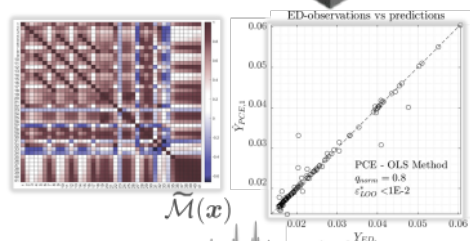
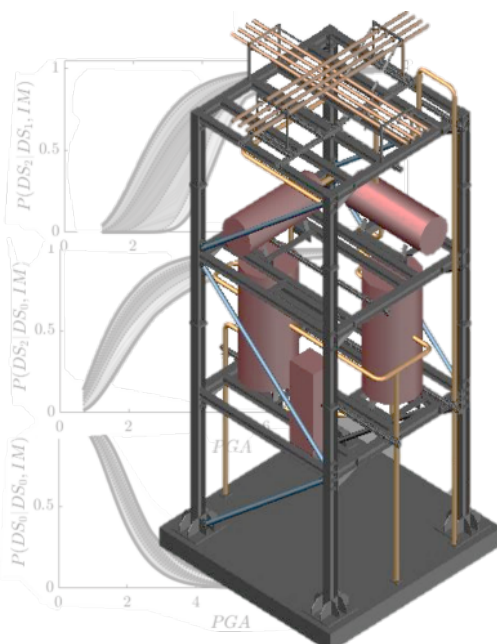


Curriculum 3. Modelling and Simulation

Chiara Nardin

Seismic experimental analyses and surrogate models of multi-component systems in special-risk industrial facilities



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**UNIVERSITY
OF TRENTO**

**Doctoral School in Civil, Environmental
and Mechanical Engineering**

Curriculum *Modelling and Simulation*

XXXV cycle - 2019-2022

Doctoral Thesis - December 2022

**Seismic experimental analyses and surrogate models
of multi-component systems in special-risk industrial facilities**

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*If you want to find the secrets of the universe, think in terms of energy,
frequency and vibration.*

*Se vuoi scoprire i segreti dell'universo, pensa in termini di energia,
frequenza e vibrazione.*

Nikola Tesla

Acknowledgements

I want to seize this moment to thank all who shared with me and made it possible to complete this PhD journey: if I leave out some names here, I certainly will not forget my memories of our experiences together.

First of all, I would like to thank my Supervisor and Professor Oreste S. Bursi, for his guidance in this up-to-date learning experience and for sharing invaluable work and life-related lessons with me.

A special thanks to Marco Broccardo for his advice, talks and sharing of experiences and ideas on research. Thank you for letting me discover the amazing world of ETH and the RSUQ group led by Professor Bruno Sudret and Stefano Marelli. I got to know a dynamic environment full of continuous interactions, built up of unique and incredible people, among whom undoubtedly Stefano, Nora, Katerina, Anderson and the whole group.

I would also like to thank UNITN: from fellow students first, to colleagues and dear friends later, a special mention goes to Giulia, Roberto, Luca, Patrick and many others without whom I would not have achieved so many goals.

A due thank you to my new and lifelong friends, Maria, Alison, Andrea, Daniele, Margherita, Arianna and all, who have supported and sustained me on this journey...even with massive 5kg each for 600km! I am lucky to have you as my friends.

Last but not least, a grateful thank you to my family: without you, none of this would have been possible. Thank you for being there at all times, thank you for the values with which you have raised me and for the continuous care and unconditional love that you donate. A special memory goes out to my grandparents, Alba and Luigi, AnnaMaria and Renzo, to whom I dedicate this thesis: thank you for always being with me.

Chiara

Ringraziamenti

Vorrei cogliere questo momento per ringraziare tutti coloro che mi hanno permesso di portare a termine e hanno condiviso con me questo percorso di Dottorato: se qualche nome qui tralascio, di sicuro lo porto con me nei ricordi delle esperienze vissute insieme.

Innanzitutto desidero ringraziare il mio relatore e Professore Oreste S. Bursi, per avermi guidato in questa esperienza formativa e aver condiviso con me preziosi insegnamenti, sia lavorativi che di vita. Un grazie speciale a Marco Broccardo, per i suoi consigli, le chiacchierate ed aver condiviso scambi ed idee sulla ricerca. Grazie, per avermi consentito di conoscere il meraviglioso mondo di ETH e del gruppo RSUQ guidato dal Professor Bruno Sudret e Stefano Marelli. Ho avuto modo di conoscere una realtà dinamica e ricca di continui scambi, fatta di persone speciali ed incredibili, fra cui indubbiamente Stefano, Nora, Katerina, Anderson e tutto il gruppo.

Non da meno, desidero ringraziare anche la realtà Unitn: da compagni di studio prima, a colleghi e cari amici poi, una menzione speciale la trovano Giulia, Roberto, Luca, Patrick e tutti coloro senza i quali non avrei raggiunto tanti obiettivi.

Un ringraziamento doveroso agli amici nuovi e di sempre, Maria, Alison, Andrea, Daniele, Margherita, Arianna e tutti, che mi hanno sostenuto e supportato in questo viaggio..portando addirittura ben 5kg a testa per 600km! Sono fortunata ad avervi come amici.

Infine, non da ultimo, un doveroso grazie alla mia famiglia: senza di voi, nulla di tutto questo sarebbe stato mai possibile. Grazie per esserci in qualsiasi momento, grazie per i valori con cui mi avete cresciuta e per il continuo affetto e amore incondizionato che donate. Un pensiero speciale lo rivolgo ai miei nonni, Alba e Luigi, AnnaMaria e Renzo a cui dedico questo lavoro: grazie per essere sempre con me.

Chiara

Abstract

Nowadays, earthquakes are one of the most catastrophic natural events that have a significant human, socio-economic and environmental impact. Besides, based on both observations of damage following recent major/moderate seismic events and numerical/experimental studies, it clearly emerges that critical non-structural components (NSCs) that are ubiquitous to most industrial facilities are particularly and even disproportionately vulnerable to those events. Nonetheless and despite their great importance, seismic provisions for industrial facilities and their process equipment are still based on the classical load-and-resistance factor design (LRFD) approach; a performance-based earthquake engineering (PBEE) approach should, instead, be preferred. Along this vein, in recent years, much research has been devoted to setting computational fragility frameworks for special-risk industrial components and structures. However, within a PBEE perspective, studies have clearly remarked: i) a lack of definition of performance objectives for NSCs; ii) the need for fully comprehensive testing campaigns data on coupling effects between main structures and NSCs.

In this respect, this doctorate thesis introduces a computational framework for an efficient and accurate seismic state-dependent fragility analysis; it is based on a combination of data acquired from an extensive experimental shake table test campaign on a full-scale prototype industrial steel frame structure and the most recent surrogate-based UQ forward analysis advancements.

Specifically, the framework is applied to a real-world application consisting of seismic shake table tests of a representative industrial multi-storey frame structure equipped with complex process components, carried out at the EUCENTRE facility in Italy, within the European SPIF project: Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities.

The results of this experimental research campaign also aspire to improve the understanding of these complex systems and improve the knowledge of FE modelling techniques. The main goals aim to reduce the huge computational burden and to assess, as well, when the importance of coupling effects between NSCs and the main structure comes into play. Insights provided by innovative monitoring systems were then deployed to develop and validate numerical and analytical models.

At the same time, the adoption of Der Kiureghian's stochastic site-based ground motion model (GMM) was deemed necessary to severely excite the process equipment and supplement the scarcity of real records

with a specific frequency content capable of enhancing coupling effects.

Finally, to assess the seismic risk of NSCs of those special facilities, this thesis introduces state-dependent fragility curves that consider the accumulation of damage effects due to sequential seismic events. To this end, the computational burden was alleviated by adopting polynomial chaos expansion (PCE) surrogate models. More precisely, the dimensionality of a seismic input random vector has been reduced by performing the principal component analysis (PCA) on the experimental realizations. Successively, by bootstrapping on the experimental design, separate PCE coefficients have been determined, yielding a full response sample at each point. Eventually, empirical state-dependent fragility curves were derived.

Publications

As a result of the work conducted in this thesis, the following publications have been produced:

Journal Publications:

- J-1** Paolacci, F., Quinci, G., **C. Nardin**, Vezzari, V., Marino, A., Ciucci, M., (2021). Bolted Flange Joints equipped with FBG sensors in industrial piping Systems subject to seismic loads. *Journal of Loss Prevention in the Process Industries*. doi.org/10.1016/j.jlp.2021.104576
- J-2** C. Butenweg, O.S. Bursi, F. Paolacci, I. Lanese, M. Marinkovic, **C. Nardin**, G. Quinci (2021). Seismic performance of an industrial multi-storey frame structure with process equipment subjected to shake table testing. *Engineering Structure*. doi.org/10.1016/j.engstruct.2021.112681
- J-3** **C. Nardin**, O.S. Bursi, F. Paolacci, A. Pavese, G. Quinci (2022). Experimental performance of a multi-storey braced frame structure with non-structural industrial components subjected to synthetic ground motions. *Earthquake Engineering & Structural Dynamics*. doi.org/10.1002/eqe.3656
- J-4** G. Quinci, **C. Nardin**, F. Paolacci, O.S. Bursi (2022). Modelling issues in seismic risk analysis of Non-Structural Components installed on industrial plant subsystems. *Bullettin of Earthquake Engineering*. (under submission)
- J-5** **C. Nardin**, S. Marelli, O.S. Bursi (2023). Meta-model state-dependent demand fragility curves for non-structural components in special risk industrial facilities. *Earthquake Engineering & Structural Dynamics*. (in progress)

Conferences:

- C-1** **C. Nardin**, R. di Filippo, R. Endrizzi, I. Lanese, F. Paolacci, O.S. Bursi (2020). Ground Motion Model For Seismic Vulnerability Assessment Of Portotype Industrial Plants. *Proceedings of the ASME 2020 Pressure Vessels & Piping Conference PVP2020*, July 19-24 2020 Minneapolis, Minnesota, USA.

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- C-2** C. Butenweg, O.S. Bursi, **C. Nardin**, I. Lanese, A. Pavese, M. Marinkovic, F. Paolacci, G. Quinci (2020). Simulated Investigation On The Seismic Performance Of A Multi-Component System For Major-Hazard Industrial Facilities. *Proceedings of the ASME 2020 Pressure Vessels & Piping Conference PVP2020*, July 19-24 2020 Minneapolis, Minnesota, USA.
- C-3** C. Butenweg, M. Marinkovic, A. Pavese, I. Lanese, B. Hoffmeister, M. Pinkawa, C. Vulcu, O. S. Bursi, **C. Nardin**, F. Paolacci, G. Quinci, M. Fragiadakis, F. Weber, P. Huber, P. Renault, M. Gündel, S. Dyke, M. Ciucci, A. Marino (2020). Simulated Investigation On The Seismic Performance Of A Multi-Component System For Major-Hazard Industrial Facilities. *17th World Conference on Earthquake Engineering, 17WCEE*, September 13-18 2020 Sendai, Japan.
- C-4** **C. Nardin**, R. di Filippo, R. Endrizzi, I. Lanese, F. Paolacci, O.S. Bursi (2020). A Ground Motion Model For Seismic Vulnerability Assessment Of Prototype Industrial Plants. *EURODYN 2020 - XI International Conference on Structural Dynamics*, 23—25 November 2020 Athens, Greece.
- C-5** C. Butenweg, O.S. Bursi, I. Lanese, M. Marinkovic, **C. Nardin**, F. Paolacci, A. Pavese, G. Quinci (2021). Experimental Investigation on the Seismic Performance of a Multi-Component System for Major-Hazard Industrial Facilities. *Proceedings of the Online ASME 2021 Pressure Vessels & Piping Conference PVP2021*, June 12-16 2021, Virtual Conference.
- C-6** **C. Nardin**, O.S. Bursi, I. Lanese, A. Pavese, F. Paolacci, G. Quinci (2021). Shake Table Testing for a Multi-Component Prototype Industrial Plant: Input and System Modelling Issues. *Proceedings of the Online ASME 2021 Pressure Vessels & Piping Conference PVP2021*, June 12-16 2021, Virtual Conference.

Projects:

- P-1** *Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities* (SPIF), European Union's Horizon 2020 Research and Innovation Programme - Grant Agreement No. 730900.

Deliverables:

- D-1** D10.1 - Technical report on SERA Transnational Access activities TA1-TA10 M36, Project #34 – *Seismic Performance of multi-component systems in special risk Industrial Facilities* (SPIF).

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List of abbreviations and notations

▪ Chapter 1:

ATC	Applied Technology Council
DS	Damage State
FEM	Finite Element Model
IM	Intensity Measure
LRFD	Load and Resistance Factor Design
MH	Metropolis Hastings
NIST	National Institute of Standards and Technology
NSC	Non Structural component
PBEE	Performance Based Earthquake Engineering
PCE	Polynomial Chaos Expansions
PSHA	Probabilistic Seismic Hazard Analysis
QoI	Quantity of Interest
UQ	Uncertainty Quantification

▪ Chapter 2:

ASD	Allowable Stress Design
BF	Braced Frame
BFJ	Bolted Flange Joint
DBE	Design Basis Earthquake
FE	Finite Element
FFT	Fast Fourier Transform
FIR	Finite Impulse Response
FRF	Frequency Response Function
FRS	Floor Response Spectra
LoC	Loss of Containment
LRFD	Load and Resistance Factor Design
MRF	Moment Resisting Frame
NC	Near Collapse
OBE	Operating Basis Earthquake
PBD	Performance Based Design
PBEE	Performance Based Earthquake Engineering
PGA	Peak Ground Acceleration
PSD	Power Spectral Density
QRA	Quantitative Risk Analysis
RC	Reinforced Concrete

RFP	Rational Fraction Polynomial
RMS	Root Mean Square
RND	Random
SERA	Seismology and Earthquake Research Alliance
SG	Strain Gauge
SI	System Identification
SIMO	Single Input Multiple Output
SPIF	Seismic Performance Industrial Facilities
SSE	Safe Shutdown Earthquake
UNITN	University of Trento

▪ Chapter 3:

ASD	Allowable Stress Design
BF	Braced Frame
BFJ	Bolted Flange Joint
DBE	Design Basis Earthquake
DP	Deaggregation Probabilistic
FRF	Frequency Response Function
FRS	Floor Response Spectra
FO	Fully Operational
GMM	Ground Motion Model
HR	Hazards Reduced
INGV	Istituto Nazionale di Geofisica e Vulcanologia
ITACA	Italian Accelerometric Archive Database
LRFD	Load-and-Resistance Factor Design
LF	Low Fidelity
LS	Life Safety
MRF	Moment Resisting Frame
NC	Near Collapse
NIST	National Institute of Standards and Technology
NSC	Non-Structural Component
PBEE	Performance-Based Earthquake Engineering
PGA	Peak Ground Acceleration
PR	Position Retention
QoI	Quantities of Interests
SG	Strain Gauges
SHA	Seismic Hazard Analysis
SI	System Identification

RFP	Rational Fraction Polynomial
RND	Random
SSE	Safe Shutdown Earthquake
TMD	Tuned Mass Damper

▪ **Chapter 4:**

AET	Acoustic Emission Technique
API	American Petroleum Institute
BFJ	Bolted Flange Joint
FBG	Fiber Bragg Grating
FIR	Finite Impulse Response
ISO	International Organization for Standardization
LD	Leakage Detection
LoC	Loss of Containment
MEMS	Micro-Electro-Mechanical-System
NDT	Non-Destructive Techniques
OF	Optical Fiber
RFID	Radio Frequency Identification
SG	Strain Gauges
SLED	Super Luminescent LED

▪ **Chapter 5:**

BF	Braced Frame
BFJ	Bolted Flange Joint
CBF	Concentrically Braced Frame
CBFEM	Component-Based Finite Element Model
CDF	Cumulative Distribution Function
DBE	Design Basis Earthquake
EDP	Engineering Demand Parameter
FEMU	Finite Element Model Updating
LoC	Loss of Containment
GMPE	Ground Motion Prediction Equation
IM	Intensity Measure
MAC	Modal Assurance Criterion
MAF	Mean Annual Frequency

MC	Monte Carlo
MCMC	Monte Carlo Markov Chain
MH	Metropolis-Hastings
MRF	Moment Resistant Frame
NSC	Non-Structural Component
PBED	Performance-Based Earthquake Design
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Analysis
QRA	Quantitative Risk Analysis
RFP	Rational Fraction Polynomial
SPIF	Seismic Performance of Industrial Facilities
SSE	Safe Shutdown Earthquake
TH	Time-History
TMD	Tuned Mass Damper
UHS	Uniform Hazard Spectrum

▪ Chapter 6:

AIC	Akaike Information Criterion
AS	After Shock
BF	Braced Frame
CP	Collapse Prevention
DBE	Design Basis Earthquake
DoF	Degree of Freedom
DS	Damage State
ED	Experimental Design
EDP	Engineering Demand Parameter
IM	Intensity Measure
IO	Immediate Occupancy
IRF	Impulse Response Function
MLE	Maximum Likelihood
MS	Main Shock
NLTHA	Non Linear Time History Analyses
NSC	Non Structural Component
PBEE	Performance Based Earthquake Engineering
PCA	Principal Component Analysis
PC	Principal Component
PCE	Polynomial Chaos Expansions
PSHA	Probabilistic Seismic Hazard Analysis

SNLTHA	Sequential Non Linear Time Histories Analyses
SPIF	Seismic Performance of Industrial Facilities
SS	Seismic Sequence
SSE	Safe Shutdown Earthquake
UQ	Uncertainty Quantification

1. Introduction

1.1 Background and motivation

Earthquakes are among the most destructive natural disasters. Increasing urbanisation and the accumulation of civil and industrial assets in severe seismic-prone zones have led to greater exposure to earthquake hazards worldwide, [1]. Over the past decade, a series of strong earthquakes - *e.g.* Tohoku 2011, Emilia 2012, New Zealand 2016 - have caused loss of life and inflicted catastrophic damage in several regions, leading to unprecedented reconstruction and retrofitting intervention costs. Losses include both damages to buildings and business interruptions. However, the indirect costs and consequences of the latter can be far-reaching, [2]. The losses inflicted on businesses by earthquakes, indeed, can devastate the industry and the economy in general. Damaged buildings, machinery and equipment are generally costly to repair, and the scale of resulting business interruption losses can be very high and relatively difficult to estimate.

