multilevel procedures - steel mill.

The procedure for the seismic resilience assessment and facilities prioritization provides a clear tool for the estimation of resilience and the prioritization of facilities, directly exploitable, in a decision-making perspective, to plan interventions on the facilities. Clearly its application requires an elevated knowledge level of the facilities and the expected consequences. The multilevel procedure, as it was conceived, demonstrated to be a flexible tool that allow to optimize:

- the number of in-field surveys and the amount of detailed analyses, only on few critical facilities, avoiding a waste of economic and temporal resources;
- the interaction among different technical figures, the team of engineers in charge to perform the assessment and the technical office of the plant; their cooperation is mandatory to correctly define the consequences.

The quality of results obtained applying the multilevel procedure on the two case studies depend upon two different aspects.

The steel mill is characterized by rather simple facilities. Therefore, improving the complexity of the numerical models could not lead to significant improvement of the results. The only way to improve the results are the execution of further in-field surveys, through which lower confidence factors might be applied and the informations about the consequences can be specialized.

The chemical plant's resilience is governed by the vessels. Their analysis, at the first level, is performed with a very simplified method. Thus, at the second level, the results can be improved, carrying out refined structural analyses (static non linear analyses), increasing the knowledge level of the vessels by means of targeted in-field surveys and estimating the consequences in a detailed way.

Also the facilities prioritization, defined by means of the Resilience Indicators, is in good agreement with the results obtained from the detailed procedure. In particular, the RI obtained at the first and second iterations of multilevel procedure applied on the steel mill, and the second iteration of the procedure applied on the chemical plants, fit the results the detailed procedure.

7.2 Further developments

Further analyses on different case studies can certainly be developed in order to refine the procedure and validate its reliability on a wider ranges of

cases.

A mutual collaboration with the technical offices of the industrial Plants, would also allow to specialize the definition of the residual functionality and the consequences matrix. Such a collaboration would enrich the amount of available information, allowing the definition of a direct connection among the seismic damage, the residual functionality and the required recovery time to restore the fully operational condition.

Within th perspective to widen the applicability of the proposed procedures, the multi-hazard problem is of great importance. In fact, the presence of hazardous materials might cause additional consequences after the seismic damage, leading to huge structural, environmental and social losses. Accounting for such problems, the asset of the plant and the aspects on which pay more attention might strongly change, and further investigations and new measures aimed at gaining resilience should be developed. From this point of view, also the optimization procedure can be seen in a multi-objective perspective, joining the reconstruction costs of damaged facilities and casualties and injuries associated with the seismic event.

Appendix A

Vulnerability assessment of the Steel Mill

A.1 Vulnerability assessment

This section reports the vulnerability assessment of all the facilities. In particular, the numerical models, used for the detailed assessment and for the simplified approach, proper of the first level of the multilevel procedure, are described, along with the results of the vulnerability assessment.

A.1.1 Numerical models

Almost all the numerical models are realized in SAP2000 and translated in OpenSees by means of a suitable script realized in MatLab. The detailed assessment of the structural behaviour of vessels has been investigated with a refined model developed in ABAQUS.

As previously described, almost all the facilities are characterized by a rather simple structural configuration. Thus, is also simple to detect the structural elements that are mainly involved in the collapse of the facility.

The adopted assumptions are reported in the following. Where is not explicitly said, the numerical models used for the refined and simplified analyses are the same.

Water treatment unit. All the RC supporting structures are modelled with a lumped plasticity approach, using the Ibarra Medina Krawinkler hysteretic law, for columns and beams as well. The shear failure is not specifically modelled, but assessed *a posteriori*, during the post-process of the results. It is assumed that when the maximum shear capacity is achieved, the collapse is triggered. Such models are adopted, being rather simple and easy to imple-

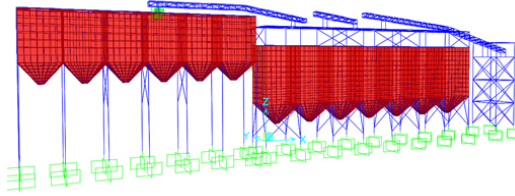


Figure A.1: detailed modelling of the additional alloys unit.

ment in the numerical computation, are adopted for both the simplified and the detailed analyses.

For what concern the sand filters, two different models are used. For the detailed analyses, an ABAQUS model has been adopted, in order to correctly catch all the possible mechanisms of collapse.

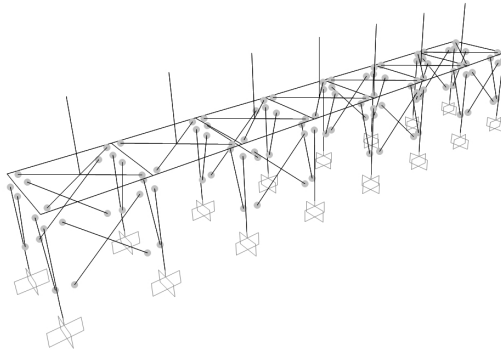
Additional alloys. For what concern the numerical modelling of the additional alloys, the main issue is associated with the assessment of the coupling effects provided by the belt conveyors. To this aim, two different modeling approaches have been performed: (i) the first considers all the facilities coupled by the belt conveyors (see figure A.1); (ii) the second is represented by the modelling of each facility as standalone. In both the cases, the numerical models, developed in SAP2000, explicitly considered the supporting structure and the steel plates of the silos as well. The seismic masses and the weight of the silos have been distributed along the walls of the silos.

Comparing the results of the modal analysis performed in the two different conditions, it has been noted that the fundamental periods of each facility does not substantially changes, as shown in table A.1. Therefore it can be assumed that the belt conveyors does not provide any coupling effects and the facilities can be modelled as standalone.