Focusing on the industry world, from the last decade of major earthquake disasters, a key realisation has been that non-structural components (NSCs) of industrial plant facilities have been a significant driver for overall physical damage and disruption to business. To this scope, the Applied Technology Council (ATC) under the National Institute of Standards and Technology (NIST) invested in a year-long study to collect and document the body of available knowledge related to the seismic performance of NSCs, summarized in [3]. From these studies, it clearly emerges: (i) a large adoption of classical load-and-resistance factor design (LRFD) and code-compliant methods often dated and inadequate, as highlighted also in [4]-[5]; (ii) a gap in performing fully comprehensive testing campaigns and investigations on coupling effects between primary structures and NSCs; (iii) a need to improve and enforce code requirements along with the development of reliable NSCs seismic demands models. More precisely, codes and regulations establish the minimum requirements for structures to resist the effects of earthquakes with defined probability levels (or return periods), providing a basic level of seismic risk reduction. The primary objective of codes and regulations on structures is to save lives, not the structures themselves. When properly implemented, codes have significantly reduced the loss of life in major seismic events in recent years. However, from the perspective of economic resilience, the enormous scale of associated financial losses requires a paradigm shift. Encouragingly, in recent times,

earthquake building codes have been revised to implement elements of a performance-based earthquake engineering (PBEE) framework to address issues of damage limitation and reparability as well as life safety.

Based on the definition of PBEE stated in the Structural Engineers Association of California (SEAOC) Vision 2000 document [6], the first step in PBEE is to determine and select the performance objectives (PO), i.e. the link between expected performance levels and seismic ground motion intensities. In more detail, a performance level represents a distinct band in assessing damage to the overall structural and NSCs elements that also considers the consequences of the damage to the occupants and functions of the facility, [7]. POs typically include multiple goals for the performance of the constructed facilities. To this scope, Figure 1.1 initially presented in the SEAOC document [6], illustrates the PBEE framework applied to engineering facilities. A discrete set of pre-defined system POs and a set (discrete or continuous) of seismic hazard levels are represented on the two axes of the schematics. The domain defined by the two axes is then partitioned into two subdomains, the acceptable domain and its complement, the not acceptable. Within the acceptable domain, designers and stakeholders define the desired combination of POs w.r.t. the hazard level and its likelihood and the monetary cost of the system. According to this scheme, four discrete performance levels are identified and linked to earthquake design levels. Thus, the selection of POs sets the acceptability criteria for the design, i.e. the rules and guidelines to satisfy in order to ensure the operativity, safety, and business performance objectives.

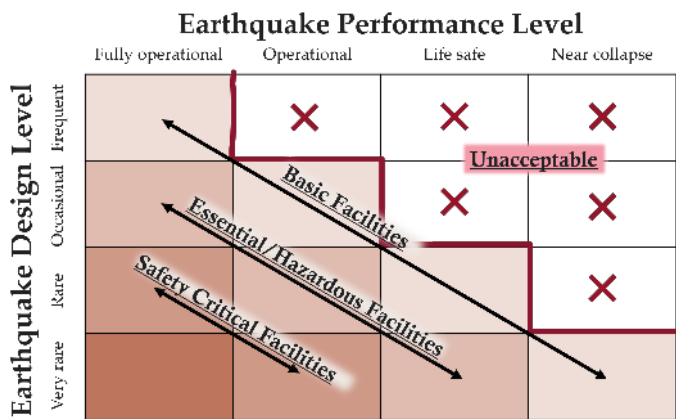


Figure 1.1: PBEE schematics: minimum seismic performance objectives coupled with earthquake levels for buildings, according to [7].

Also, SEAOC Vision 2000 describes various hazard levels with spe-

cific return periods: (1) the frequent intensity level with a 50% exceedance probability in 30 years, (2) the occasional intensity level with a 50% exceedance probability in 50 years, (3) the rare intensity level with a 10% in 50 years, and (4) the very rare intensity level with a 10% in 100 years.

Similarly, FEMA 365 “Prestandard and Commentary for the Seismic Rehabilitation of Buildings” [8] provides professional engineers with a tool for designing seismic rehabilitation measures for existing structures in a PBEE perspective. Indeed, the document defines various target building performance levels and earthquake hazard levels similar to those presented in SEAOC Vision 2000 by means of quantitative tables and qualitative descriptions of corresponding physical damage for both structures and NSCs.

Along the same line, in 1997 the Pacific Earthquake Engineering Research Center (PEER) decided to develop a more robust methodology for PBEE. The PEER PBEE framework developed by PEER facilitates direct calculation of the effects of uncertainty and randomness on each step in the performance-based procedure.

The framework is simply a statement of the total probability theorem for a selected decision variable’s yearly mean number of events. In specific, the so-named PEER formula was originally written as follows:

$$\lambda(dv) = \int_d \int_{im} G(dv|d) |dG(d|im)| |d\lambda(im)| \quad (1.1)$$

and later rewritten as

$$\lambda(dv) = \int_d \int_{edp} \int_{im} G(dv|d) |dG(d|edp)| |dG(edp|im)| |d\lambda(im)| \quad (1.2)$$

where IM (intensity measure), EDP (engineering demand parameter), D (damage state), and DV (decision variable) are continue random variables describing respectively the hazard intensity, the structural response, the state of the structure, and a suitable decision variable for decision making. $\lambda(x)$ is the mean annual rate of events exceeding a given threshold for a given variable x , and $G(y|x) = P(Y \geq y|X = x)$ is the conditional complementary cumulative distribution function (CCDF). Figure 1.2 shows the four steps into which the seismic assessment is destructured.

In this respect, the PBEE PEER framework has gained momentum above all to perform seismic risk analyses, thanks to its inherent versatility, [9]. Its strength, indeed, relies on the simple yet effective implementation of the total probability theorem, which allows to decouple and then combine the output of probabilistic seismic hazard analysis (PSHA) with fragility, damage, and loss analysis.

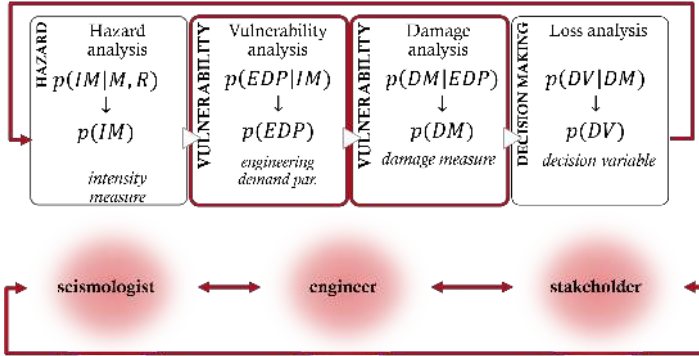


Figure 1.2: The PEER-PBEE schematics.

The huge structural damages caused by strong earthquakes, such as in the 1989 Loma Prieta, the 1994 Northridge, the 1999 Chi-Chi, Taiwan, the 1999 Kocaeli and 2011 Tohoku, Japan, earthquake [10]-[11]-[12], demonstrate that a dependable evaluation of seismic hazard appears of paramount importance. The definition of the levels of earthquake motions to be considered for analysis requires special attention. Several standards dealing with the seismic analysis of process plant components, e.g., NFPA 59A [13], in view of enhanced performance and damage limitation, use the same seismic hazard definitions adopted by nuclear standards NEA/CSNI/R (2007)17 [14]. These standards prescribe two different limit states: the design basis earthquake (DBE) and the safe shutdown earthquake (SSE). Nevertheless, intended safety objectives are different: in fact, (i) conventional facilities are designed for human lives protection and damage limitation [15]; conversely, (ii) NPPs rules enforce integrity and functionality of structures systems and safety-critical components; as a result, inelastic behaviour is not allowed. Thus, in moderate seismicity regions, a correspondence between the 10% probability of exceedance in 50 years, i.e., 475 years return period used in Eurocode 8 [15], and the design basis earthquake ground motion defined in nuclear standards can be established. Increased return periods can be achieved through the importance factors. Safe shutdown earthquake ground motion is usually defined according to rules properly defined for the special-risk industries. Therefore, the definition of the corresponding return periods for the design/assessment of process plants is more involved [16]. In any case, a full probabilistic seismic hazard analysis (PSHA) of the site is necessary. The hazard analysis provides the probability of exceeding a selected intensity measure (IM) value in a given

time interval. For the construction of fragility curves, it is often necessary to use nonlinear analysis, which requires seismic motion records [17]. If a scalar IM is used, such as the peak ground acceleration or spectra acceleration at the fundamental period ($Sa(T)$), the correlation with the response, expressed in terms of engineering demand parameters (EDPs), is generally low as shown in [18]. Thus, further uncertainty is introduced in addition to the attenuation law's uncertainty. Similar remarks can be made if the ground motion records are selected on the basis of a given range of frequencies usually provided by a target uniform hazard spectrum (UHS); see, in this respect, [19]. As an alternative, accelerograms whose spectrum individually matches the UHS could be used. These records can be artificially built or obtained by modifying, with appropriate techniques, the frequency content of records. The first approach has been completely abandoned, while the second one still represents an interesting alternative [20]-[21]. However, process plant components are often characterized by a variegate frequency content. This clearly complicates the problem of the accelerogram selection. In civil engineering, where complex constructions exhibit similar problems, vector-valued IMs have been defined [22].

To assess seismic risk, the aforementioned fragility analysis step offers the critical link between seismic hazard and structural modelling, since it estimates the conditional probability of exceeding or attaining a certain damage state (DS) given an intensity measure (IM) of earthquake motion. In this regard, a growing number of studies, the increase of community-funded research projects and experimental campaigns focused on unfolding the complex and highly diverse behaviour of these process components. For example, the European research project INDUSE-2-SAFETY 2012-2015 [23], investigated through experimental tests the performances of several NSCs and their limit state levels to be compliant with. Conversely, the United States Nuclear Regulatory Commission agency carried out a multi-year test programme to better understand the elastoplastic response and ultimate strength of nuclear power piping systems, [24]. As a result, in view of the enhancement of structural integrity of industrial, energy and nuclear components as well as fitness for service, design guidelines and recommendations were issued [25] - [26].

However, the problem's complexity and its inherent modelling, coupled with a general scarcity of available data and the model's limited capability, concurred in limiting or developing risk assessment methods based on meaningful reduced and quick-to-run models. Indeed, the computation of fragility curves requires a realistic estimation of the structure performance subjected to seismic excitations *via* the quantification and

propagation of uncertainties existing in seismic ground motions, structural material properties and so on. Thus, it requires hundreds of heavy numerical FE model simulations. Recently, new studies regarding the application of metamodels or surrogate models in fragility analysis have been realized. Most of them focus on using seismic intensity measures (IMs) to characterize earthquakes. Metamodels are then constructed to calibrate the relation between damage measures (DMs) and uncertain inputs of the structural models, including IMs and structural parameters. Among surrogate families that provide more than just an approximation to the model, polynomial chaos expansions (PCE) [27], Kriging, or Gaussian process modelling [28]-[29], support vector machines, artificial neural networks [30]-[31]-[32] became the most popular in the civil engineering domain. For example, they might additionally yield analytical estimates of the moment response model, sensitivity indices or confidence levels for their predictions.

Moreover, in recent years, also several novel methodological contributions to fragility analysis have been made that include: i) the development of multivariate fragility functions [31]; ii) the introduction of seismic fragility analysis based on a combination of artificial ground motions and surrogate modeling [28]-[33]; iii) the consideration of both state and time-dependent fragility [34]-[35]. On the latter point, Iervolino et al. [36] focused on the estimation of state-dependent fragility curves, which are fragility models that allow for the possibility that a structure may have already been damaged by previous shocks. To overcome the computational cost of the methodology, the authors proposed a solution applicable to reinforced concrete single degree of freedom (SDOF) systems, calibrated through pushover analyses [37] and based on discrete-time homogenous Markovian process. In particular, the fragility functions, or probabilities of transition from a damage limit state to a severe one, are interpreted as the element of a Markovian transition matrix, [38]-[36].

1.2 Objectives of the thesis and core contribution

On these premises, this dissertation aims to develop an innovative, efficient and accurate framework for evaluating state-dependent fragility functions for critical industrial process components. The relevant vulnerability assessment is based on the integration of data derived from an intensive experimental campaign, accomplished within the European SERA SPIF project, with the computational potential obtained from the most advanced metamodeling techniques available. The intrinsic value of this framework resides in the experimental data on which it is built

and implemented. These are the offspring of the comprehensive experimental programme completed at the EUCENTRE laboratories in Pavia. In particular, the tests carried out on a shaking table of two archetypes of industrial systems full-scale steel substructures, equipped with NSCs such as tanks, cabinets, BFJs, and piping elements, represents an added value of the thesis. In fact, experimental data fill the gap in terms of experimental evidence of tests conducted in a PBEE framework capable of taking into account the effects of NSCs-primary structure coupling as a whole.

Therefore, another added value of the thesis lies in the performance evaluation and study of the complex interactions between NSCs and the primary structures through the development of different highly refined numerical models capable of capturing interaction effects. They are conceived to be quicker and inexpensive-to-run and able to capture the global behaviour of each coupled system.

1.3 Organization of the thesis

The thesis presents the experimental and numerical research the Author performed during her PhD programme, along with the relevant analytical findings and achievements. Thus, the thesis is organized as follows.

Chapter 1 provides a concise overview of the problem and the statement of the research objectives. It includes the experimental shake table test campaigns and the vulnerability seismic assessment for special risk industrial substructures.

The subsequent Chapters 2 and 3 move into the core of the experimental research activity. Drawing on papers already published by the Author and her research group, the comprehensive shake table test campaigns of the two most widespread steel frame configurations for industrial plant substructures are presented. In detail, Chapter 2 presents the shake table tests of a moment resisting frame structure excited by an EC8-spectrum-compatible accelerogram. Then, Chapter 3 describes the experimental campaign carried out by means of a synthetic seismic input, derived from a site-based ground motion model, on the same mock-up converted into a braced frame configuration, thanks to *ad-hoc* design and installation of a seismic retrofitted bracing system.

Within the experimental domain, Chapter 4 provides insights into the performance of an innovative fiber-optic monitoring system, which was deployed and tested during the shake table tests, for one of the most critical components in major-hazard industrial facilities: a bolted-flange joint.

Chapter 5 discusses the analytical and numerical models developed: (i) to monitor the experimental campaign; (ii) to better interpret the physical phenomena that governed the dynamic behaviour of the coupled system; and (iii) to perform seismic risk analyses of non-structural components. For this purpose, particular emphasis is devoted to developing a simple global model that is calibrated on complex local high-fidelity models and balances the accuracy of results with the computational-time burden.

Along this vein, Chapter 6 discusses an innovative framework for the formulation of state-dependent fragility curves that combines experimental data with the most advanced computational metamodeling tools, in a forward uncertainty quantification (UQ) perspective. By leveraging the latter, expensive-to-evaluate computational models, e.g., FE models, are substituted with inexpensive-to-evaluate surrogates, preserving the required accuracy for the quantity of interest (QoI) investigated.

The findings are finally summarized in Chapter 7, where the main research results are underlined together with future perspectives.

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2. Experimental performance of multi-storey MR frame structure with process equipment subjected to artificial ground motions

This chapter contains the post-print of

Butenweg, C., Bursi, O.S., Paolacci, F., Marinković, M., Lanese, I., **Nardin, C.**, Quinci, G. (2021). Seismic performance of an industrial multi-storey frame structure with process equipment subjected to shake table testing, *Engineering Structures*, Volume 243, doi:10.1016/j.engstruct.2021.112681,

as published in the Journal of *Engineering Structures*, differing from the published paper only in terms of layout and formatting.

Author contributions. **C. Butenweg:** Funding Acquisition. **O.s. Bursi:** Supervision, Writing – Review. **F. Paolacci:** Writing – Original Draft. **M. Marinkovic:** Methodology. **I. Lanese:** Investigation. **C. Nardin:** Formal analysis, Software, Data Curation, Writing - Original Draft. **G. Quinci:** Formal analysis, Software, Data Curation.

Summary

Past earthquakes demonstrated the high vulnerability of industrial facilities equipped with complex process technologies leading to serious damage of process equipment and multiple and simultaneous release of hazardous substances. Nonetheless, current standards for seismic design of industrial facilities are considered inadequate to guarantee proper safety conditions against exceptional events entailing loss of containment and related consequences. On these premises, the SPIF project -Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities- was proposed within the framework of the European H2020 SERA funding scheme. In detail, the objective of the SPIF project is the investigation of the seismic behaviour of a representative industrial multi-storey frame structure equipped with complex process components by means of shaking table tests.

Along this main vein and in a performance-based design perspective, the issues investigated in depth are the interaction between a primary moment resisting frame (MRF) steel structure and secondary process components that influence the performance of the whole system; and a proper check of floor spectra predictions. The evaluation of experimental data clearly shows a favourable performance of the MRF structure, some weaknesses of local details due to the interaction between floor crossbeams and process components and, finally, the overconservatism of current design standards w.r.t. floor spectra predictions.

2.1 Introduction

2.1.1 Background and Motivation

Most recent destructive earthquakes in China (Sichuan, 2008 and Yushu, 2010), Japan (Tohoku, 2011) and Italy (Emilia, 2012) highlighted the social and political consequences of seismic risk due to industrial facilities.

Several reports, like [1] and [2], along with a growing body of literature by European and United States researchers, (e.g. [3] - [4]) investigated numerous industrial accidents that have resulted in severe loss of life and injury, damage to natural and built environment, as well as significant economic losses.

As past earthquakes have demonstrated, in critical facilities, interactions between supporting structures and plant components or process-related interactions are above all vulnerable and capable of leading more severe serious secondary damages. If hazardous substances are released due to loss of containment (LoC), in addition to loss of production, they also entail a major danger to both humans and environment .

As many contributions demonstrated in literature, see, among others, in [5] - [6], seismic behaviour of support structures in major-hazard industrial/nuclear plants is therefore of paramount importance. Besides, recent studies [7] - [8] - [9], focused on the analysis and design of industrial and nuclear plant substructures equipped with process components such as pipes, tanks, etc. It is noteworthy that whilst several aspects in terms of design and analysis are still unresolved, code-compliant methods demonstrated to result in very poor predictions and often dated and inadequate. as highlighted in [10] - [11].