It should be also noted that the modelling of the supported steel structure of the silos, does not provide any further improvement in the results. In fact, as it can be observed by past earthquake scenarios, the damage concentrates within the supported structure, while the supported elements remain undamaged until the possible collapse. For that reason, the numerical models of the silos have been simplified, focusing only on the superstructures, while the supported silos are modelled as a simple elastic beams, with an distributed weight and mass equivalent to the effective value of the silos, with an equivalent stiffness. Such elastic element is connected to the supporting structure by means of rigid elements, as schematically shown in figure A.2.

Table A.1: comparison of the coupled versus the uncoupled modelling approach.

Silo 1			
Mode	Coupled m.	Uncoupled m.	Difference
T1 (translational X)	0.813 s	0.817 s	0.49%
T2 (translational Y)	0.682 s	0.687 s	0.73%
T3 (torsional)	0.453 s	0.440 s	2.95%
Silo 2			
Mode	Coupled m.	Uncoupled m.	Difference
T1 (translational X)	3.94 s	3.95 s	0.25%
T2 (translational Y)	2.836 s	2.841 s	0.18%
T3 (torsional)	2.391 s	2.401 s	0.42%

**Figure A.2:** simplified modelling of the silos of the additional alloys unit.

Following the approach proposed by Uriz (2008), all the braces, belonging to the silo and supporting tower as well, are modelled by means of force-beam fiber elements (REF) with an initial imperfection equal to $1/200$ of the length of the element (EN 1993-1-1:2005, 2005). The detailed modelling of the belt conveyors considered all the structural elements (see figure), while the simplified analyses are conducted considering a simplified elastic-perfectly plastic behaviour of the columns, supposed pinned-fixed at the ends. With such a simplification, simple hand calculations can be easily performed.

Main building. The main building has a structural configuration that makes possible to consider separately the behaviour of the two principal directions. Hence, two separate 2D numerical models have been realized. It has been considered for each direction the worst load configuration: for the X direction (portal frame), all the cranes are considered aligned and fully loaded; for the Y direction, it has been considered that all the cranes are loaded. For this configuration, the prescription of the Eurocode 1 (EN 1991, 1991) about the combination of loads are neglected, in order to provide results on the safe side. Such decision is led by the absence of enough informations about the loading protocol of the cranes during the fully working condition of the steel mill.

Nitroge-argon vessels. The particular configuration of the nitrogen argon vessels requires two different models: one for the detailed analyses, able to catch all the possible collapse mechanism and realized in ABAQUS, and one simplified, with which is supposed to know *a priori* the effective collapse mechanism.

A.1.2 Results of the vulnerability assessment

The results of the vulnerability assessment on the Steel Mill are detailed in the following. For each facility, the results obtained from the analyses performed with the mean and the characteristic mechanical properties are reported. The first one is used for the detailed assessment and the second and third level of the multilevel procedure, while the second one is adopted to perform the analyses foreseen at the first level of the proposed multilevel procedure.

Table A.2 summarizes the variable loads, the seismic masses, and the associated ψ_2 factors used for the seismic vulnerability assessment of all the facilities.

for the sake of clarity, the results of the vulnerability assessment, provided in the following paragraphs, are shown in a tabular format.

Table A.2: masses considered for the seismic vulnerability assessment.

Facility	Mass [Ton]
Main building	360.00
Silo 1	1235.00
Silo 2	1240.00
Support tower	80.00
BC 1	-
BC 2	-
Dust filters	1340.00
Mud Container	100.00
Support 1	96.00
Support 2	193.50
Sand filters	175.00
Nitrogen- Argon	15

A.1.3 Water treatment unit

A.1.3.1 Mud container

The mud container structure is composed of two parts, that can be considered as in series element: the RC support structure and the mud container itself. Those two parts can be considered as in series elements and, therefore, they share the same force.

A detailed vulnerability assessment is performed executing analyses on two numerical models: one representing the RC super-structure and one representing the behaviour of the mud container.

Supporting structure. The supporting structure, as above-mentioned, has been built to withstand vertical load and an horizontal wind action.

The symmetric structural configuration leads to the same capacity curve in both the directions. Since the beams result strongly over-resistant than the columns, the plastic deformations are expected within the columns. Thus, 2 plastic hinges, which behaviour is represented through the modified Ibarra-Medina-Krawinkler model, are applied at ends of the columns. The mud container is modelled with an equivalent elastic element, characterized by the same stiffness of the silos, with a vertical load applied at 2/3 of its height. Beams are modelled as elastic elements with a uniform distributed load, rep-

representative of the maintenance condition, as suggested in the original design report.

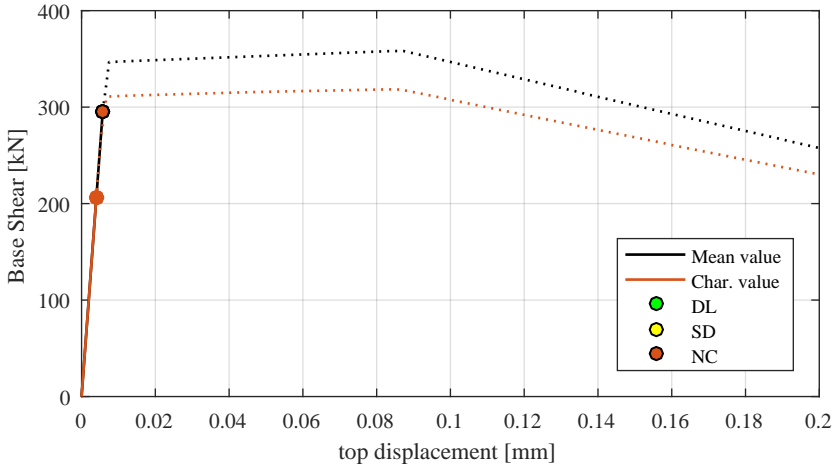


Figure A.3: capacity curves of the mud container.

The support structure reaches the collapse due to shear failure of the columns. Equations (A.1) and (A.2) evidence that the capacity design approach is not applied. Being the failure is attained in the elastic branch, it is assumed that the collapse is achieved when the shear failure is reached. Since the shear failure is attained in the elastic branch, it is assumed that all the three limit states are achieved at the same time, as shown in figure A.3.

$$V_{Rd} = \min \begin{cases} f_{ym} \cdot d \frac{A_s}{s} \cot \theta & = 64.83 \text{ kN} \\ b_w \cdot d \cdot v \cdot f_{cd} \frac{\cot \theta}{1 + \cot^2 \theta} & = 138 \text{ kN} \end{cases} \quad \cot \theta = 2.5 \quad (\text{A.1})$$

$$V_{Ed} = \frac{2 \cdot M_{max}}{H} = \frac{2 \cdot 183.6}{5} = 73.44 \text{ kN} \quad (\text{A.2})$$

The vulnerability index I_v and the parameters to build the fragility curves are given in table A.3.

Mud container. The analysis of the mud container has been performed with ABAQUS in order to carefully assess the stress distributions on the four an-

Table A.3: Vulnerability indexes for the superstructure of the mud container, assuming a fundamental period T_1 equal to 0.297 s (the structure is symmetric, therefore the fundamental period along the two principal directions is equal).

Detailed assessment				
	Mechanism	$S_a(T_1)^*$	β	I_v
DL	Shear failure	0.21	0.09	-
SD	Shear failure	0.21	0.09	-
NC	Shear failure	0.21	0.09	0.24
Simplified analysis				
	Mechanism	$S_a(T_1)^*$	β	I_v
DL	Shear failure	0.17	0.10	-
SD	Shear failure	0.17	0.10	-
0.19	Shear failure	0.17	0.10	0.19
NC				

* $S_a(T_1) = \exp(\eta)$

chor legs (HEA500) of the mud container. In particular, the legs are modelled as pinned to the beams of the superstructure. The mud container evidenced a strong over-resistance of the legs, reaching the plastic deformation strongly after the shear failure of the columns of the RC superstructure. Figure A.5 shows the details of the connection and the plastic equivalent strains (PEEQ) distribution suddenly after the peak elastic displacement.

It can be concluded that the supporting structure rules the whole structural behaviour, being the columns characterized by an inadequate shear strength.

A.1.3.2 Support 1 and support 2

The structural configuration of the support 1 and support 2 is very similar to the mud container. In fact, also in this case, the structural deficiencies are concentrated within the column, which failure is governed by shear collapse in the elastic field. Shear capacity and shear demand is equal to the ones calculated in equations (A.1) and (A.2).

The parameters to build the fragility curves and the vulnerability indexes are reported in table A.4 for the support 1 and in table A.5 for the support 2.

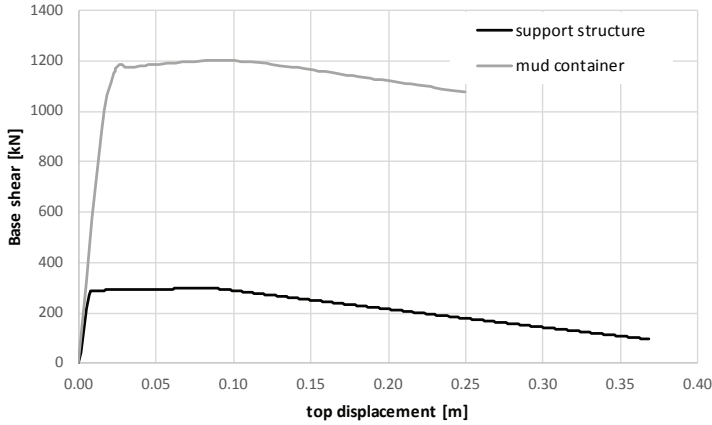


Figure A.4: capacity curves comparison of the mud container and the support structure: the sudden stiffness and resistance change at the elastic limit of the capacity curve of the mud container is due to a local instability of the compressed flange of the supporting leg.

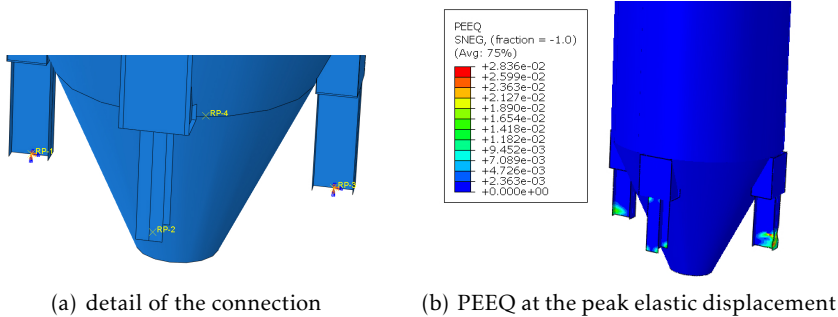


Figure A.5: ABAQUS model and results at the plastic deformation at the elastic limit.

A.1.3.3 Sand Filters

The sand filter has been modelled in ABAQUS, detailing all the parts of the filter using 4 node shell elements, associated with a constitutive law of the material that account for the elasto plastic behaviour of the steel. In particular, basing on the mean values provided in table 5.1, the Hollomon law has been used to calibrate the post-elastic branch of the material.