As a matter of facts, nowadays seismic provisions of industrial facilities are based on classical load-and-resistance factor design (LRFD) or on allowable stress design (ASD), like ASME B31.1 [12], ASME

B31.3 [13] and EN 13480 [14], while a performance-based earthquake engineering (PBEE) approach should be preferred. In fact a LRFD approach guarantee performance primary in terms of failure probability of individual structural components, while a PBD approach can be used to assess several metrics of seismic performance, including the previously highlighted economic losses due to repair costs, downtime and fatalities.

While for civil-engineering structures- probabilities of exceedence of peak ground acceleration (PGA) vs limit states are clearly identified, little information can be found in literature for hazardous industrial equipment. Along this vein, a first attempt of correlation for petrochemical plants can be found in [15], where typical refinery piping systems were analysed referring to Operating Basis Earthquakes (OBEs) and Safe Shutdown Earthquakes (SSEs). The former corresponds to a probability of exceedence of 10% in 50 years and is defined as the condition under which plants remain fully functional without undue risk to health and safety of public. Increased return periods can be achieved through additional PGA multipliers. The latter is related to a lower probability of exceedence under which certain relevant structures, systems and important components must be designed to remain operational and allow a safe closure. In order to cover a greater ensemble of industrial facilities, the authors suggested to extend these considerations to other types of configurations.

The importance of well-established relationships between limit states and reached damages in an industrial or nuclear system is clearly demonstrated by the consequences of failures of components in several accidents in industrial plants, as already illustrated in [1] - [4]. Consequences include the release of hazardous materials, human injuries and the increasing of overall damage to nearby areas, proving this to be a key emerging risk issue. More precisely, in view of quantitative seismic risk analysis (QRA) Caputo et al. [16] estimated consequences of some major-hazard process plants subjected to earthquakes. It emerged on one hand, a limited knowledge of the seismic behaviour of both critical units, components and support structures; on the other hand, the lack of information about their interactions. In fact, release of hazardous materials from secondary elements, i.e. pipes, tanks, etc., even not directly considered by current standards, could generate uncontrollable disasters, dramatically exacerbated by domino effects [17]. Therefore, researchers, [18] - [19], focused on the relation quantification between loss of containment (LoC) and seismic damage; moreover, since current design practice for piping systems contain little information about seismic component details [20], Bursi et al. [21] suggested innovative solutions based on non-standard bolted flange joints (BFJs).

Along the same line, also the quantification of the behaviour factor q for components on support structures has revealed to be inaccurate. For instance, while for piping systems [22] prescribes a q -factor in the range of 6 – 12, [23] demonstrated through direct nonlinear analyses, values of q not greater than 3 – 4. Besides, the authors explained this overestimation due to nonlinearities in the supporting structure which decreased the coupling effect and, hence, damage both in pipes and supports. Moreover, Bursi et al. [24] showed that a typical petrochemical piping system coupled with a steel support structure is endowed with a limited q factor, because both pipes and the support floors cannot ensure a rigid floor behavior and a great involvement of dissipative elements. In addition, [25] and [26] demonstrated that the response of a piping system can be greatly reduced by including the nonlinear behaviour of local supports.

Both the 2001 El Salvador [10] and the 2006 Hawaii earthquakes [27] have demonstrated that non-structural damage has clearly limited the functionality of critical facilities and, highlighted the importance of non-structural component performance. Furthermore, [28] and [29] underlined a definite lack of investigation as well as protocols for testing the interactions between primary structures and components under seismic loading.

As regards the analysis of global systems based on PBEE, only a few studies can be found, see [28] and [24], among others, where interactions between structural elements and components have been performed. For instance, the application of PBEE to two realistic substructures of petrochemical piping systems showed a general overconservatism in piping system design. Conversely, some studies, e.g. the European research projects INDUSE 2009-2012 and INDUSE-2-SAFETY 2012-2015 [30] clearly focused on experimental testing of non-structural components and performance levels to be compliant with. More precisely, Bursi et al. [31] investigated the behaviour of non-standard BFJs endowed with plastic deformation capacity, and demonstrated that, in case of strong earthquakes, a limited localized plastification before LoC can be beneficial. Other studies investigated the mechanical behaviour of tanks, pipes, nozzles and elbows subjected to strong seismic loading, based on large-scale experimental work with extensive numerical simulations. As a result, in view of the enhancement of structural integrity of industrial and nuclear components, design guidelines and recommendations were issued.

Strictly related to PBEE, the development of involved nonlinear analysis methods for accurate evaluation of both limit states and damage represents another salient issue [32]. Step-by-step dynamic integration of a structural system, where explicit nonlinear behaviour of both primary

and secondary components are considered, is clearly the most reliable analysis method. Nonetheless, simplified approach such as the use of nonlinear static analysis or the use of floor response spectra (FRS) are generally preferred. For instance, [33] demonstrated that common nonlinear static analyses cannot be used to assess the response of a coupled pipe-rack system. The main difficulty derives from the presence of non-building characteristics of this type of structures [22]. Both the absence of a rigid floor and the lack of dynamic interaction between primary (support) and secondary (components) system are among the main causes of discrepancies [34].

Along the same vein, floor acceleration/displacement spectra represent a viable and promising approach which, however, has been applied to a lesser extent to the case of industrial and nuclear power plant structures; see, among others, [7] - [35]. For instance, Merino Vela et al. [7] focused on the development of floor acceleration spectra for an industrial concentrically braced frame supporting a liquid storage tank. The authors demonstrated that in most of the cases, code-compliant analytical approaches overpredict responses w.r.t. analyses that explicitly take into account the interaction between a supporting structure and a liquid storage tank. When [36] is used for the estimation of floor spectrum values, differences up to 30-35% were detected. Finally, Ezeberry and Combescure, [35], proposed a suitable methodology to determine seismic FRS at a particular location of a structural system by a direct method based on the frequency domain.



Figure 2.1: Mock-up photographs at the EUCENTRE Laboratory.

The technique was validated on nuclear reactor mechanical devices inside the nuclear-rated structure of the ITER Tokamak Complex.

2.1.2 Scope

On these premises and to crystallize the ideas, both the interaction between primary and secondary elements that influence the performance of the whole system and proper and check of FRS predictions represent the main issues that the project SPIF want to investigate in depth in a PBEE perspective. Therefore, in order to fill the lack of experimental evidences, the SPIF project [37] has foreseen an experimental campaign of a representative industrial structure equipped with complex process technology subjected to shake table tests. Thus, the present paper, gathers the main characteristics of an archetype industrial facility, the test programme, and main results.

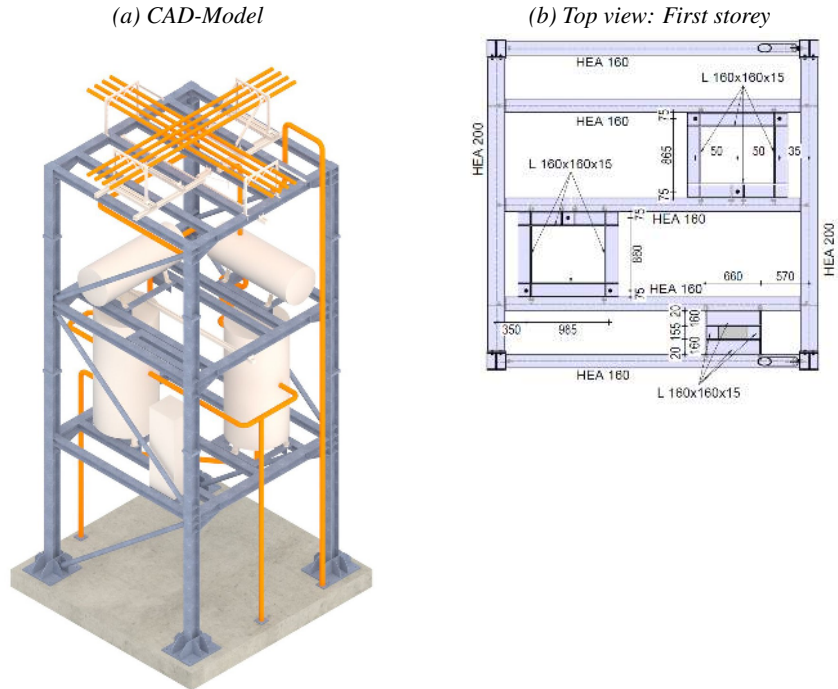
In particular, Section 2 describes the case study, a steel 3-levels prototype of an industrial plant structure equipped with typical industrial components such as vertical and horizontal tanks, cabinet and pipes with elbows, special flanges and T-joints; also the sensor layout is presented. Then, Section 3 deals with the definition of the test programme and the seismic input selection. More precisely, shaking table tests, characterised by spectrum-compatible accelerogram levels ranging from 0.16g to 0.71g PGA, have been performed. In Section 4, a significant amount of acquired data is assessed and system identification of modal frequencies, mode shapes and damping coefficients through a rational fraction polynomial (RFP) method are carried out. The definition of relationships between damage levels, limit states and performance of components are carried out in Section 5. Moreover, comparisons between FRS estimated through experimental data and those evaluated in compliance with codes and standards are also performed, resulting the latter in a clear underestimation when close to the resonance condition. Finally, conclusions and future developments are drawn in Section 6.

2.2 Experimental Mock-up and Setup

2.2.1 Mock-up Description

The full scale mock-up consists of a primary steel structure supporting horizontal and vertical tanks, piping installations and a cabinet. Furthermore, a conveyor was placed on the reinforced concrete slab without any connection to the test structure. The main structure was conceived as a three-storey steel frame with flexible diaphragms made of crossbeams hinged to the frame beams. Figure 2.1 shows the test structure on the shaking table with different views on each storey and Figure 2.3a depicts

the top view of the CAD-model. The dimensions of the structure are 3.7 m x 3.7 m in plan with a storey height of 3.1 m, which leads to a total height of 9.3 m. The columns were fully clamped to a reinforced concrete base plate. This latter has dimensions of 4.8 m x 4.8 m and a thickness of 0.4 m and was constructed with a concrete of class C30/37. The horizontal load bearing system of the steel structure consisted of two moment resisting frames (MRFs) in the direction of the seismic excitation and two braced frames in the transverse direction equipped with tension/compression bracings with circular section. These latter were used in order to limit translational movements and torsional effects. The arrangement of the profiles on each storey level is shown on the example of the first storey in Figure 2.3b. The two MRFs were connected through crossbeams, which are fastened to the frame beams by standardized bolted connections with web stiffeners as shown in Figure 2.3c.



(c) Connections of crossbeams (HEA 160) to the MRF and bearing platforms (L 160x160x3)

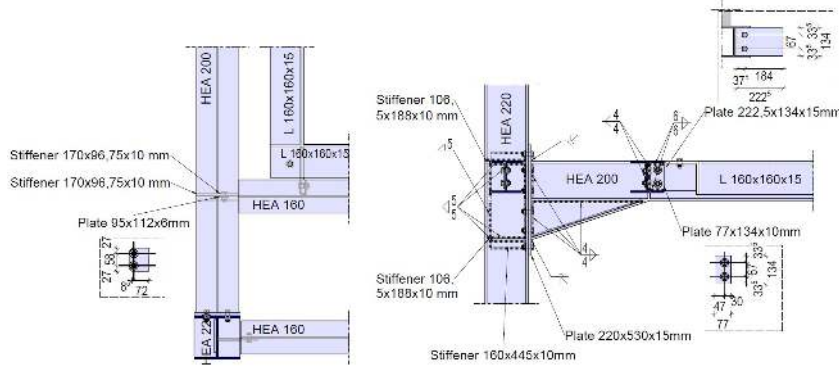


Figure 2.3: Primary steel structure

These crossbeams served in turn as bearing supports for bearing platforms which were installed with bolted connections to place the ancillary elements. In particular, HEA220 profiles were used for the columns, whereas HEA200 and HEA180 profiles were employed for the longitudinal girders on the first level and the remaining ones, respectively. Moreover, the transversal beams were made of HEA160 profiles, the horizontal frames with L-Profiles (160 x 160 x 15 mm) and the diagonal bracings with a tube of cross section 101.6 x 3 mm. All profiles were made of steel grade S355. The MRF joints were bolted connections with welded stiffeners and haunches; whereas, the diagonal bracings were connected to the columns using bolted connections as well. The bearing platforms were bolted to the cross-beams. All bolted connections are executed with bolts M16 with strength class 10.9. The columns of the steel frame were welded on steel anchor plate of 30 mm thickness which, in turn, were anchored to the reinforced concrete slab. The anchorage system was composed of a combination of both special chemical anchors and continuous anchors. More precisely, five M30 anchors per column, going all the way through the concrete slab, were used, along with three M24 bars chemically anchored to the slab. In addition, to guarantee the necessary stiffness and strength of column base joints, they were equipped with six stiffeners of 20 mm thickness welded both to the column and to the anchor plate, as depicted in Fig. 2.4.



Figure 2.4: Typical anchorage of columns to the reinforced concrete plate: base plate with welded stiffeners and anchors.

The secondary elements of the industrial structure included tanks, pipes, elbows, bolted flange joints (BFJs), tee-pipe joints, see Figure 2.7, and a cabinet. Four unpressurized tanks made of steel S235JRG2 with a yield strength of 235 N/mm^2 were installed: two vertical tanks on the first floor, i.e. Tank #1 and #2; and two horizontal tanks on the second floor, i.e. Tank #3 and #4, respectively. Their locations are shown in Figure 2.9. In order to simulate the liquid typically stored in such tanks, they were filled with granular material with a density equal to water. This allowed for the protection of the shaking table and the technical installations against liquid release. The geometry and dimensions of the tanks are shown in Figure 2.5. In particular, both vertical tanks were 2.76 m high, with a diameter of 1.25 m and a thickness of 5.6 mm, with a self-weight of around 0.65 tons. They were supported on three symmetrically positioned supports, 120° apart from each other. They were endowed with a filling capacity of 3000 liters and were filled up to 80 percent of their capacity, which resulted in an additional mass due to filling of 2.4 tons. Each support of the tank was fastened to the corresponding bearing platform with a M20 bolt of grade 10.9. The horizontal tanks were instead 2.12 m long, with a diameter of 0.8 m, a thickness of roughly 3.9 mm and a self-weight of about 0.3 ton. They were supported on four symmetrically positioned supports, 1.14 m apart from each other in length. They are endowed with a filling capacity of 1000 liters and were filled up to 80 per cent of their capacity, which resulted in an additional mass due to filling of 0.8 ton.

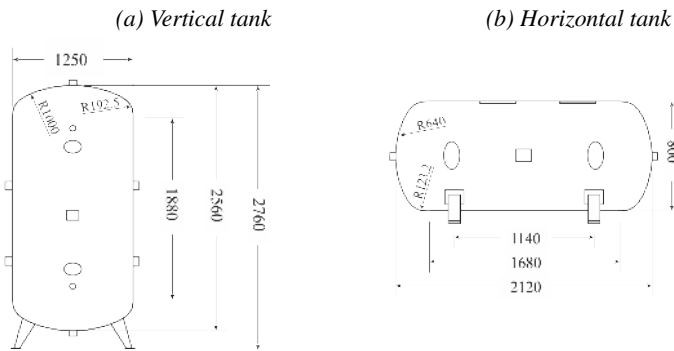
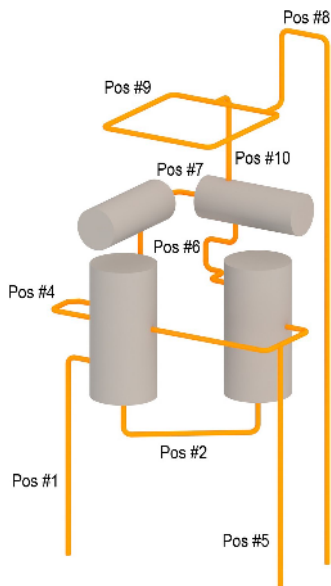


Figure 2.5: Geometry of the vertical (a) and horizontal (b) tanks

The tanks were mutually connected to each other through a piping system consisted of nine DN100 pipes, with a diameter $D=100$ mm and a thickness $t=3.6$ mm.

(a) Secondary elements layout



(b) Loose flange joint pipe Pos #2



(c) Base plate pipe Pos #3



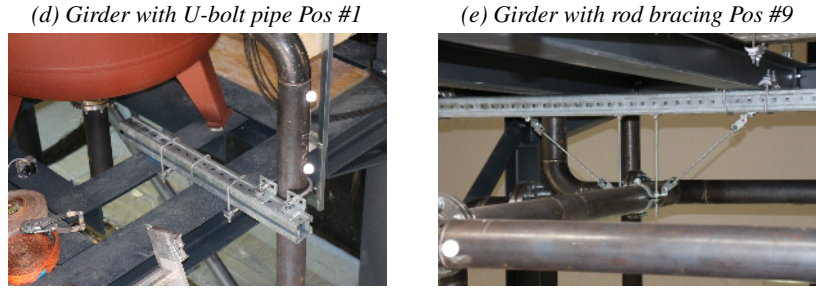


Figure 2.7: Piping system and layout

Some pipes were also connected to the concrete slab. The relevant pipe layout is shown in Fig. 2.7, and consists of straight branches, elbows and tee pipe joints. The material used for the pipes was P235 with a yield strength of 235 N/mm^2 . Moreover, a suspended pipe rack was also installed on the third floor which is endowed with DN80 pipes. The pipe positions #1, #3 and #5, see Fig. 2.7, were attached to the concrete slab through a 20 mm thick baseplate, which was welded to the pipes and bolted to the concrete plate with 4 x M16 injected bolts at each plate. Moreover, bolted flange joints were necessary in order to connect the several parts of pipe. More precisely, the loose flanges adjacent to the tanks were conceived as moving flanges, while the others were simple flanges welded to pipes. Both flange joints can be seen in Fig. 2.7. Moreover, the pipe branches with the most critical bolted flange joints, i.e. Pos #1, Pos #4, Pos #5 and Pos #6 were filled with water and pressurized at 20 bars. The filling was conducted, after the pipes had been installed, through retaining valves integrated into the pipes.

The fastening and positioning of the pipes were accomplished by implementing a channel support system. This consisted of a series of girders with a hollow square cross section of 90 mm x 90 mm in different lengths that were clamped to the steel structure. The pipes were held in place either by a U-bolt, that prevented them from a horizontal movement, or by a beam clamp, i.e. Pos #4 and Pos #9, that prevented them from moving vertically. Additionally, the support of Pos #9 was accompanied by a truss system, made of steel rods, that also held the pipes horizontally in place.