The main issue that strongly influence the vulnerability of the system is the clear lack of anchor bolts that fix the sand filter to the ground. In such a configuration, the collapse of the filter is considered achieved when the hor-

Table A.4: Vulnerability indexes for the support 1, assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ equal to 0.28 s.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.53	0.07	-
SD	Shear failure	0.53	0.07	-
NC	Shear failure	0.53	0.07	0.61
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Shear failure	0.63	0.13	-
SD	Shear failure	0.63	0.13	-
NC	Shear failure	0.63	0.13	0.73
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.44	0.07	-
SD	Shear failure	0.44	0.07	-
NC	Shear failure	0.44	0.07	0.50
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Shear failure	0.49	0.10	-
SD	Shear failure	0.49	0.10	-
NC	Shear failure	0.49	0.10	0.56

$$^*S_a(T_1) = \exp(\eta)$$

horizontal action induced by earthquake exceeds the steel-to-concrete friction, assumed equal to $0.3 \cdot W$, being W the total vertical load. Due to the particular over-strength of sand filters supports, the sliding collapse is attained in the elastic field, as it is clearly observable in figure A.6. Table A.6 reports the vulnerability indexes and the parameters required to build the fragility curves, assuming a lognormal shape.

Table A.5: Vulnerability indexes for the support 2, assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 0.31 s and 0.34 s.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.27	0.00	-
SD	Shear failure	0.27	0.00	-
NC	Shear failure	0.27	0.00	0.32
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Shear failure	0.49	0.08	-
SD	Shear failure	0.49	0.08	-
NC	Shear failure	0.49	0.08	0.58
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.23	0.00	-
SD	Shear failure	0.23	0.00	-
NC	Shear failure	0.23	0.00	0.27
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Shear failure	0.35	0.08	-
SD	Shear failure	0.35	0.08	-
NC	Shear failure	0.35	0.08	0.42

$$*S_a(T_1) = \exp(\eta)$$

A.1.4 Additional alloys

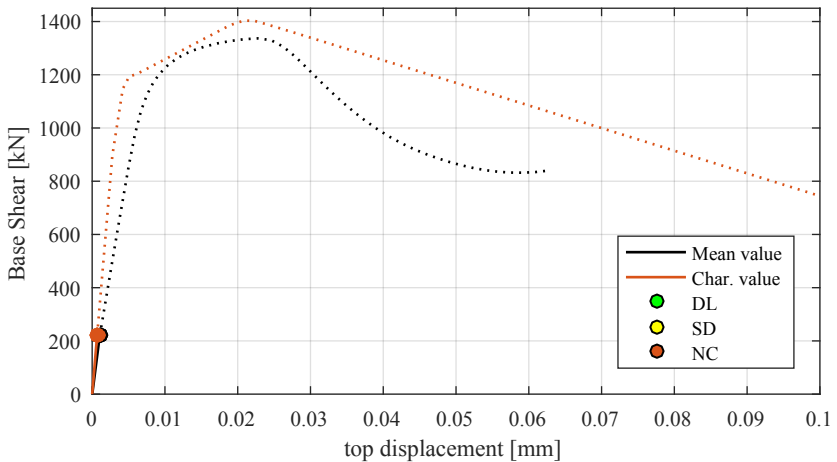
A.1.4.1 Silo 1

As above-mentioned, the silos 1 has been designed to withstand only vertical loads. The particular height of the silo (9.82 m) and its particular heavy weight, make this structure strongly sensitive to the $P - \Delta$ effects, reaching the collapse at very low level of seismic action. Such behaviour is clearly depicted by the shape of the capacity curves (see figure A.7), which exhibit a strong non-linear behaviour just at few levels of displacement.

Table A.6: Vulnerability indexes for the sand filters, assuming a fundamental period T_1 *s.

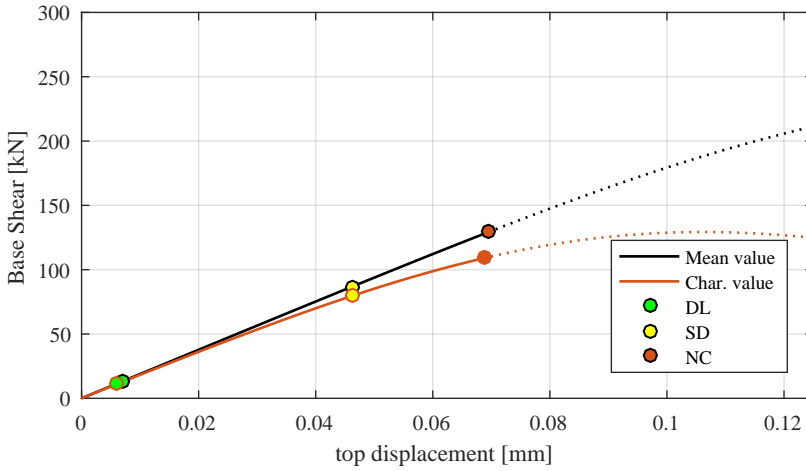
Detailed assessment				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.08	0.0	-
SD	Shear failure	0.08	0.0	-
NC	Shear failure	0.08	0.0	0.1
Simplified assessment				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Shear failure	0.08	0.0	-
SD	Shear failure	0.08	0.0	-
NC	Shear failure	0.08	0.0	0.1

$$*S_a(T_1) = \exp(\eta)$$

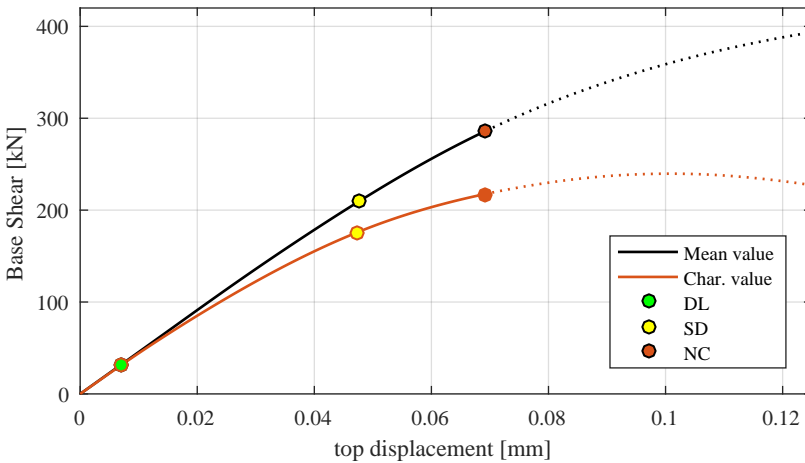
**Figure A.6:** capacity curves and related limit states of the sand filters.