The different configurations are illustrated in Fig. 2.7. In total eight pipes, each 4.9 m long, were installed: four parallel to the motion direction and four in the orthogonal direction as shown in Fig. 2.8. The pipes were secured to the main structure with frames and they were suspended with beam clamps connected through a steel rod as depicted in Fig. 2.8.

The pipes were also sealed at both ends, as they were completely filled with water.



Figure 2.8: Pipe rack on the third floor

2.2.2 Sensors Layout

A series of sensors were positioned on both the primary structure and process units. The instrumentation consists of accelerometers, displacement transducers (LVDTs) and strain gauges (SGs). The distribution of the sensors is summarized in Table 2.1. In particular, in order to analyze the dynamic behaviour of both the primary structure and process units a total of 54 uniaxial accelerometers have been placed at various floors and on the tanks. The location of the accelerometers is summarized in Fig. 2.9. In particular, in order to identify the seismic input to which the structure was actually subjected to, three accelerometers A #2, A #3 and A #4 were installed on the RC slab. Moreover, data provided by the accelerometers at each floor allowed for the dynamic identification of the primary structure and the definition of floor response spectra. Accelerometers A #9, A #12, A #17 and A #18, located respectively on Tank #1, Tank #2, Tank #3 and Tank #4, allowed for the dynamic identification of the equipment; while two accelerometers A #8 and A #11 placed at the base of the vertical tanks were used to quantify floor accelerations. In order to evaluate the local behaviour of the primary steel structure, a series of LVDT transducers and SG sensors were installed as indicated in Fig. 2.9. More precisely, in order to monitor uplift of base joints LVDT #3 to LVDT #10 were installed in the vertical direction between the column base plate and the RC slab; conversely, to estimate the average curvature and the axial deformation of member cross sections, strain gauges were installed at the base of columns.

The most critical BFJs of the piping system, i.e. Pos #1, Pos #4,

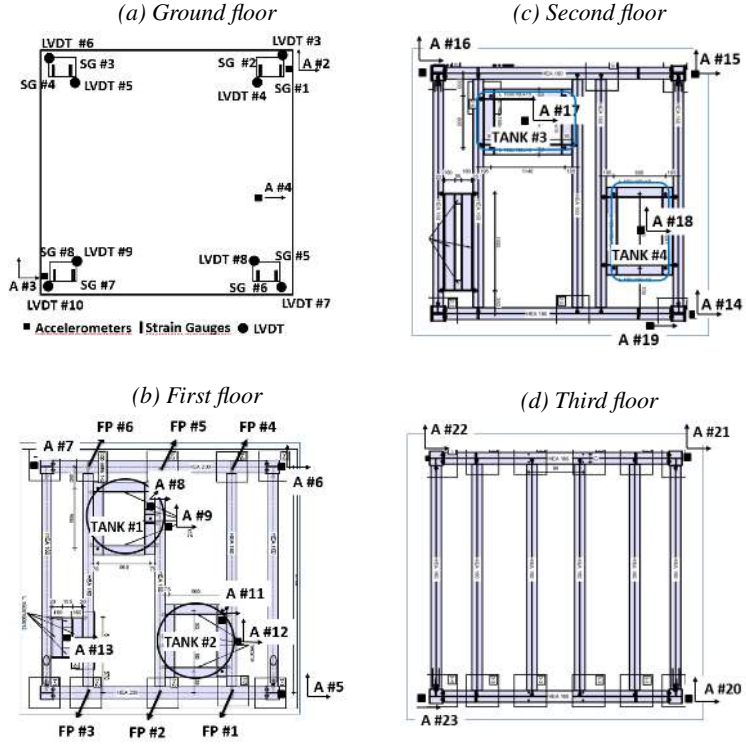


Figure 2.9: Sensors location at each floor

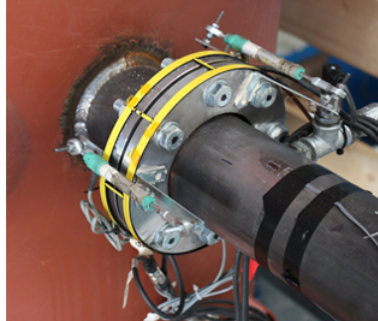
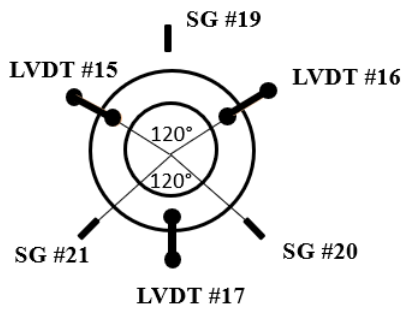
Pos #5 and Pos #6 shown in Fig. 2.10, were monitored by using LVDTs and SGs. In particular, with the aim to quantify strain amounts during seismic excitations and check possible yielding three SGs were installed on pipes just close to flange joints. Furthermore, in order to monitor the flange opening and to derive the relative rotation of flanges during seismic loading, three LVDTs were installed in each BFJ with an angular distance of 120° . In addition, with the aim to monitor a potential drop of pressure due to leakage, a pressure transducer was installed in each flange.

The displacement sensor setup was also complemented with an optical measurement system, made of 78 markers applied to different points of both the primary structure and secondary elements. They are indicated in Fig. 2.11. In order to allow for estimation of both local and global movements they were tracked by 16 cameras equipped with an infrared sensors for movement recording.

Table 2.1: Sensors type and numbers in the mock-up

Level	Accelerometer	LVDT	Strain gauges
Floor 0	6	15	12
Floor 1	16	-	-
Floor 2	11	-	-
Floor 3	7	-	-
Roof top	14	4	-
Pipes (Pos. #1, #4, #5, #6)	-	12	12

(a) BFJ Pos #4 - Top and side view



(b) BFJ Pos #1 - Top and side view

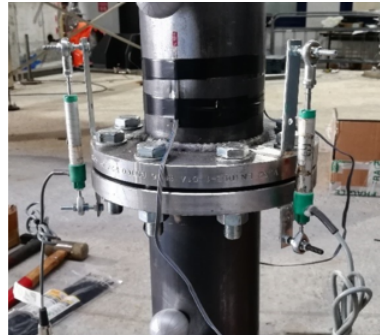
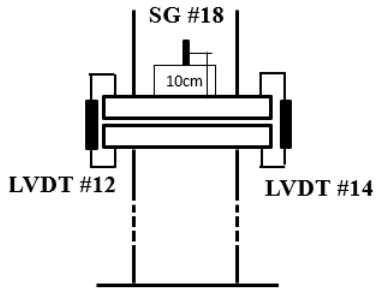


Figure 2.10: Piping monitoring: sensors location on the BFJs

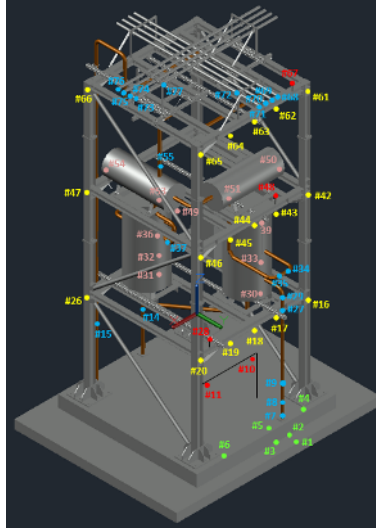


Figure 2.11: Markers layout

2.3 Selection of seismic input and Test Programme

2.3.1 Performance Based Design and Reference Limit States

In order to define proper limit states for the multi-storey frame with process equipment, we recall the main goals of the SPIF project i.e. the quantification of the interaction between the primary MRF structure and process components and a proper check of floor spectra predicted by standards and specialised literature. With respect to the first objective and according to the PBEE approach [38] and [39], the primary structure was designed to cope with the *Near Collapse* (NC) limit state; as a result, a severe earthquake prone-scenario was assumed leading to a design PGA of 0.69g. Conversely, the secondary process components were designed according to nowadays common practice; nonetheless, these components satisfy both the design basis earthquake (DBE) and safe-shutdown earthquake (SSE) requirements [40], i.e. for the foremost, they remain functional for even strong earthquake that occurs during the service life of the plant, for the latter, they allow for safe closure of process plants without undue risk to the health and safety of the public. Therefore, in agreement with the cited standards, for the primary structure we correlated different limit states to PGA levels as collected in Tab. 2.2; whilst Tab. 2.3 gathers limit states to PGA levels for process plant components. Those PGA values were considered in the test programme discussed in Subsection 3.3. In view of the preliminary design of the test structure, two models of different complexity were used: i) a sim-

plified model with secondary components idealized with concentrated masses and subjected to elastic and pushover analysis; ii) a sophisticated model, characterised by a detailed discretization of secondary components and connecting pipes and devoted to damage analyses, subjected to non-linear time histories.

Table 2.2: Significant limit states for primary structures.

Limit States		P_{VR}		T_R	$S.F.$	PGA
Fully Operational	- OP	81	%	75	0.31	0.22
Damage Limitation	- DL	63	%	125	0.37	0.26
Significant Damage	- SD	10	%	1188	0.79	0.54
Near Collapse	- NC	5	%	2438	1.7	0.69

Table 2.3: Significant limit states for process plants.

Limit States		P_{VR}		T_R	$S.F.$	PGA
Operating Basis	- DSE	10	%	1200	0.80	0.54
Safe Shutdown	- SSE	5	%	2475	1.01	0.69

2.3.2 Spectrum-compatible Accelerograms

The applied seismic action was based on a linear elastic response spectrum Type 1, with soil condition C, a $PGA=0.6g$ and a damping ratio of 5 %, in agreement with Eurocode 8-Part 1 [41]. The spectrum, depicted in Figure 2.12, exhibits a spectral acceleration, $S_a(T = 0) = 0.63g$ and a plateau between 0.2s and 0.6s with a maximum value $S_a(T = 0.2 \div 0.6) = 1.725g$.

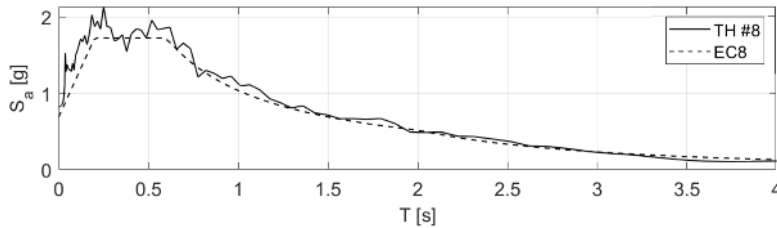
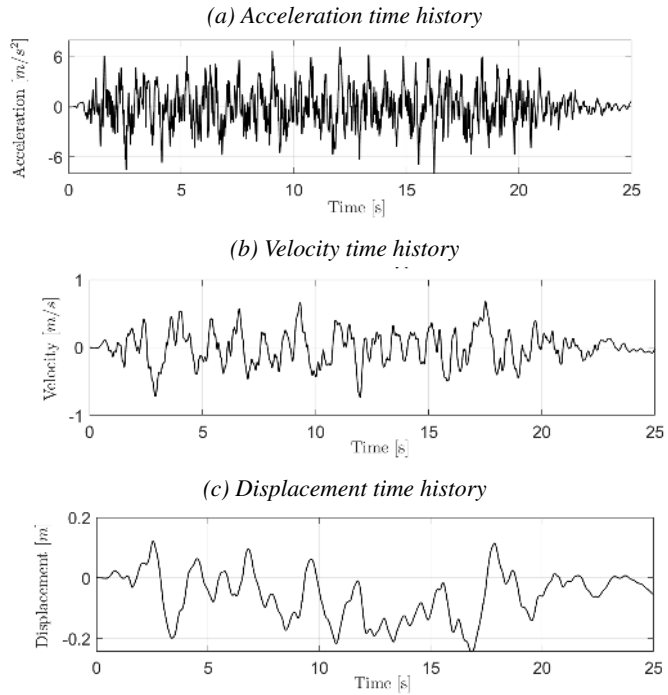


Figure 2.12: Eurocode 8-Part 1 Target spectrum and artificially generated spectrum for time histories TH #8.

In compliance with the aforementioned spectrum, a set of 10 stochastically independent accelerograms, TH #1-TH #10, was generated. In more detail, the accelerograms were generated with a trigonometric window function for a duration of 25s, a time step of 0.0025s, a rise time of

2s, a strong motion phase of 13s and a decay phase of 10s. The simulations conducted on a finite element (FE) model of the mock-up, with all generated accelerograms, allowed for the identification of the accelerogram TH #8, due to the relative amount of damage induced. In this respect, Figure 2.13(d) shows the acceleration time history TH #8 for a reference PGA of 0.63 g and its corresponding velocity and displacement time histories. The signal was applied through the shaking table with a sampling rate of 256 Hz, a total duration of 25 s and a baseline correction. The spectrum tolerance, i.e. the accepted difference between the spectrum of each generated accelerogram and that of the target spectrum, was satisfied with an absolute value of 0.1. The generation of accelerograms was based on a quadratic baseline correction. Moreover, during signal generation, the compatibility with the shaking table was checked both in terms of velocity and displacement capacities; therefore, no further signal filtering and processing was needed.



(d) Artificially-generated time histories TH #8.

2.3.3 Test Programme

The objective of the test programme was to impose different PGA levels -defined in Tabs. 2.2 and 2.3- to the test structure. Besides, in

view of the tuning of the shaking table and the dynamic identification of the mock-up, a series of tests with random excitation and low PGA levels ranging between 0.05 g and 0.15 g were performed. Thus, the seismic excitation was then scaled with respect to (w.r.t.) the design PGA of 0.63 g to 25 %, 37 %, 50 %, 70 %, 100 % and 110 %. For the sake of clarity, white noise random excitation signals with constant power spectral density (PSD) in a certain frequency range, are commonly used in the progressive system identification of mock-ups tested on a shake table. Therefore, the reference for the random excitation was a constant root mean square (RMS) intensity on frequencies in the range of 0.5 – 50 Hz, with the aforementioned PGA levels. A higher intensity allows for a better signal tracking and a higher signal-to-noise ratio; nonetheless, excessively high intensities may produce unwanted inelastic responses.

The testing campaign and the corresponding nomenclature is collected in Table 3.4. In detail, the first two runs were conducted with the random excitations RND #13 and RND #15. Successively, runs with PGA levels of 0.16 g, 0.23 g and 0.31 g were carried out. Before the test to be conducted at a level of 0.44 g, a further tuning of the shaking table was performed with a random excitation RND #16 and a relevant seismic excitation at 0.23 g PGA

After the successful completion of the test with a PGA level of 0.44 g, a last tuning phase was performed with a random excitation RND #17 and a seismic excitation with 0.23 g PGA. The final tests were executed for the maximum levels of 0.64 g and 0.71 g PGA. Lastly, a closing test with the random excitation RND #18 was carried out.

Table 2.4: Nomenclature, characteristics and synthetic observations of SPIF Test Programme.

Label	Excitation	State	Scaling factor	PGA Level	Observations
RND #13	Random	DBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
RND #15	Random	DBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-25%	Seismic	DBE - OP	25%	0.16 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	DBE - OP	37%	0.23 g	- Overall elastic linear behaviour
TH #8-50%	Seismic	DBE - DL	50%	0.31 g	- Overall elastic linear behaviour
RND #16	Random	DBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	DBE - OP	37%	0.23 g	- 3 rd floor: collapse of pipe rack in the transversal direction
TH #8-70%	Seismic	DBE - DL	70%	0.44 g	- Large displ.($\pm 30\text{ mm}$) of vertical Tank #2 at the 1 st floor
					- Relevant rotation of the cross beam under the vertical Tank #2 and warping of the web of fin plate D-FP #1
					- Strengthening of the cross-beam underneath Tank #2 in correspondence of D-FP #1
RND #17	Random	DBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	DBE - OP	37%	0.23 g	- Overall elastic linear behaviour
TH #8-100%	Seismic	SSE - SD	100%	0.64 g	- D-FP #2,5,6 fin plate experienced cracking
TH #8-110%	Seismic	SSE - NC	110%	0.71 g	- Cracking on previously detail D-FP #2,5,6 significantly increased
					- Failure of bracing supporting the conveyor
RND #18	Random	DBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour

2.4 System Identification and Observations

In this section, the dynamic identification of the SPIF structure and relevant observations on the experimental campaign are discussed.

2.4.1 Identification of frequencies, mode shapes and damping ratios

The transition from raw data into systems' performance meaningful information, see Subsection 3.5.2, requires the adoption of proper system identification (SI) methods [42]. Within the framework of SPIF, two approaches have been followed: i) the adoption of the PSD of recorded accelerometers' signals, to estimate accurate frequencies' shifts and rapid checks on the structural behaviour of the mock-up along the experimental campaign; ii) the use of rational fraction polynomials (RFP) applied to frequency response functions (FRFs) in order to achieve a more reliable estimate of frequencies, mode shapes and, especially, damping ratios.

In order to reduce interference from unwanted noise due to power lines or power supply ripple, a finite impulse response (FIR) low-pass filter with cutoff frequency of 30 Hz was applied to recorded signals. In particular, the FIR filter uses a least-squares approximation to compute the filter coefficients and then smooths the impulse response with a Hamming window. Then, for each acquired time-history of accelerometers both fast Fourier Transform (FFT) and power spectrum density (PSD) based on the Welch's method have been performed. In particular, the discrete PSD P_{yy} reads,

$$P_{yy}[i] = \begin{cases} \frac{Y[i] \cdot Y[i]^*}{NG \cdot F_s/N}, & \text{if } i = 0 \\ \frac{2 \cdot Y[i] \cdot Y[i]^*}{NG \cdot F_s/N}, & \text{otherwise} \end{cases} \quad (2.1)$$

where, to obtain a clear spectral representation of the time series $x[i]$, the Hamming window $w[i]$ has been used as follows,

$$Y[i] = \frac{1}{N} \text{FFT}\{x[i] \cdot w[i]\} \quad (2.2)$$

In Eq. 2.1 * stands for transpose, $Y[i]$ has been firstly normalized by the noise gain, $NG = \frac{1}{N} \sum_{i=0}^{N-1} w[i]^2$ and, finally, divided by the sampling frequency, F_s , over the total length N of the signal.

Results of SI either along the MRF direction, i.e. the shaking direction, and the orthogonal BF direction are reported in Figure 2.14, for both Random #13 and Random #18, listed in Table 3.4.

The response of the structure along the MRF direction, subjected to random inputs of PGA intensity in the range 0.05g -0.15g, allows to reliably estimate the first frequency around 2.9 Hz and other frequencies at 6.0 Hz, 9.0 Hz as other main frequencies. There is also a peak around 5.0 Hz, especially pronounced with FFT's signals recorded at the 3rd floor.

Instead, for the same random excitation along the BF direction, peaks near 5.2 Hz and 7.2 Hz have been identified. Obviously, they exhibited a minor amplitude in comparison with the MRF direction; this can be observed from the PSD amplitudes of Figure 2.14.

Table 2.5: Identified frequencies from Random signals through PSDs analyses along the MRF direction. Also $\Delta[\%]$ variations between current and initial frequencies are reported.

MRF Direction								
	$f_{n1}[Hz]$	$\Delta[\%]$	$f_{n2}[Hz]$	$\Delta[\%]$	$f_{n3}[Hz]$	$\Delta[\%]$	$f_{n4}[Hz]$	$\Delta[\%]$
Random #13	2.91	-	5.20	-	6.33	-	8.73	-
Random #15	2.87	-1.37	5.17	-0.58	6.41	1.26	8.87	1.60
Random #16	2.88	-1.03	5.13	-1.35	6.36	0.47	8.82	1.03
Random #17	2.86	-0.35	4.99	-3.48	6.33	-1.25	8.62	-2.82
Random #18	2.84	-2.41	4.68	-10.00	6.38	0.79	8.57	-1.83

Table 2.6: Identified frequencies from Random signals through PSDs analyses along the BF direction. Also $\Delta[\%]$ variations between current and initial frequencies are reported.

BF Direction								
	$f_{n1}[Hz]$	$\Delta[\%]$	$f_{n2}[Hz]$	$\Delta[\%]$	$f_{n3}[Hz]$	$\Delta[\%]$	$f_{n4}[Hz]$	$\Delta[\%]$
Random #13	5.21	-	7.12	-	-	-	14.48	-
Random #15	5.01	-3.84	7.10	-0.28	-	-	14.45	-0.21
Random #16	4.94	-5.18	7.13	0.14	-	-	14.38	-0.69
Random #17	4.95	-1.20	7.14	0.56	10.12	-	13.52	-6.44
Random #18	4.75	-8.83	6.38	-10.39	9.69	-4.25	13.30	-8.15

The resulting identified frequencies for all random signals are collected in Tab. 2.5. A careful reader can notice that till 70% of TH #8 the overall system exhibits a linear response; therefore, the identified frequencies gathered in Tab. 2.5 remain almost unchanged. Nonetheless, it is worth noticing that Random #18 is the only random signal adopted after TH #8 at 110% and, therefore, some damage happened in the fin plate connection D-FP #1 between a primary and a secondary beam depicted in Fig. 2.9. As a result, a reduction in frequency is evident in Table 2.6 in terms of percentage.

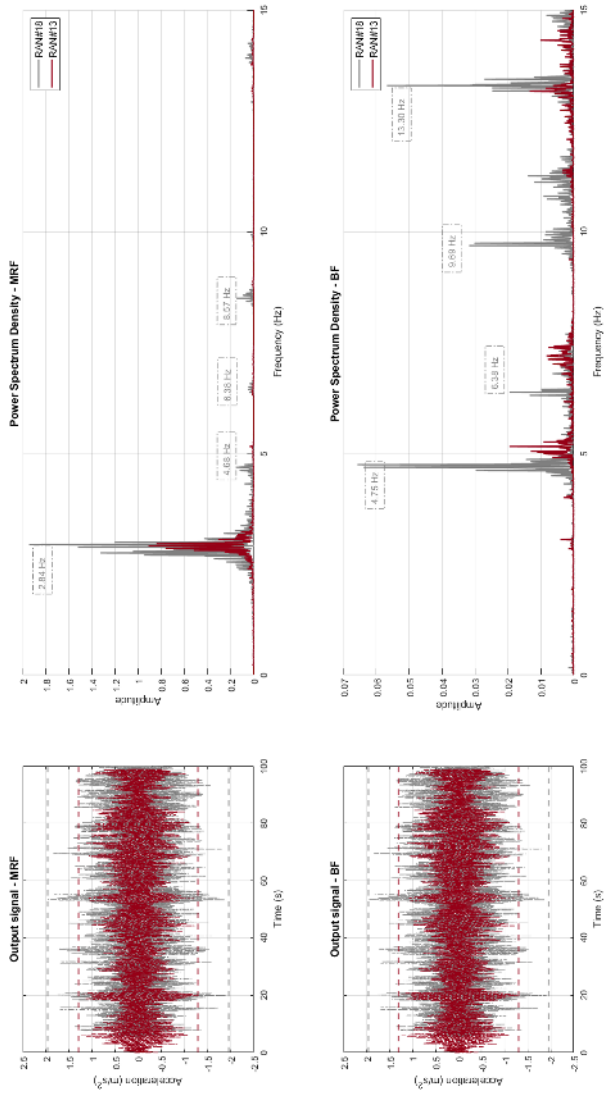


Figure 2.14: Acquired signal and PSDs either on MRF and BF direction for Random #13 - excitation at the beginning of the test programme - and Random #18 - excitation at the end of the test campaign -, respectively: a shift to the left of frequencies especially those in the BF direction is evident.

As mentioned above, data have also been processed by means of the RFP, [43]. This technique overcomes many of the numerical analysis

problems associated to parameter estimation of structures, such as damping ratio estimation, high computational burden for processing, etc.; moreover, it allows for a precise identification of poles, zeros and resonances. Firstly, we compute the matrix of frequency response functions (FRFs) obtained from excitation signals of the shaking table and the corresponding response signals of the whole structure, processed at a sampling rate of 256 Hz. Then, in order to define system's transfer function we perform the ratios between input and outputs in the frequency domain. Therefore, we assume that the response to the input applied at location q is measured at location p , and thus, the related FRF can be expressed as follows,

$$H_{pq}(\omega) = \frac{Y_p(\omega)}{F_q(\omega)} = \frac{\sum_{k=0}^n \beta_k (i\omega)^k}{\sum_{h=0}^m \alpha_h (i\omega)^k} \quad (2.3)$$

For a generic single input-multiple output (SIMO) case, Eqn. 3.1 can be expressed in matrix form in the unknown α_h and β_k terms as,

$$\sum_{h=0}^m \left([\alpha_h] (i\omega)^k \right) \cdot [H(\omega)] = \sum_{k=0}^n \left([\beta_k] (i\omega)^k \right) \quad (2.4)$$

Therefore, by curve fitting the aforementioned analytical form to measurement data, the unknown coefficients ($\alpha_h, h = 0, \dots, m$) and ($\beta_k, k = 0, \dots, n$) of both the numerator and denominator polynomials can be determined in a least-squared-error sense, [43].

Successively, it was possible to evaluate damped frequencies and mode shapes. As demonstrated by several authors [44]-[45]-[46], indeed, the denominator of the RFP holds the information about the poles or eigenvalues of the structure, whilst the numerator holds the information about the mode shapes or eigenvectors of the dynamic system.

More precisely, the poles $\lambda_r = \sigma_r + i\omega_{d,r}$, that hold the information about damped frequencies $f_{d,r} = \omega_{d,r}/2\pi$, are obtained as roots of the denominator polynomial; the mode shapes instead are provided by the numerator. Moreover, damping ratios are calculated as $\xi_r = -\text{Re}(\lambda_r)/|\lambda_r|$, where r denotes the relevant mode shape number. For the sake of brevity, Tab. 2.7 summarizes the main results in terms of eigenvalues -frequencies - whilst damping ratios and some mode shapes are reported in Fig. 2.15.

A careful reader can observe a slight decrease of frequency values in the DBE range, i.e. from 2.88 Hz to 2.86 Hz for f_{n1} ; a frequency jump from the DBE to the SSE range, i.e. from 2.86 Hz to 2.76 Hz always for

Table 2.7: Identified frequencies by the RFP's signal analysis on the TH #8 excitation both for the MRF and BF directions. The horizontal line separates OBE from SSE seismic inputs.

	MRF Direction			BF Direction		
	$f_{n1}[Hz]$	$f_{n2}[Hz]$	$f_{n3}[Hz]$	$f_{n1}[Hz]$	$f_{n2}[Hz]$	$f_{n3}[Hz]$
TH #8-25%	2.88	6.42	9.17	4.24	6.38	9.33
TH #8-37%	2.88	6.38	9.19	4.11	6.30	9.21
TH #8-50%	2.86	6.38	9.06	4.07	6.30	9.17
TH #8-37%	2.86	6.36	8.94	4.06	6.22	9.15
TH #8-70%	2.85	6.35	8.83	4.03	6.32	9.12
TH #8-37%	2.86	6.55	8.84	4.05	6.29	9.15
TH #8-100%	2.76	6.39	8.87	4.01	6.26	9.16
TH #8-110%	2.75	6.20	8.90	4.01	6.27	9.13

f_{n1} . Moreover, the high stiffness values of the BF system w.r.t. the MRF system is evident in the f_{n1} values.

To be thorough, the coherence function $C_{xy}(\omega)$ has been evaluated as

$$C_{xy}(\omega) = \frac{|P_{xy}(\omega)|^2}{P_{xx}(\omega) \cdot P_{yy}(\omega)} \quad (2.5)$$

where P_{xx} and P_{yy} denote power spectral densities of the input, x and output y signals, while P_{xy} denotes the cross power spectral density of x and y . In particular, the coherence represents a useful tool to discriminate whether detected peaks correspond to real frequencies or to artefacts of disturbances. As it can be observed in Fig. 2.16, the coherence function is close to unity for frequency values collected in Tables 2.5 and 2.7, while tends to zero for artefact or mismatched frequencies. Moreover, based on magnitude and phase of FRFs, mode shapes of the tested structure have been derived and reported in Fig. 2.15. In particular, mode shapes are illustrated for frequencies $f_n = 2.88 - 4.24 - 6.38 - 9.17$ Hz. In addition to expected mode shapes for MRF and BF structures, a certain coupling of the aforementioned systems can be noticed at $f_n = 8.95$ Hz.

Finally, based on gathered data from Tables 2.5-2.7, it may be argued that the PSD approach allows for a reliable detection of frequency variations and a rapid check of system's performance during experimental phases. Conversely, the RFP method exhibits a better performance w.r.t. PSD; in fact, it allows for frequency shift detection, a more accurate peak amplitude evaluation and, especially, mode shape and damping estimations.

2.4.2 System performance

Main observations and comments on the structural response of both primary systems and process units subjected to the test programme listed in Table 3.4 are summarized herein. From Tab. 3.4, the main structure and the process components behaved elastically. The elastic regime is confirmed by sensors and visual inspection of the mock-up that resulted in no damage detection both in the primary structure and secondary elements.

Main structure response and damage

The seismic response of the MRF frame can be appreciated in terms of both floor accelerations and displacements for each run of the test programme. From Fig. 2.17(b)(a) a careful reader can observe the high values of accelerations experienced by the MRF frame; moreover, no residual displacements in sensors' data were recorded, see Fig. 2.17(b)(b), in agreement with the aforementioned observed elastic regime of the MRF frame. Also strain values at base columns highlighted in Fig. 2.18 and located as depicted in Fig. 2.9, confirm the elastic behaviour of the main structure up to TH #8-110%; both SGs #1 and #8 associated to the most stressed column, showed indeed strain levels quite far from the yielding condition, characterized by a strain value of 1.7‰.

In addition, by the data acquired from markers located in Fig. 2.11, the motion of the whole structural system was reconstructed. With regard to the main structure, it is worthy to show in Fig. 2.23, the diaphragm floor displacement for each of the three storeys due to the last seismic run TH #8-110%. One can observe that the MRF frame experiences a limited torsional behaviour and each floor practically keeps its original shape. Although the overall behaviour of the primary steel structure appears rather linear from the data recorded by the sensors, during the test with TH #8-70% large displacements of Tank #2 of about 25 mm caused additional rocking on the supporting crossbeams. As a consequence, the interaction between the main beams of the MRF frame and the crossbeams caused damage to the fin plate joint FP #1. The location of the FP #1 on the first floor is reported in Fig. 2.9 and building details are shown in Fig. 2.3c. More precisely, the FP #1 fin plate connection experienced warping of crossbeam's web and cracks in the transition zone from beam to web. After the run TH #8-70%, in order to avoid collapse due to the strong interaction at higher PGA levels, the crossbeam underneath Tank #2 was strengthened by a bolted tee stub as depicted in Figure 2.19. The crack initiation, propagation and the entailing warping was then more consistent in the subsequent runs with higher PGA-levels also on other crossbeams, i.e. fin plate FP #3, FP #4 and FP #6 labelled in Fig. 2.9.

The last test carried out with the record TH #8-110% entailed additional damage on fin plate connections, especially in connections FP #2 and FP #5, see Fig. 2.20(c), underneath both vertical tanks.

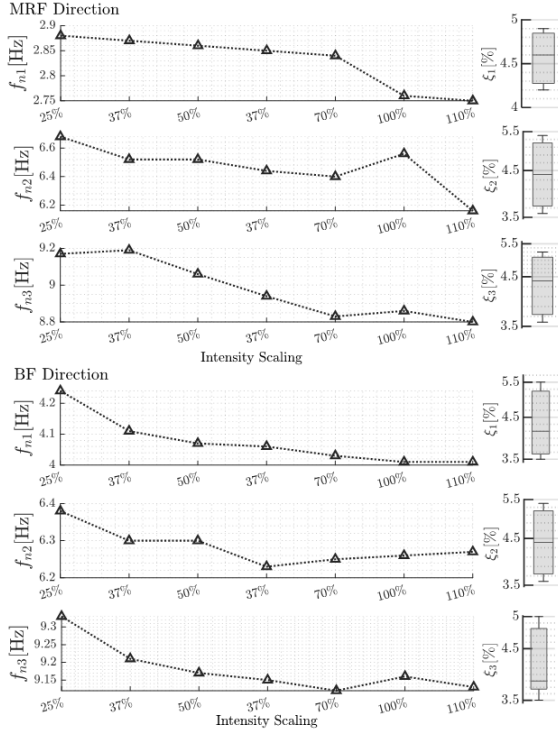
Secondary components system response and damage

Secondary elements seismic response has been analyzed in terms of both accelerations and displacements for each run of the test programme. As anticipated, both vertical and horizontal tanks were monitored. In particular, we focused our attention on vertical Tank #2 -with sensor A #12- and horizontal Tank #4 - with sensor A #18-. In this respect, both absolute acceleration and relative displacement at the top of Tank #2 - between markers #36 and #5- and Tank #4 -between markers #53 and #42- subjected to the record TH #8-110% are shown in Fig. 2.24a. One can observe that accelerations on the vertical tank are similar to the ones of the horizontal tank, even though the horizontal tank is located on a second floor that has higher floor accelerations than the first level. Figure 2.24b depicts the relative displacement of Tanks #2 and #4 for PGA level of 110%. The relative displacement is measured by the marker at the top of the vertical tanks and at the middle height of the horizontal ones. The results show that vertical tank on the first level exhibit almost four times higher displacements in comparison to the horizontal tank on the second level. Owing to significant rocking, we focused our attention on the vertical Tank #2 located in the first floor and depicted in Fig. 2.9(b). It experienced a horizontal displacement of 19 mm along the MRF direction during the run T-H #8 110%. Although the Tank #2 exhibited significant rocking at this level of seismic excitation, limited damage was observed at the tank base. This experimental evidence confirms some statements of Section 1 about the overconservatism of current standards for secondary elements, i.e. process equipment, in industrial facilities. Concerning the piping system, it is worth noticing the collapse of one suspended pipe system at the third floor, see Fig. 2.22, while performing the tuning phase of the record TH #8-37%. Furthermore, in order to evaluate both maximum values of flange opening and possible leakage detection, measures from SGs and LVDTs were recorded. Relevant maximum values are reported in Tab. 2.8. Measured values entail no leakage or LoC at flanged joints; moreover, standard pressure transducers confirm this conclusion. In addition, the conveyor located at the base suffered damage, see Fig. 2.21, due to failure of the bracing system as indicated in Fig. 2.21. Fig. 2.25a shows the acceleration time history of the cabinet for the record TH #8-110%. The cabinet and the connection remained undamaged as the mass of 140 kg is relatively low. The cabinet experienced 26.5 m/s^2 , higher than the accelerations of the

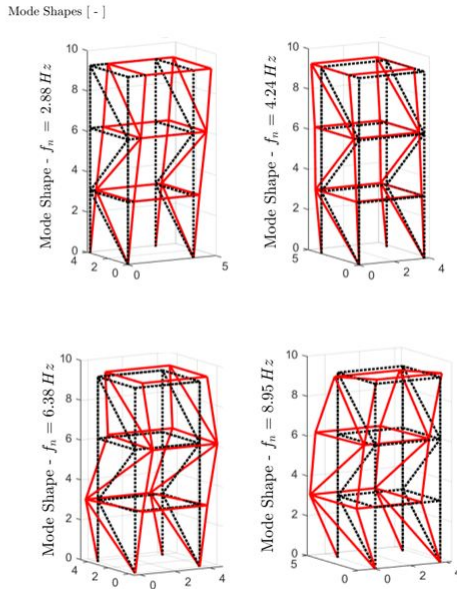
tanks on first floor, as the cabinet exhibits a high stiffness and low damping values. The dominant frequency at the highest amplitude defines the dominant frequency of the floor of 2.88 Hz, which corresponds to the first eigenfrequency of the primary structure. Moreover, eigenfrequencies in the range of 4.7 to 6 Hz shown in Fig. 2.15 correspond to the relevant frequencies of the overall system.

Table 2.8: Piping system w.r.t. Fig. 2.10 and response

Pipe position	Max Strain (‰)	Max opening (mm)
# 1	0.3	0.02
# 4	0.1	0.03
# 5	0.4	0.03
# 6	0.2	0.051



(a)



(b)

Figure 2.15: a) Relevant frequency variations for the mock-up. Damping ratio box-plots are reported with whiskers referring to the 10th and 90th-quantile; b) mode shapes relevant to frequencies $f_n = 2.88 - 4.24 - 6.38 - 9.17 \text{ Hz}$ of Table 2.7.