Along the X direction, the rotational capacity of the columns is checked as principal limit states, followed by the control of the θ factor, that account for the sensitivity to the second order effects, which, formulation, according to EC8, is reported in the following:

$$\theta = \frac{P \cdot d_r}{V \cdot H} \quad (\text{A.3})$$



(a) X direction



(b) Y direction

Figure A.7: capacity curves of the silo 1 (additional alloys unit) along the X and Y direction.

where P represents the vertical load, d_r the horizontal displacement of the silos, V the base shear and H the interstorey height (9.82 m). In both the directions, the $P - \Delta$ effects rule over the other collapse mechanisms. In fact, the θ factor exceed 0.3 at very low base shear.

Table A.7 report the vulnerability index along both the directions and the parameters required for the definition of the fragility curves.

A.1.4.2 Silo 2

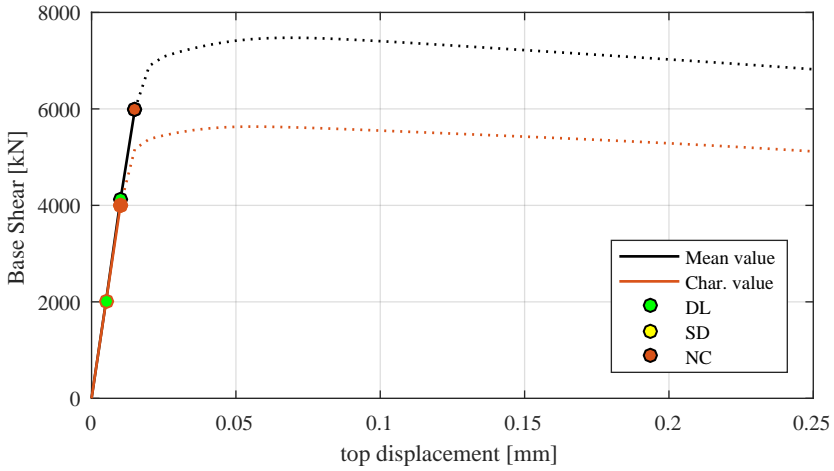
The silo 2 is characterized in both the directions by a X bracings system. The main vulnerability source relies, in both the directions, on the capacity of the connections, resulting lower than the tensile capacity of the braces. In particular, the plastic strain capacity is the weak link of the connection.

For that reason, the structure cannot develop a plastic mechanism, failing in the elastic branch, as shown in figure A.8.

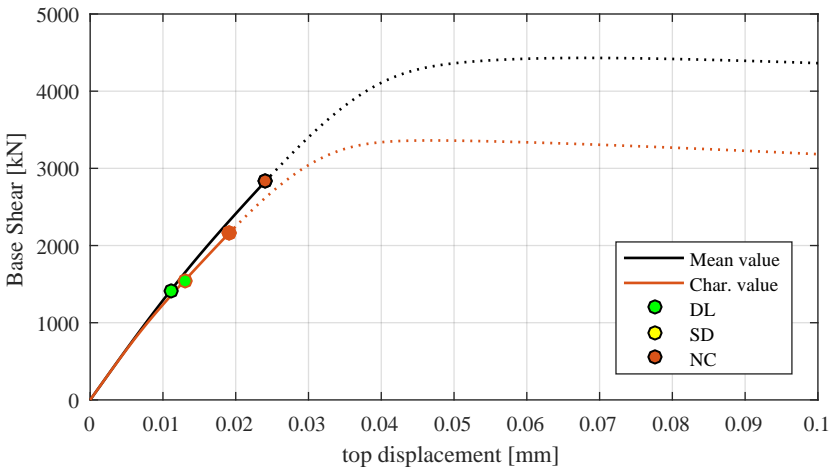
Considering the EC8-3 (EN 1998-3, 2005) limitation for tensile braces (see table A.8), table A.9 reports the vulnerability indexes and the parameters used for the estimation of the fragility curves.

A.1.4.3 Supporting tower

The vulnerability assessment is ruled by the tensile behaviour of the braces in both the principal directions. The capacity curve is graphically described in figure ???. Since no informations are available about the connections, it has been assumed that the structure is capable to develop a plastic mechanism, governed by the nonlinear behaviour of the X braces. Table A.10 summarizes the reference limit states and table A.11 reports the vulnerability indexes and the parameters required for the construction of the fragility curves.



(a) X direction



(b) Y direction

Figure A.8: capacity curves of the silo 1 (additional alloys unit) along the X and Y direction.

Table A.7: Vulnerability indexes silo 1 (additional alloys unit), assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 3.96 s and 4.00 s.

Detailed Assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Brace buckling	0.01	0.0	-
SD	Connection failure	0.02	0.0	-
NC	Connection failure	0.03	0.02	0.02
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Brace buckling	0.02	0.0	-
SD	Connection failure	0.17	0.0	-
NC	Connection failure	0.27	0.09	0.27
Simplified Assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$	β	I_v
DL	Brace buckling	0.01	0.10	-
SD	Connection failure	0.02	0.10	-
NC	Connection failure	0.03	0.10	0.02
Y direction				
	Mechanism	$S_a(T_{1,y})^*$	β	I_v
DL	Brace buckling	0.02	0.10	-
SD	Connection failure	0.17	0.10	-
NC	Connection failure	0.27	0.10	0.27

$$*S_a(T_1) = \exp(\eta)$$

Table A.8: limit states for the braces in tension according to EN 1998-3 (2005)

Limit States		
DL	SD	NC
$0.25 \cdot \Delta_t$	$7 \cdot \Delta_t$	$9 \cdot \Delta_t$

Table A.9: Vulnerability indexes silo 2 (additional alloys unit), assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 0.35 s and 0.61 s.