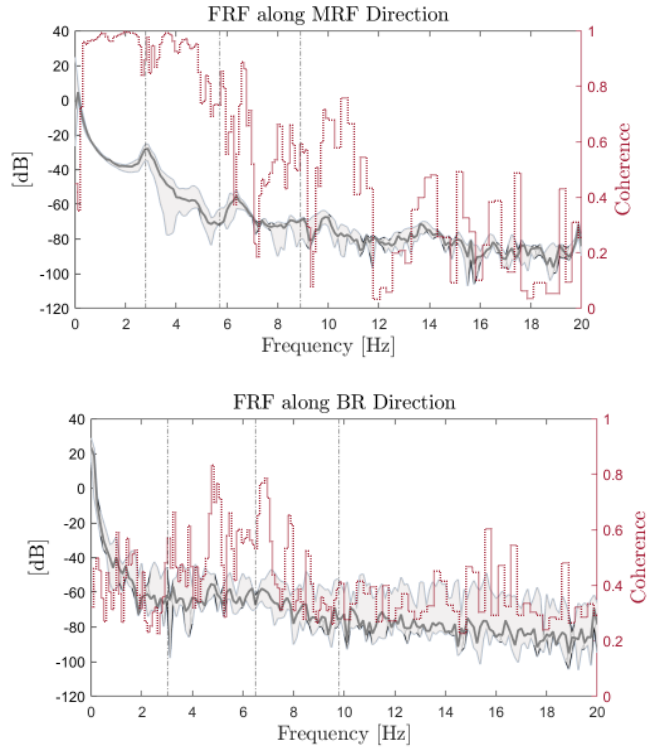
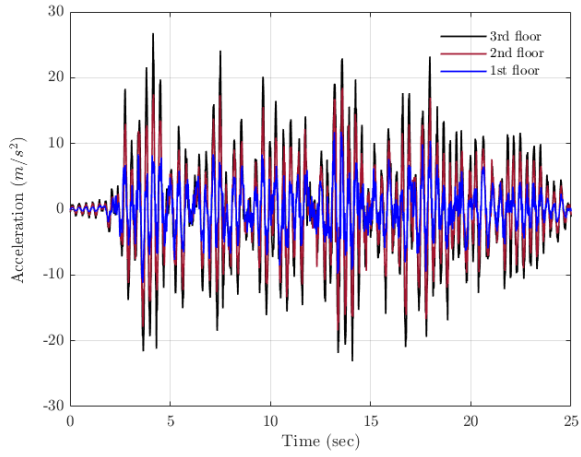
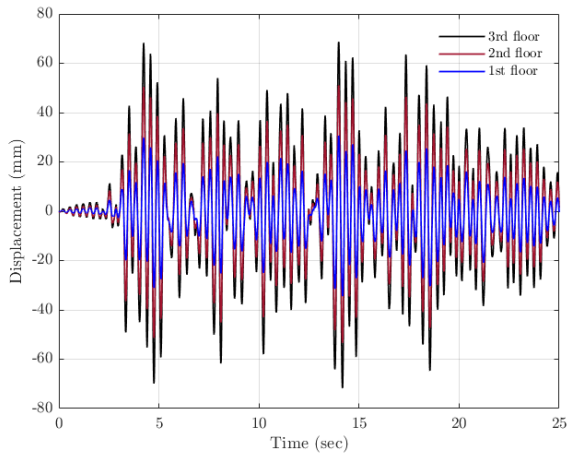


Figure 2.16: Frequency response functions and coherence $C_{xy}(\omega)$ for the record TH #8-25% both for the MRF direction and the BF direction.

(a) Time history vs. accelerations



(b) Time history vs. displacements



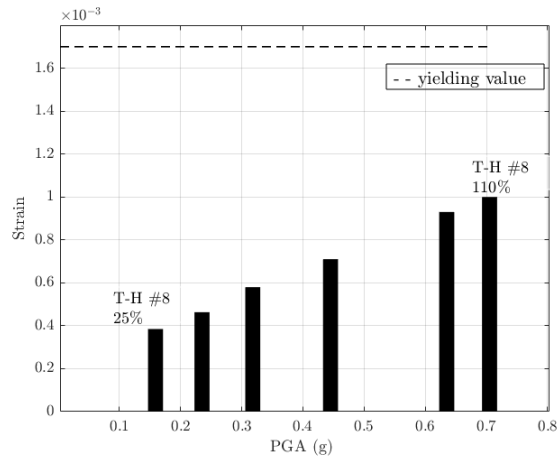


Figure 2.18: Strain values of base columns vs. PGA recorded by strain gauges for different seismic records



Figure 2.19: Additional bolted tee stub used to connect the crossbeam to the main beam.

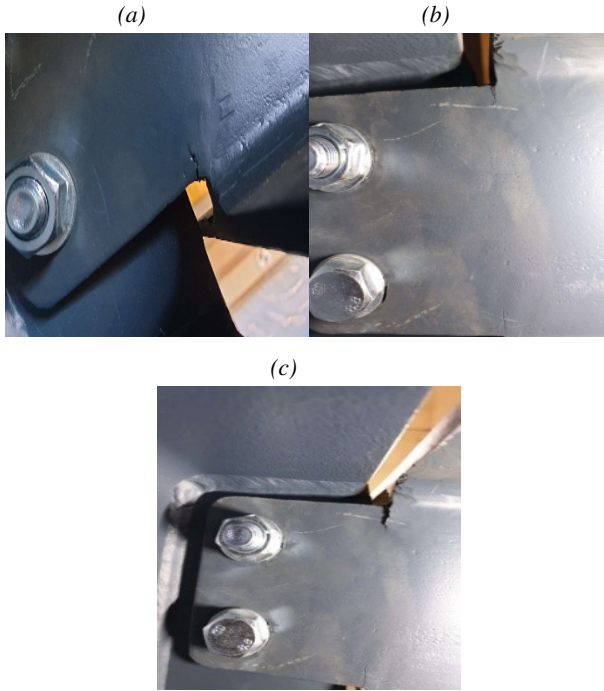


Figure 2.20: Cracks and warping of fin plate connections between primary MRF beam and secondary crossbeams at the first floor: (a) Cracking and warping of fin plate FP #1 during TH #8-70% ; (b) Damage of fin plate FP #2 during TH #8-100%; (c) Cracks and warping of fin plate FP #2 during TH #8-110%.



Figure 2.21: Collapse of the supporting system of the conveyor at base level for record TH #8-110% .

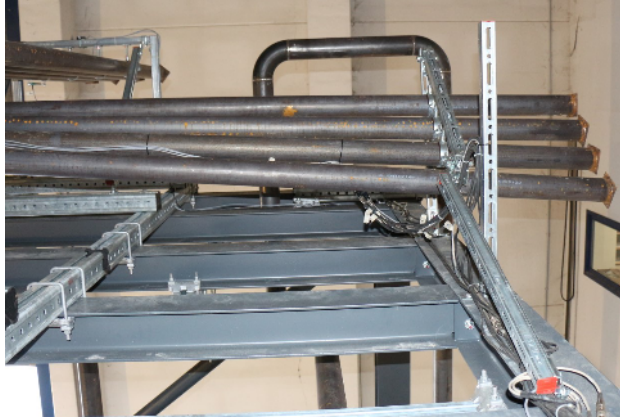


Figure 2.22: Collapse of the pipe rack at the third floor.

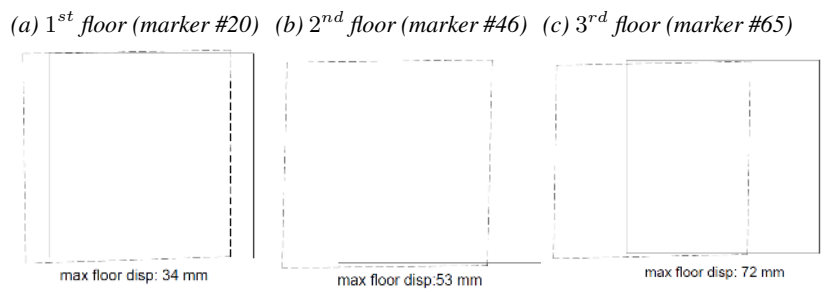


Figure 2.23: Floors motion recorded by means of marker sensors for the record TH #8-110%

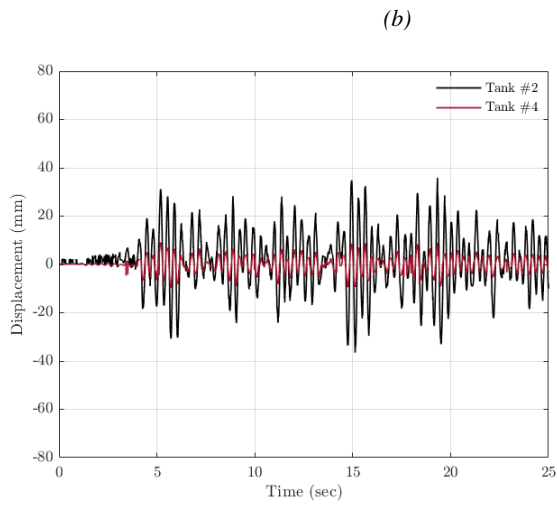
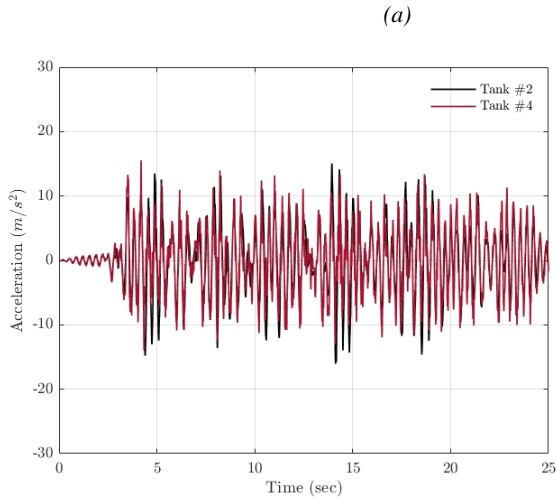


Figure 2.24: Comparison of time histories for Tank #2 and Tank #4 subjected to the record TH #8-110%: a) acceleration; b) relative displacement

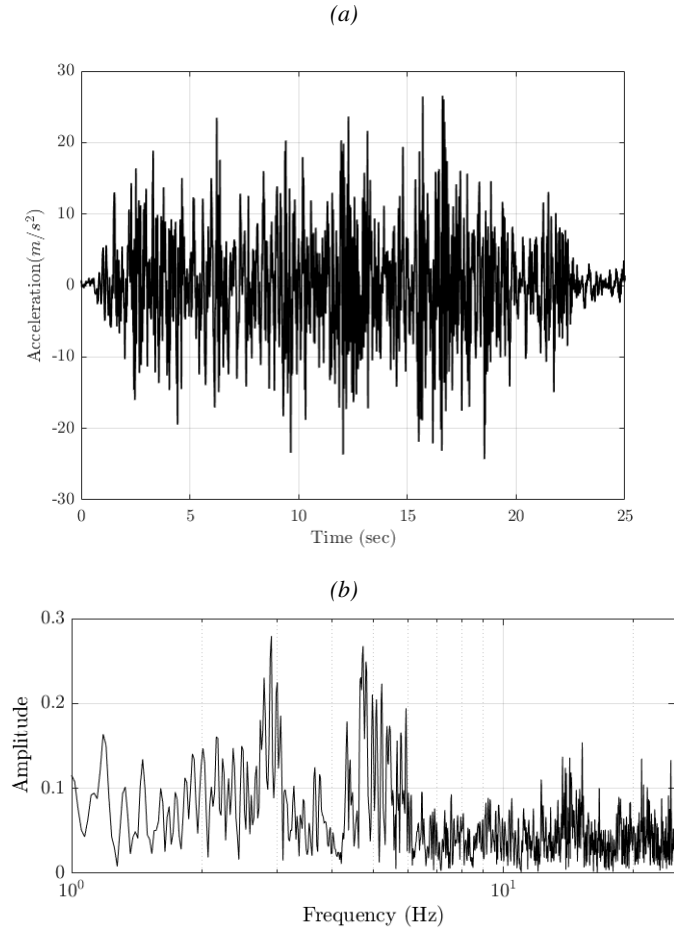


Figure 2.25: (a) Acceleration time history of the sensor A #13 of the cabinet at Level 1 for the record TH #8-110% and (b) relevant frequency spectrum

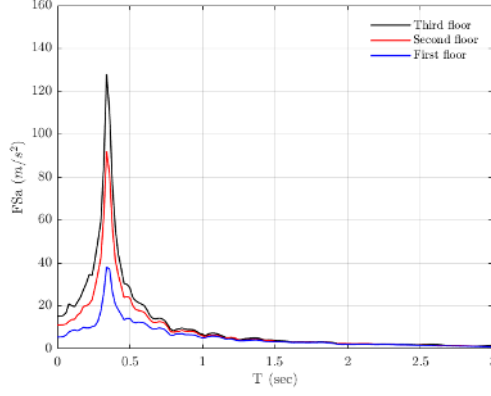
2.5 Data Analysis and Insights

The test results presented in Subsection 3.5.2 are analyzed in depth herein, completing observations provided in Subsection 3.1.1.

2.5.1 Floor responses and comparisons with literature and standards

It worthwhile to underline that the dynamic interaction between primary structure and process equipment cannot be ignored in seismic analysis when designing process or secondary elements. In this respect, EN-1998 [41] does not provide any criterion to identify thresholds for which dynamic interaction can be neglected, ASCE 7-16 adopts the so-called 25% mass rule [22]. In the present scenario, this rule is clearly not satisfied and, thus, dynamic interaction between primary and secondary components should be explicitly accounted for. Moreover, according to Merino et al. [7] the use of EN-1998 [41] is rather conservative. In order to analyze this issue in depth, FRS at first and second floor have been directly derived by experimental data. In this respect, the signals recorded by accelerometers at the base of Tank #1 and #3 located at the first and the second floor, see Fig. 2.9, for two different time histories, i.e. TH #8-50% -PGA=0.31 g- and TH #8-110% -PGA 0.71 g-, were used. Floor acceleration spectra for each of the three floors and a damping ratio of $\xi = 4.5\%$ are depicted in Fig. 2.26, where the damping value derives from experimental data treated in Subsection 2.4.1. A careful reader can notice that the dominant frequency is quite close to the natural frequency of the steel support structure, i.e. 0.34 sec from Table 2.5, which does not change both with floor and PGA level. With regard to the amplitude, the high spectral values at the first floor, i.e. 1.5 m/s^2 denote a predominant linear behaviour of the primary structure. Besides EN1998-1 [41] and ASCE 7-16 [22], several authors proposed refined approaches for the prediction of FRS. In this respect, it is worthwhile to recall the methods proposed by Calvi and Sullivan [47] and Vukobratovic and Fajfar [48]. In particular, the first approach is based on a dynamic amplification coefficient for secondary components as a function of the primary/secondary period ratio and the damping ratio of non-structural components. The authors also accounted for the nonlinear behaviour of the primary structure by using an equivalent period as a function of the kinematic ductility. The second approach instead, combines spectral accelerations of the primary and secondary system subjected to the ground acceleration by means of the well-known SRSS rule. Moreover, this method can account for the nonlinear behaviour of the primary structure. In addition, an importance factor $I_p = 1.5$ was used for ASCE 7-16 with a response

(a) Record TH #8-50%



(b) Record TH #8-110%

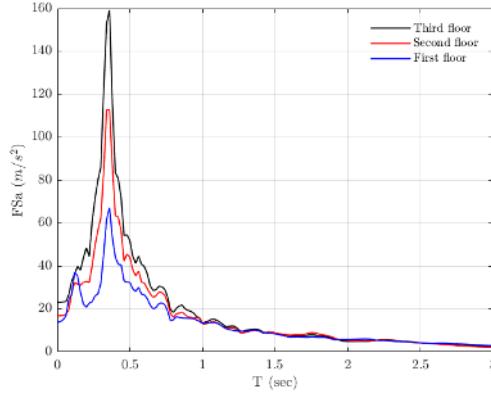
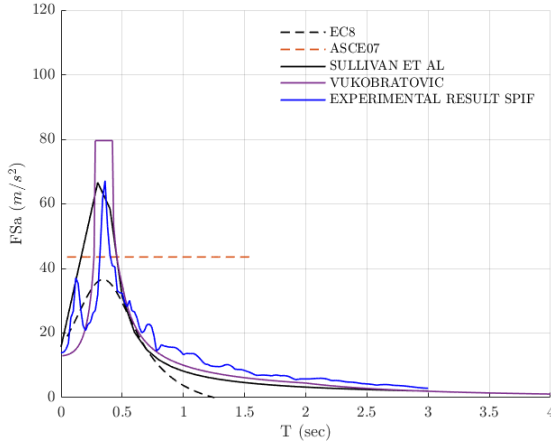


Figure 2.26: Experimental floor response spectra for TH #8-50% and TH #8-110% seismic excitations

modification factor $R_p = 2.5$ and a factor $q = 1.5$ for EN-1998. For the latter a component amplification factor equal to 2.5 has also been assumed. Relevant results are depicted in Figure 2.27 and the reader can observe a general underestimation of amplitude values at resonance conditions for both EN-1998 and ASCE 7-16 standards, in line with the findings of [7]. Conversely, the good prediction capability at resonance of the method proposed by [47] is clearly confirmed. Nonetheless, one can deduce from Table 2.9, where both Tank #1 and #3 are considered, that the aforementioned method cannot accurately predict the FRS ordinate at the natural frequency of tanks. Differently, the method proposed by [48] appears more reliable in both the prediction of spectral shape, see Fig. 2.26, and the estimation of floor spectral accelerations relative to tanks,

i.e. Table 2.9. Also in this case, a ductility value of 1.5 has been assumed to be consistent with the standard EN1998-1. Design rules offered by

(a) First floor spectra for the Record TH #8-110%



(b) Second floor spectra for the Record TH #8-110%

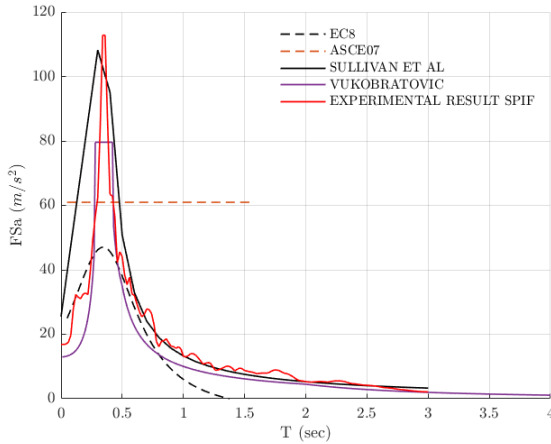


Figure 2.27: Comparison between analytical and experimental floor response spectra for the Record TH #8-110%

the European [41] and American standards [22] for secondary elements have been widely discussed in literature [49], [33]. More precisely, it was demonstrated that the prediction capacity of code-compliant formulations appear rather limited. For instance, Merino et al. [7], observed that in case of storage tanks, the EN-1998 approach wrongly estimates acceleration demands over a wide range of vibration periods. A general underestimation close to the resonance condition is also observed;

whereas a significant overestimation is present in other period ranges. Similar under/over estimations have been observed with the application of ASCE 7-16, where the dependency from the secondary/primary period ratio is not included.

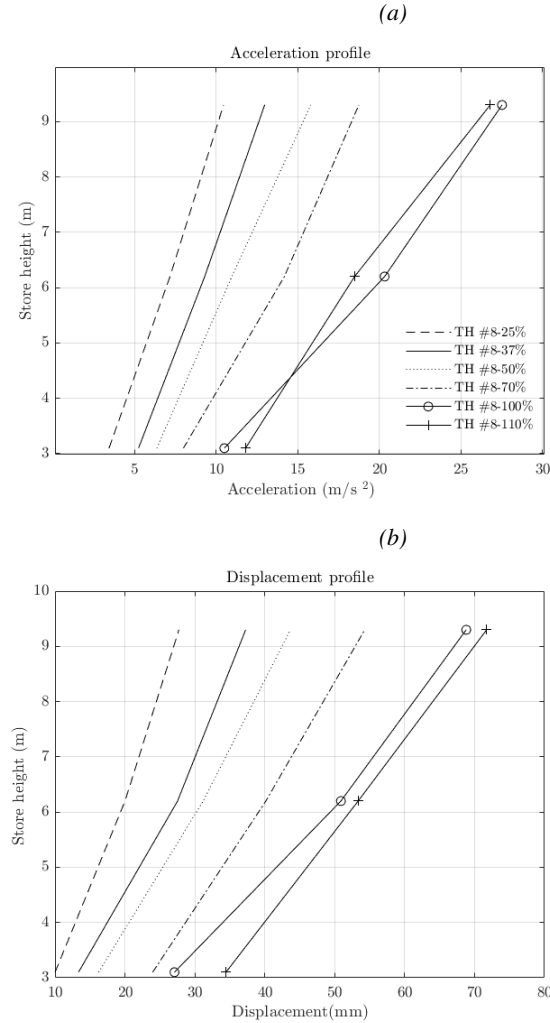


Figure 2.28: Acceleration and displacement profiles for different THs

The main reasons behind the inaccurate estimation of both codes are related to: i) the absence of damping ratio for floor spectra estimation; ii) a general overestimation of the behaviour factor [50] and the component response modification factor for secondary elements. These considerations are reflected in the acceleration value predictions of Table 2.9 where

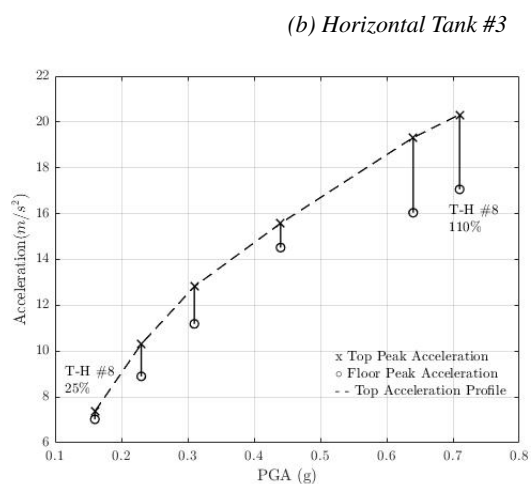
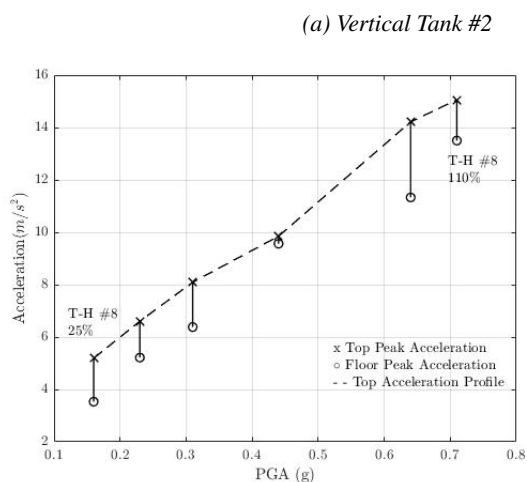


Figure 2.29: Acceleration profiles of Tank #2 and Tank #3 for different PGA levels of TH #8.

all floor accelerations were evaluated at the fundamental period of Tank #1 - 0.11s- and Tank #3 - 0.08s-, respectively. The overconservatism of predicted code values is evident.

2.5.2 Structural regularity considerations

As far as in plan regularity is concerned, current standard [41], [22] classify the primary structure as irregular because of the absence of rigid floors. This feature can also be investigated from the analysis of both acceleration and displacement profiles of the primary structure, as illustrated in Figure 2.28(a) and 2.28(b). The distribution of the kine-

matic responses along the height for increasing seismic intensity, clearly demonstrates a regularity, even in the absence of rigid floors and with unequal mass distribution. As a result any damage concentration can be excluded. The exception is only represented by the case characterized by the seismic record TH #8-100% . The reason is probably due to the local damage occurred during the previous test, i.e. due to the record TH #-70%. As already observed in Subsection 3.5.2, during the seismic response, a relevant sectional rotation of the crossbeam under vertical Tank #2 located at the first floor, caused connection web warping and generated important cracks, as depicted in Fig. 2.20. Eventually, we can argue that the structure exhibited a general regular behaviour during testing.

Along this vein, in order to study the acceleration profile and the amplification factors, also the structural regularity of secondary elements like tanks, has been investigated. In this respect, Fig. 2.29, depicts the peak acceleration both at the base and at the top of the vertical Tank #2 and the horizontal Tank #3, see Fig. 2.9, for each run of the test programme. The top acceleration increase appears rather linear with the PGA level growth and the amplification of the base acceleration is almost constant for each run, except for the run T-H #8 70%, where damage of the crossbeam occurred. We can argue that the linear behaviour of the secondary elements agrees with that of the main structure.

Table 2.9: Comparison of floor response spectra ordinates at the fundamental period of Tanks #1 and #3

PGA	Tank	Experimental acceleration (g)	FRS acceleration (m/s ²)	EN 1998-1 acceleration (m/s ²)	ASCE-7 acceleration (m/s ²)	Sullivan et al. (m/s ²)	Vukobratovic and Fajfar (m/s ²)
0.31	#						
	1	7.68	10.96	10	20	22	7
	3	12.82	14.31	27	27	40	13
0.71	1	15.06	30.10	22	43	32	14
	3	18.48	25.10	25	60	53	17

2.6 Conclusions and future developments

The SPIF project -Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities- was proposed within the framework of the European H2020 - SERA (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe) funding scheme -. It focuses on the investigation of the seismic behaviour of a representative industrial moment resisting frame substructure equipped with process secondary components by means of shaking table tests. The test campaign was comprehensive and carried out at the EUCENTRE Laboratory of Pavia, Italy. A full scale mock-up composed by two single-bay moment resisting frames with three floors, total height 9.3 m, was built and tested using a unidirectional shaking table with several earthquake levels. In this respect, a spectrum-compatible record, based on an EN1998-1 [41] spectrum was selected. The testing campaign demonstrated a dynamic interaction between the primary steel structure and secondary process units that clearly influenced the performance of the whole system. More precisely, the primary steel structure remained undamaged also for SSE earthquake levels mainly due to its design and inherent flexibility that avoided a strong interaction with secondary components. Some damage instead, appeared in web plates of supported crossbeams in the area of fin plate connections between main members and secondary beams; that happened due to the interaction between primary and secondary components, that justified a light seismic retrofit intervention in that area by means of a bolted tee stub. Moreover, results clearly showed a general overconservatism of current design standards with respect to floor response spectra prediction of process units evaluated at their fundamental period, due to the lack of consideration of damping and overestimation of behaviour/reduction factors.

Overall, it was shown that a proper experimental test campaign can highlight complex interactions and potential damage locations present in industrial plant substructures. The efficiency of other steel configurations, e.g. a braced frame configuration, also in view of proper seismic retrofit interventions deserves further studies.

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3. Experimental performance of multi-storey BF frame structure with process equipment subjected to site-based ground motions

This chapter contains the post-print of

Nardin, C., Bursi, O.S., Paolacci, F., Pavese, A., Quinci, G. (2022) Experimental performance of a multi-storey braced frame structure with non-structural industrial components subjected to synthetic ground motions. *Earthquake Engng Struct Dyn.*; 51. <https://doi.org/10.1002/eqe.3656>

as published in the Journal of *Engineering Structures*, differing from the published paper only in terms of layout and formatting.

Author contributions. **C. Nardin:** Formal analysis, Software, Data Curation, Writing - Original Draft. **O.s. Bursi:** Supervision, Writing – Review, Funding Acquisition. **F. Paolacci:** Supervision, Funding Acquisition. **A. Pavese:** Head of laboratory. **G. Quinci:** Formal analysis, Data Curation.

Summary

The seismic risk assessment of industrial facilities mainly relies on historical data and design of uncoupled secondary components. Accordingly, the dynamic interaction between primary structures and process equipment is overlooked. The SPIF project -Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities- was carried out to respond to this gap, within the European H2020 SERA framework. Its objective regarded the investigation of the seismic behaviour of an archetype industrial multi-storey steel moment resisting frame (MRF) structure equipped with non-structural components (NSCs) by means of shaking table tests. The goal of the proposed study was to extend the interaction analysis between a primary multi-storey braced frame (BF) steel structure and NSCs in a performance-based earthquake engineering (PBEE) perspective.

To excite the vibration periods of the NSCs and thus, enhance possible coupling with the primary structure, a synthetic site-based ground motion model (GMM) was employed. More precisely, the proposed research intended: i) to severely excite the process equipment and supplement the scarcity of real records with a specific frequency content by means of a stochastic GMM; ii) to quantify seismic-induced force and displacement demands of secondary components and their effects on the primary BF structure. The evaluation of the experimental data clearly shows buckling in the bracing system of the BF configuration and a strong interaction between vertical tanks and floor crossbeams of the BF.

3.1 Introduction

3.1.1 Background and Motivation

Past destructive earthquakes in China (Sichuan, 2008 and Yushu, 2010), Japan (Tohoku, 2011) and Italy (Emilia, 2012) have emphasized social and political attention on seismic risk arising from process plants. Both observations of damage following major/moderate seismic events [1] and numerical/experimental studies [2] have reaffirmed that petrochemical facilities are particularly and even disproportionately vulnerable to earthquakes. Besides, several researchers investigated numerous industrial accidents that have resulted in severe loss of life and injury, damage to natural and built environment as well as significant economic losses. As a matter of facts due to earthquake damage, it emerges that non-structural components (NSCs) and support systems interactions account for the majority of direct property losses [3]. In this respect, to collect and document the body of available knowledge related to the seismic performance of NSCs, the National Institute of Standards and Technology (NIST) invested on a year-long study summarized in [4]. From these studies, it clearly emerges: (i) a lack of definition of performance objectives for NSCs in a performance-based design methodologies perspective; (ii) a gap in performing fully comprehensive testing campaigns and investigations on coupling effects between primary structures and NSCs; (iii) a need to improve and enforce code requirements along with the development of reliable NSCs seismic demands models.

In recognition of (i), i.e. the absence of comprehensive non-structural performance objectives, NEHRP Provisions [5] proposed a framework for non-structural performance targets by delineating two categories of NSCs: (a) those not required to operate following a design basis earth-

quake (DBE); (b) and those expected to function following a DBE, defined in [6]. Despite their great importance, however seismic provisions for petrochemical facilities are based on classical load-and-resistance factor design (LRFD), like EN 1998-1 [7], VCI-Guideline [8], or on allowable stress design (ASD), like ASME B31.1 [9] and EN 13480 [10]; a performance-based earthquake engineering (PBEE) approach should, instead, be preferred. Nevertheless, it is noteworthy that whilst several aspects in terms of design and analysis are still unresolved, code-compliant methods demonstrated to result in very poor predictions and often dated and inadequate, as highlighted in [11] - [12] - [13].

With regard to the (ii) issue, also [14] underlined a definite lack of investigation as well as protocols for testing the interaction between primary structures and NSCs under seismic loading. As regards the analysis of global systems based on PBEE, only a few studies can be found [14] and [15]. For instance, the application of the PBEE to two realistic sub-structures of petrochemical piping systems showed a general overconservatism in piping system design. Conversely, some European research project, i.e. INDUSE-2-SAFETY 2012-2015, clearly focused on experimental testing of NSCs and performance levels to be compliant with. To develop a better understanding of the elasto-plastic response and ultimate strength of nuclear power piping systems, other agencies carried out a multi-year test programme [16]. As a result, in view of the enhancement of structural integrity of industrial, energy and nuclear components, design guidelines and recommendations were issued [6] - [17].

In this respect, also a recent European project was proposed: the SPIF project -*Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities*. More precisely, the objective of the SPIF project [18], was the investigation of the seismic behaviour of a representative industrial multi-storey frame structure equipped with complex process components by means of shaking table tests designed in a PBEE perspective. The testing campaign highlighted the dynamic interaction between the primary steel structure and secondary process units that clearly influenced the performance of the whole system.

On this latter aspect and strictly related to PBEE, the development of involved non-linear analysis methods for an accurate evaluation of both limit states and damage represents another salient issue [19]. Step-by-step dynamic integration of a structural system, where explicit nonlinear behaviour of both primary and secondary components are considered, is clearly the most reliable analysis method. Nonetheless, simplified approaches such as the non-linear static analysis or the use of floor response spectra (FRS) are generally preferred. Along the same vein, floor acceleration/displacement spectra represent a viable and promising approach

which, however, have been applied to a lesser extent to the case of industrial structures, see [11]. For instance, Merino Vela et al. [11] focused on the development of floor acceleration spectra for an industrial concentrically braced frame supporting a liquid storage tank. The authors demonstrated that in most of the cases, code-compliant analytical approaches overpredict responses w.r.t. analyses that explicitly take into account the interaction between a supporting structure and a liquid storage tank. Moreover, recent studies, as in [20] - [21] - [12], investigated the main factors that mostly influence the amplification or decrease of FRS values: inelasticity in the primary structure; location of the NSC in the supporting structure; periods of the component and main structure; involved damping ratio of the component. As results, improved equivalent-static equations were proposed for designing acceleration-sensitive NSCs, see [4] - [20]. Besides, De Biasio et al. in [12] selected intensity measures (IMs) capable of predicting NSCs acceleration demands and proposed a new IM, namely, the equipment relative average spectral acceleration ($E - ASA_R$). On this respect, to guarantee the IMs efficiency and sufficiency, a large and potentially expansible seismic dataset is required.

In addition to or in place of recorded motions, in recent years there has been growing interest in earthquake ground motion models and in methods capable of generating synthetic ground motions, which can be used in PBEE. One method to generate synthetic motions relies on a site-based stochastic ground motion model (GMM), which directly describes the ground motion time-series recorded at a site. Recent examples of site-based stochastic ground motion models include the non-stationary filtered white-noise model for far-field ground motions [22]-[23], a wavelet-based model [24], and a multimodal non-stationary spectral model [25]. All these models account for both temporal and spectral nonstationarity, which is an important characteristic of earthquake ground motions. Moreover, predictive relations for the model parameters were developed in terms of parameters describing the earthquake source and site characteristics. The predictive relations and stochastic model can then be used together to generate synthetic ground motions for any set of earthquake source and site characteristics of interest.

Inside this framework, the results of the SPIF project focused on an archetype MRF endowed with process equipment [26], [18], clearly showed how a proper experimental test campaign can highlight complex interactions between primary structures and NSCs. Besides, some left open issues aroused: (i) the test of the efficiency of other structural configurations as BFs; (ii) the selection of a seismic input capable of highlighting the coupling between primary structure and secondary elements. These are the issues that the paper explores further.

3.1.2 Scope

Along this vein, the SPIF #2 project was conceived and realized by means of a shaking table test programme in a PBEE perspective. Thus, the present paper proposes limit states and performance objectives for a primary BF structure and NSCs; then, it presents the main results of the experimental campaign performed with a shaking table; successively, it compares experimental EDPs with reference values; and, eventually, it explains the interaction between the primary BF structure and the NSCs. The paper comprises 6 Sections; Section 2 provides a briefly overview of the SPIF project and its main findings. Then, Section 3 provides both limit states and performance objectives for a primary BF structure and NSCs; and deals with the design of the SPIF's mock-up for the BF configuration. Section 4 instead, presents the synthetic seismic signal employed and derived from a GMM selection and the shaking table test programme. More precisely, tests characterised by synthetic accelerogram levels ranging from $0.22g$ to $0.79g$ PGA, have been carried out. In Section 5, acquired data is assessed and system identification of modal frequencies, mode shapes and damping ratios is performed. Insights of the experimental data campaign are also provided with regard to seismic-induced force and displacement demands of secondary components, their effects on the primary structure and, a comparison between experimental EDPs and reference values. Finally, main conclusions and outlooks are drawn in Section 6.