X direction				
Detailed Assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.33	0.09	-
SD	Connection failure	0.49	0.09	-
NC	Connection failure	0.49	0.09	0.59
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.04	0.0	-
SD	Connection failure	0.22	0.0	-
NC	Connection failure	0.22	0.0	0.36
Simplified Assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.16	0.09	-
SD	Connection failure	0.33	0.09	-
NC	Connection failure	0.33	0.09	0.39
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.03	0.0	-
SD	Connection failure	0.20	0.0	-
NC	Connection failure	0.20	0.0	0.31

$$*S_a(T_1) = \exp(\eta)$$

Table A.10: limit states for the braces in tension according to EN 1998-3 (2005)

Limit States		
DL	SD	NC
$0.25 \cdot \Delta_t$	$7 \cdot \Delta_t$	$9 \cdot \Delta_t$

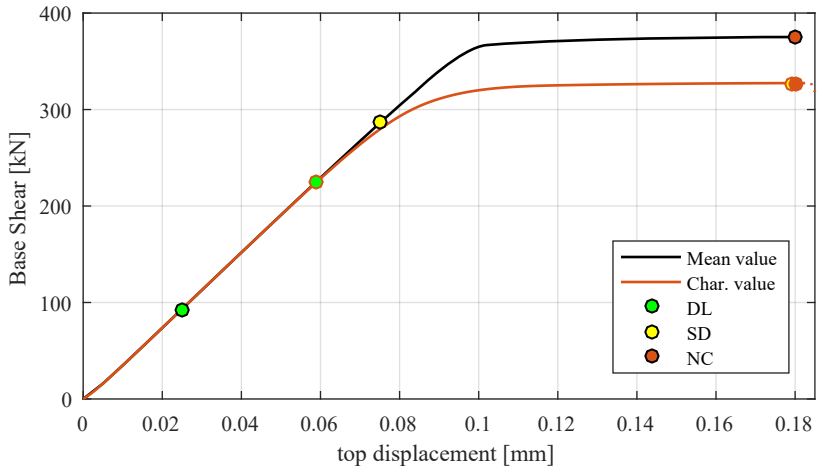


Figure A.9: capacity curve of the supporting tower.

Table A.11: Vulnerability indexes of the supporting tower (additional alloys unit), assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 0.97 s and 1.51 s.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.28	0.0	-
SD	Brace plastic deformation	0.86	0.20	-
NC	Brace plastic deformation	0.88	0.21	2.36
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.1	0.16	-
SD	Brace plastic deformation	0.38	0.17	-
NC	Brace plastic deformation	0.51	0.21	2.33
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.28	0.0	-
SD	Brace plastic deformation	0.50	0.20	-
NC	Brace plastic deformation	0.50	0.21	2.06
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.1	0.16	-
SD	Brace plastic deformation	0.38	0.17	-
NC	Brace plastic deformation	0.47	0.21	2.27

$$*S_a(T_1) = \exp(\eta)$$

A.1.4.4 Belt conveyors

The vulnerability assessment is performed on a local model representative of the belt conveyor itself. The only source of vulnerability is represented by the supporting columns (HEA100 for BC1 and UPN100 for the BC2). The analysis and the substructuring is performed on the basis of the following considerations:

- the structural behaviour of the BC1 is ruled by the silo 1. In fact, the silo 1 is more vulnerable than the silo 2. Therefore, for lower level of seismic action, the top displacement to which the silo 1 is subjected to is greater than the one on the silo 2.
- The collapse of BC1 is considered as the minimum among the NC limit state for silo 1, silo 2 and BC1.
- For the same reason the structural behaviour of the BC2 is ruled by the silo 2, instead by the supporting tower.
- The collapse of BC2 is considered as the minimum among the NC limit state for silo 2, supporting tower and BC2.

The capacity curves and the position of the limit states are reported in figures A.10 and A.11. For the BC1 the capacity curve of silo 1 is adopted, while for the BC2 the capacity curve of the silo 2 is followed. Tables A.12 and A.13 summarize the reference limit states along both the directions for BC 1 and BC 2.

Table A.12: Vulnerability indexes for the BC1 (additional alloys unit).

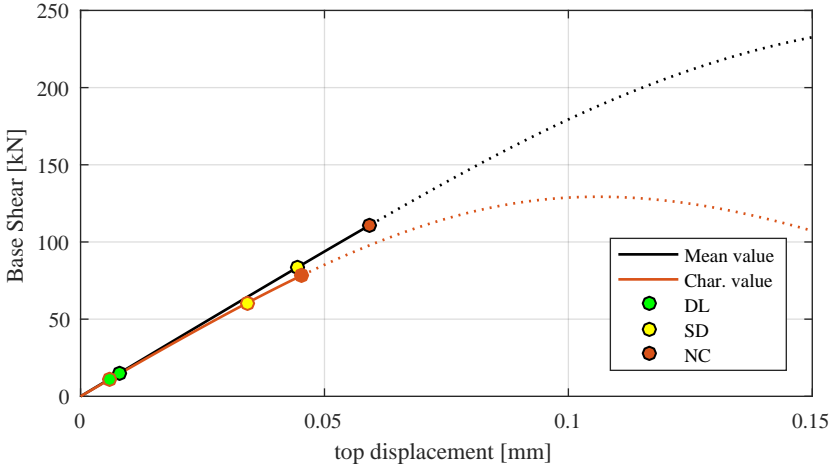
Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.01	0.10	-
SD	Brace plastic deformation	0.01	0.10	-
NC	Brace plastic deformation	0.02	0.10	0.03
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.01	0.11	-
SD	Brace plastic deformation	0.03	0.11	-
NC	Brace plastic deformation	0.03	0.12	0.34
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.01	0.00	-
SD	Brace plastic deformation	0.01	0.01	-
NC	Brace plastic deformation	0.01	0.07	0.02
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.01	0.00	-
SD	Brace plastic deformation	0.02	0.01	-
NC	Brace plastic deformation	0.02	0.07	0.24

$$*S_a(T_1) = \exp(\eta)$$

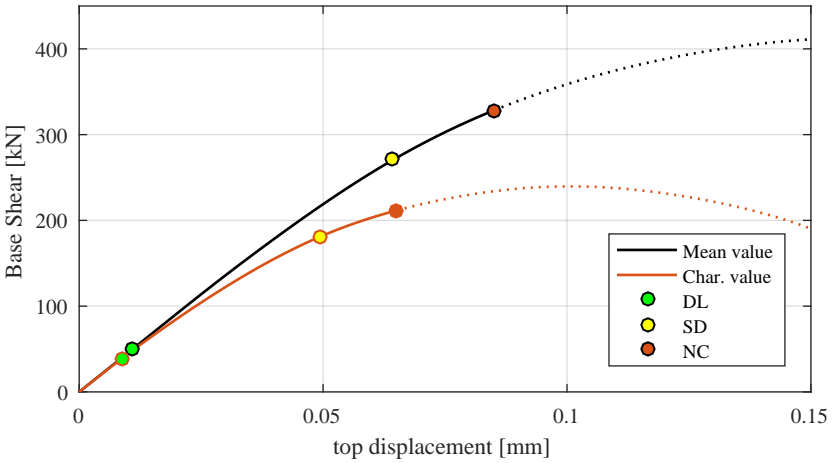
Table A.13: Vulnerability indexes for the BC2 (additional alloys unit).

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.24	0.09	-
SD	Brace plastic deformation	1.27	0.20	-
NC	Brace plastic deformation	1.56	0.26	1.85
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.08	0.14	-
SD	Brace plastic deformation	0.48	0.16	-
NC	Brace plastic deformation	0.63	0.20	1.04
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.18	0.01	-
SD	Brace plastic deformation	0.95	0.17	-
NC	Brace plastic deformation	1.18	0.24	1.40
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.06	0.01	
SD	Brace plastic deformation	0.36	0.07	-
NC	Brace plastic deformation	0.49	0.14	0.79

* $S_a(T_1) = \exp(\eta)$

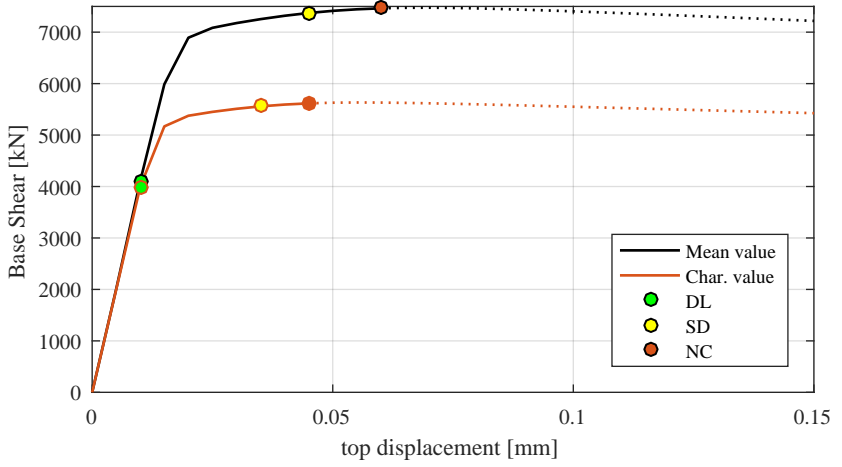


(a) X direction

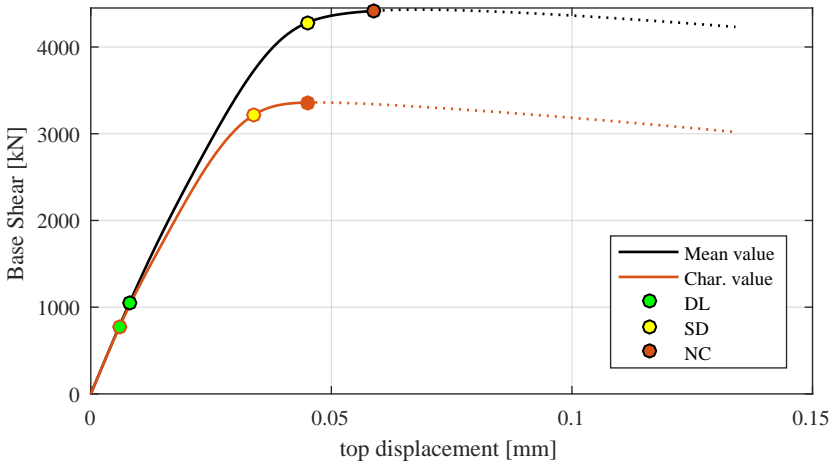


(b) Y direction

Figure A.10: capacity curves of the BC 1 (additional alloys unit) along the X and Y direction.



(a) X direction



(b) Y direction

Figure A.11: capacity curves of the BC 2 (additional alloys unit) along the X and Y direction.

A.1.5 Nitrogen argon vessels

The detailed vulnerability assessment is performed on the nitrogen-argon vessels and on the pipelines, considering a fully decoupled structural behaviour.

The detailed assessment on the pipelines is performed considering two conditions: the empty condition and the in pressure situation. A displacement is applied in correspondence of the vessel-to-pipe connection.

Due to the significant slenderness of the pipes and the absence of intermediate restraints, the collapse of the vessels is attained before significant plastic deformations of the pipes that might compromise their functionality¹. The plastic deformations are achieved on the elbows, representing a critical component that require detailed assessment, as evidenced by numerous scientific researches.

The collapse mechanisms of the vessels is ruled by the plastic deformation of the columns; furthermore, the overturning moment generated by the seismic action induce also a punching mechanism, localized in correspondence of the compressed column.

The simplified assessment focuses only on the vessels, neglecting the presence of the pipes system.

The results of the vulnerability assessment are shown in table A.14.

¹ According to the EN1998-1:4, two main limit states are considered: the DL, that is attained in correspondence of a displacement two times higher than the yielding displacement (the functionality is not compromised); the NC, attained when the pipes reach the ultimate plastic deformation.

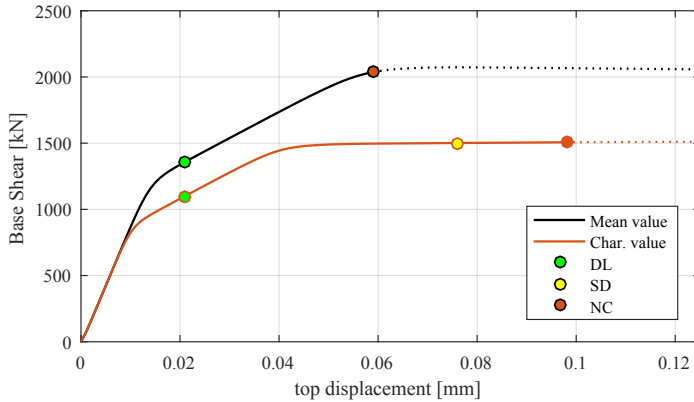
Table A.14: Vulnerability indexes for the nitrogen argon vessels.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.27	0.09	-
SD	Brace plastic deformation	1.46	0.20	-
NC	Brace plastic deformation	1.98	0.25	1.95
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.17	0.14	-
SD	Brace plastic deformation	0.42	0.16	-
NC	Brace plastic deformation	0.96	0.20	1.34
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.18	0.01	-
SD	Brace plastic deformation	1.20	0.17	-
NC	Brace plastic deformation	1.48	0.22	1.55
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.12	0.01	-
SD	Brace plastic deformation	0.36	0.07	-
NC	Brace plastic deformation	0.76	0.14	1.12

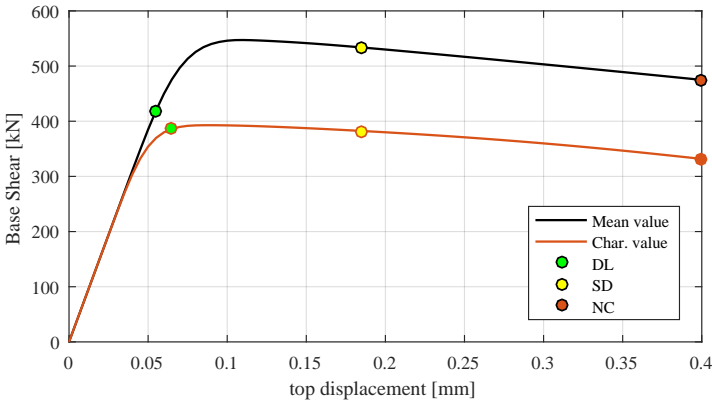
* $S_a(T_1) = \exp(\eta)$

A.1.5.1 Dust filter

The vulnerability assessment of the dust filter is mainly ruled by the particular structural configuration. In fact, the Y direction is characterized by an MRF system, while the X direction has a X bracing system; it should be noted that the strong axis of the columns is rotated on the X direction. The capacity curves are shown in figure A.12. Such a decision, probably aimed at simplify the column-to-braces welding procedures, strongly weakens the Y direction. The results of the vulnerability assessment are reported in table A.15.



(a) X direction



(b) Y direction

Figure A.12: capacity curves of the dust filter (additional alloys unit) along the X and Y direction.

Table A.15: Vulnerability indexes of the dust filters, assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 0.85 s and 0.83 s.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.02	0.15	-
SD	Brace plastic deformation	0.36	0.27	-
NC	Brace plastic deformation	0.47	0.32	1.05
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.12	0.00	-
SD	Brace plastic deformation	0.36	0.06	-
NC	Brace plastic deformation	0.49	0.14	1.09
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	0.02	0.05	-
SD	Brace plastic deformation	0.33	0.33	-
NC	Brace plastic deformation	0.43	0.34	0.98
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.12	0.00	-
SD	Brace plastic deformation	0.30	0.06	-
NC	Brace plastic deformation	0.42	0.14	1.05

$$*S_a(T_1) = \exp(\eta)$$

A.1.5.2 Main building

Along the X direction, being the columns behaving as cantilever element, the main limit states to account for are the represented by the plastic rotation at the base of the column. On the contrary, along the Y direction, the structural behaviour is governed by the plastic deformation of the braces under tension. Table A.16 reports the results of the seismic vulnerability assessment.

Table A.16: Vulnerability indexes of the dust filters, assuming a fundamental period $T_{1,x}^*$ and $T_{1,y}^*$ respectively equal to 0.36 s and 0.80 s.

Detailed assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	2.03	0.24	-
SD	Brace plastic deformation	5.72	0.28	-
NC	Brace plastic deformation	7.29	0.32	8.62
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.27	0.12	-
SD	Brace plastic deformation	0.80	0.16	-
NC	Brace plastic deformation	1.07	0.23	2.29
Simplified assessment				
X direction				
	Mechanism	$S_a(T_{1,x})^*$ [g]	β	I_v
DL	Brace buckling	2.02	0.01	-
SD	Brace plastic deformation	5.46	0.22	-
NC	Brace plastic deformation	6.88	0.28	8.15
Y direction				
	Mechanism	$S_a(T_{1,y})^*$ [g]	β	I_v
DL	Brace buckling	0.27	0.00	-
SD	Brace plastic deformation	0.70	0.16	-
NC	Brace plastic deformation	0.95	0.21	2.02

$$*S_a(T_1) = \exp(\eta)$$

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