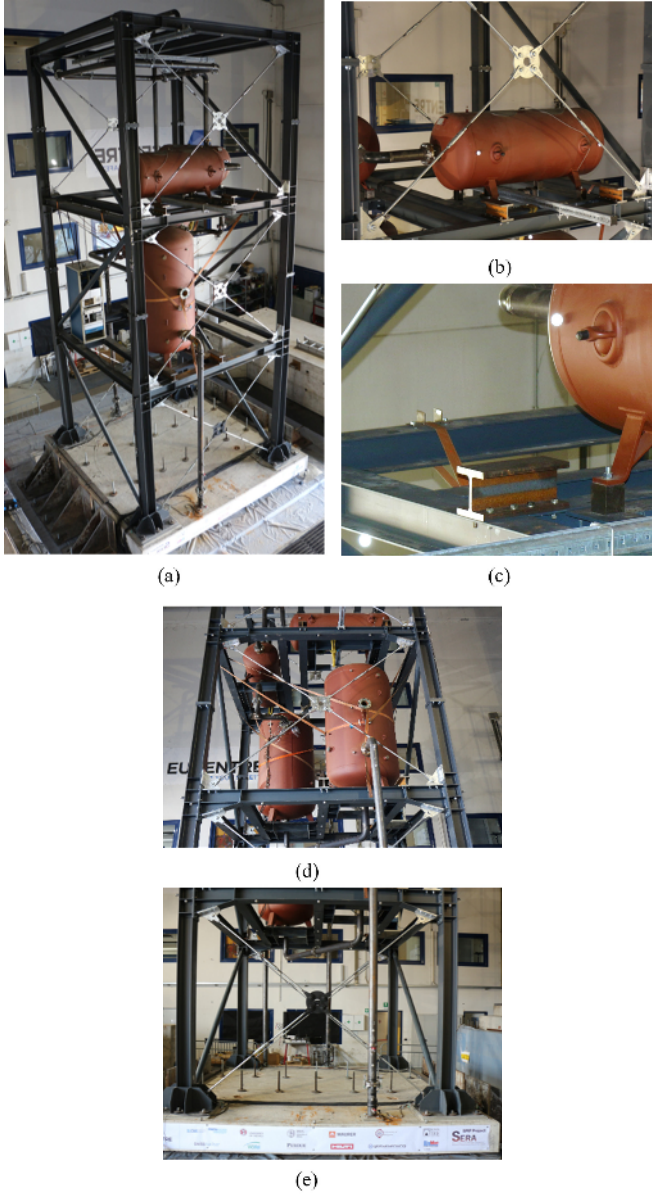


Figure 3.1: The SPIF #2 mock-up: (a) global view of the BF configuration; (b)-(c) bracing system and T-stub connections of the intervention; (d)-(e) front view of the 1st and 2nd floor at the EUCENTRE Laboratory.

3.2 The Project SPIF: Learned Lessons from a multi-storey Moment Resisting Frame

The previous SPIF project, [18]-[26], focused on the investigation of the seismic behaviour of an archetype three-storey single-bay moment resisting frame (MRF) industrial substructure, tested by a uniaxial shaking table with several PGA's levels of a scaled spectrum-compatible record [7] described in Subsection 3.2.2. More precisely, the objective of the SPIF project was the investigation in depth of the dynamic interaction between the MRF steel structure and secondary process components -that influence the performance of the whole system in a performance-based perspective- as well as a proper check of floor spectra predictions. Both the case study considered with the relevant seismic input and the main results are presented herein.

3.2.1 The Case Study

The original full scale mock-up consists of a primary steel structure supporting horizontal and vertical tanks, piping installations and a cabinet. Figure 3.1 gathers both photographs of the SPIF mock-up at the EUCENTRE Laboratory. Further information, construction details and greater insights on the SPIF campaign can be found in [18]-[26].

Briefly, the mock-up was conceived as a three-storey steel grade *S355* structure with flexible diaphragms made of crossbeams hinged to the frame beams. The dimensions of the designed structure are $3.7m \times 3.7m$ in plan with a storey height of $3.1m$.

The horizontal load bearing system of the steel structure consisted of two moment resisting frames (MRFs) in the direction of the seismic excitation and two braced frames in the transverse direction equipped with tension/compression bracings with circular cross section. These latter were used in order to limit lateral movements and torsional effects. The two MRFs are connected through two braced frames (BFs) and crossbeams, which are fastened to the frame beams by simple bolted connections with web stiffeners; some photo of the beam-to-beam connection can be observed in Figure 3.3a. These crossbeams served in turn as bearing supports for the installation of the NSC, i.e. the secondary elements.

These secondary elements of the prototype industrial structure included tanks, pipes, elbows, bolted flange joints (BFJs), tee-pipe joints, a conveyor and a cabinet. More precisely, four unpressurized tanks made of steel *S235* – JRG2 with a yield strength of 235 N/mm^2 were installed as indicated in Figure 3.4b: i) two vertical tanks on the first floor, i.e. Tank #1 and #2; ii) and two horizontal tanks on the second floor, i.e. Tank #3 and #4, respectively. Their location reflects typical configura-

tions in industrial installations. To simulate the liquid typically stored in such tanks, they were filled with granular material with a density equal to water. This allowed for the protection of the shaking table and technical installations against liquid release. The aforementioned tanks were mutually connected to each other through a piping system consisting of nine $DN100$ pipes, with a $100mm$ diameter and a $3.6mm$ thickness. Some pipes were also connected to the concrete slab. The relevant pipe layout consists of straight branches, elbows and tee pipe joints, as can be observed in Figure 3.1a-d. The material used for the pipes was $P235$ with a yield strength of 235 N/mm^2 . In order to monitor the leakage detection, four branches of pipes (Pos.#1, #3, #4, #6) were filled with water and pressurized. Moreover, a suspended pipe rack was also installed on the third floor, endowed with $DN80$ pipes.

3.2.2 Seismic Inputs

Since the primary structure was designed to cope with the *Near Collapse* (NC) limit state, a severe earthquake prone-scenario was assumed leading to a design PGA of $0.69g$. Within a PBEE perspective, the mock-up was thus subjected to several shaking table tests, characterised by spectrum-compatible accelerogram levels ranging from $0.16g$ to $0.71g$ PGA. In agreement with EN 1998-1 [7], these scaled accelerograms were based on a linear elastic response spectrum far field Type 1, with soil condition C, a $PGA=0.69g$ and a damping ratio of 5%. The relevant spectrum is labelled as EN 1998-1 in Figure 3.2a with a black line. In addition, the seismic input assigned to the MRF configuration of the SPIF mock-up is named $TH\ #8$ with a gray line in [26]. For the sake of completeness, the seismic input employed in the BF configuration of the SPIF #2 mock-up (red line) is labelled $GM7121$, and is described in depth in Subsection 3.4.1. Besides, time histories of acceleration of the assigned seismic input relevant to both the SPIF mock-up MRF and the SPIF #2 mock-up BF configuration, are shown in Figure 3.2b. In order to intensify potential coupling between the NSCs and the main structure, the $GM7121$ signal was selected and adopted in the BF configuration of the SPIF #2 experimental campaign. A careful reader can notice that $GM7121$ exhibits higher spectral intensities than the $TH\ #8$ spectrum in the range period of the primary structure and NSCs, i.e. between $0.05 - 0.25s$.

3.2.3 Main Results

Testing results definitely demonstrated a dynamic interaction between the primary MRF steel structure and secondary process units that clearly influenced the performance of the whole system. More precisely, the pri-

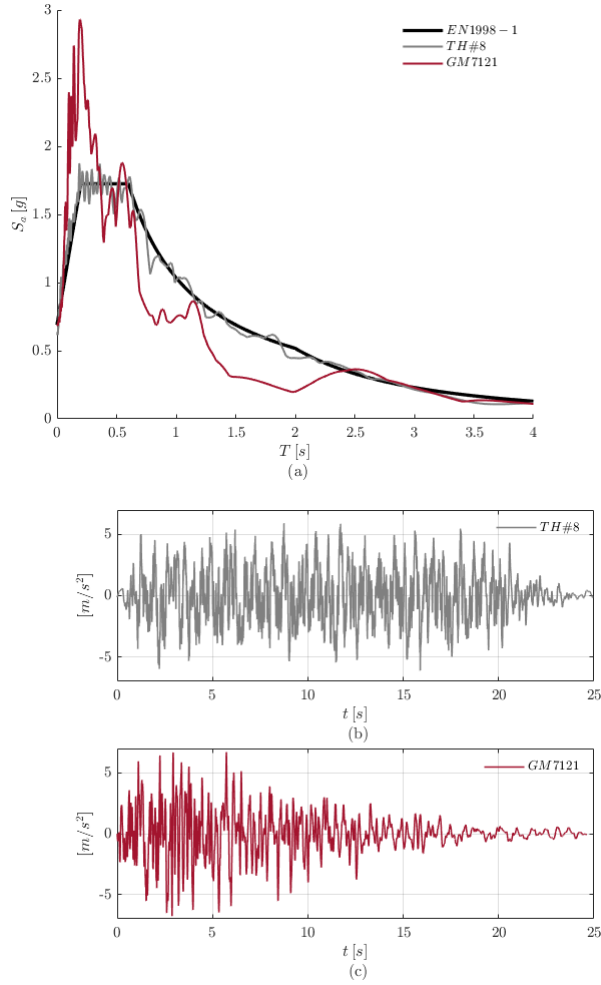


Figure 3.2: (a) Acceleration spectra according to: EN 1998-1, black line; TH #8, gray line, used in the SPIF MRF testing campaign; GM7121, red line, adopted in the SPIF #2 BF testing campaign. (b)-(c) Relevant acceleration time histories of TH #8 and GM7121, respectively.

mary steel structure remained undamaged also for SSE earthquake levels mainly due to its design and inherent flexibility. In fact, the main steel structure characterized by a period of 0.36 s , enabled a filtering effect on the NSCs, whose periods, instead, ranged between $0.10 - 0.16\text{ s}$. As a result, a strong interaction between the primary structure and the secondary components was avoided.

However, due to the use of common practice solutions for NSC components, inevitably some damage appeared in web plates of fin plate connections between main members and secondary beams. In fact, due to the strong rocking of the vertical tanks #1 and #2 during testing, the fin plate connection FP #1 shown in Figure 3.3a-b, clearly experienced warping of crossbeam's web and cracks in the transition zone from beam to web.

Moreover, in order to avoid further damage of the fin plate connection FP #1 and continue the test program at higher PGA levels, the crossbeam underneath Tank #2 was strengthened by means of a bolted T-stub depicted in Figure 3.3c. As a result in the subsequent runs at higher PGA-levels, crack initiation, propagation and subsequent warping happened also in FP connections of other crossbeams underneath the vertical tanks.

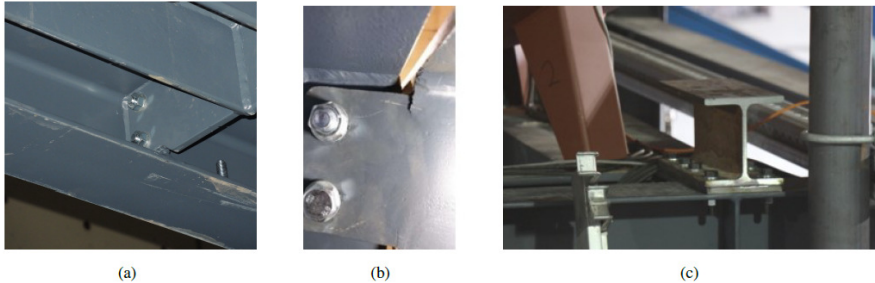


Figure 3.3: (a) Installed fin plate connections FP #1; (b) warping and crack propagation in the web of the crossbeam under Tank #2; and (c) strengthening intervention.

3.3 The Project SPIF #2: Performance of a multi-storey braced frame

3.3.1 Rationale

Some open left issues of the previous SPIF test campaign aroused. More precisely: i) the efficiency of other steel structure plant configurations; ii) the deepening of the interaction between the primary structure and the NSCs. Therefore, to answer to these questions, the MRF was converted and tested with a braced frame (BF) configuration. As a result, a bracing system was conceived and designed as primary structure. As a result, we designed the bracing system following the EN 1998-1 [7] strength-based approach. Consequently, the natural frequencies of the heaviest tanks at the first floor, of about $f_{tank} = 6.25$ Hz and of the coupled braced structure became close, as shown in Figure 3.9(c)b. In addition, to emphasize the response of the coupled system at that frequency, we selected a proper seismic input with a significant frequency content in the range of interests of both the heavy tanks at the first floor and the BF. Due to damage of fin plate connections cited in Subsection 3.2.3, bolted T-stub connections were designed and added on secondary beams. For the sake of clarity, Figure 3.1 collects photos of the new configuration BF mock-up. More precisely, the overall view of the installed bracing system can be seen in Figure 3.1a, whilst Figure 3.1d and Figure 3.1e collect front views at the 1st and 2nd floor level. Besides, detail connections of the bracing system with the primary steel structure can be observed in Figure 3.1b; while, in Figure 3.1c, the strengthening intervention on secondary beams *via* bolted T-stub elements can be appreciated.

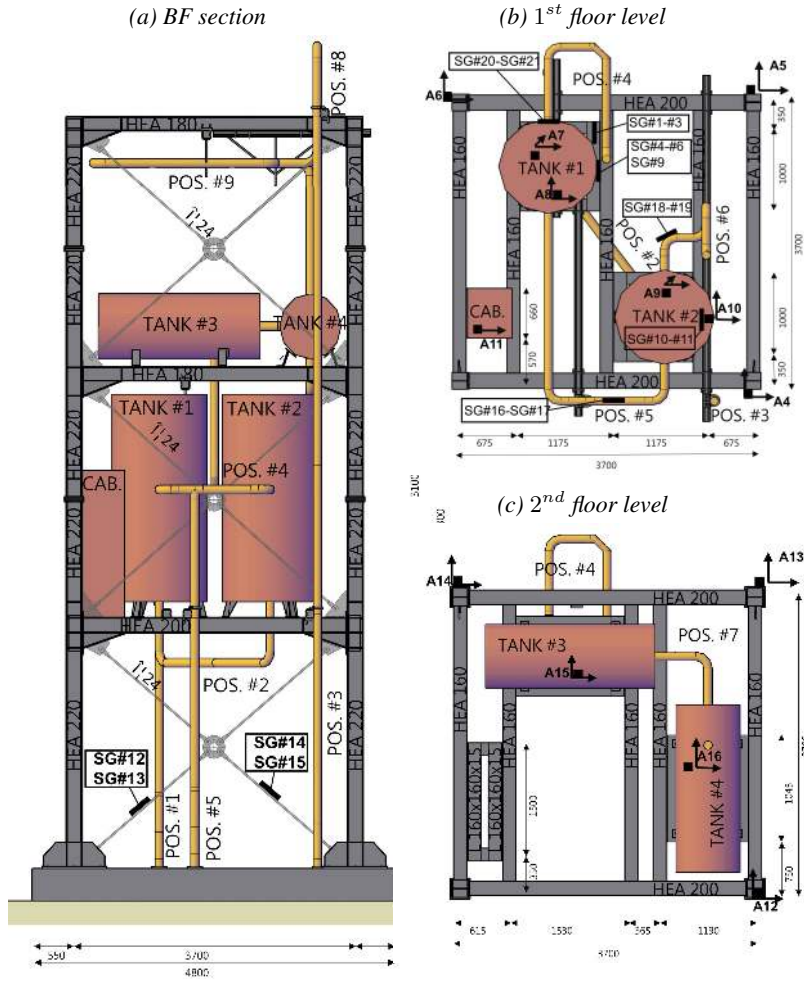


Figure 3.4: (a) Lateral view of the mock-up; (b) and (c) steel member layout, non structural components and instrumentation.

As regards the adopted bracing system, it is endowed with circular sections of 24mm diameter and with a non-dimensional slenderness $\bar{\lambda} = 1.44$. As such, it was designed in compliance with the EN 1998-1 [7] and the spectrum depicted in Figure 3.2a, following the strength-based approach. For the sake of clarity, Figure 3.4a reports the BF configuration section, where both the bracing system and the NSCs can be clearly observed; while Figure 3.4b-c depict the arrangement of the steel members and of the piping system layout of the first and second storey level, respectively. The seismic excitation GM7121 depicted in Figure 3.2c was applied along the aforementioned direction. Moreover,

both the sensing equipment and the relevant nomenclature of interests are depicted too. As a result, the first period of the BF structure is equal to $T_1 \simeq 0.20s$ which results to be significantly lower than that of the MRF $T_1 \simeq 0.35s$. As anticipated, to enhance coupling effects between the primary structure and NSCs, the aforementioned *ad-hoc* seismic input GM7121 was selected. Therefore, severe floor acceleration spectra in the range periods of the equipment, i.e. between $0.05 - 0.20s$ were induced. In particular, the heavy vertical tanks at the first floor, see Figure 3.4, whose periods approach $0.16s$ have been excited.

3.3.2 Performance based design and reference limit states

According to the PBEE approach, the probability of exceeding specific damage states is quantified by using relevant thresholds, i.e. limit states. Four common-practice limit states for civil engineering buildings are typically assumed and correlated to PGA levels: Fully Operational, Damage Limitation, Significant Damage and Near collapse, as defined in [27]. Multi-storey frame structures, like that of Figure 3.1, are typically considered as non-building structures for which the above-mentioned limit states are still considered applicable [27]. Conversely, performance objectives related to NSCs still remain an open issue. Along this vein, [28] defines the performance objectives for NSCs as Fully Operational (FO), Position Retention (PR), Life Safety (LS) and Hazards Reduced (HR) which are related to increasing seismic intensity levels. With regard to NSCs installed in civil engineering structures, [28] prescribes that the FO limit state must be guaranteed for the lowest earthquake level whilst HR is the limit state associated to rare earthquakes. In this case, classical approaches classify NSCs as drift-sensitive and acceleration-sensitive, whose performance are based on global response parameters as interstorey drift or absolute floor acceleration, [4]. On the other hand, the same standard [28] prescribes that the FO limit state must be guaranteed for hazardous material storage also for higher earthquake intensity levels. Consequently, the limit states defined for civil structures have been reconsidered when applied to industrial plants [2], [15]. Several standards, indeed, dealing with seismic analysis of chemical/petrochemical plants, e.g., [27], [29], among others, use the same limit states adopted by nuclear standards [6]. More precisely, they prescribe two distinct thresholds, namely the Design Basis (DBE) and the Safe Shutdown (SSE) earthquakes. DBE is defined as the condition under which facilities remain fully functional without undue risk to health and safety of people and is typically associated to a 10% probability of exceedance in 50 years, i.e. 475 years, [29], and is modified using specific importance factors [7]. The SSE limit state is instead related to a

lower probability of exceedance under which certain relevant structures, systems and important components must be designed to allow for a safe shutdown. The American standard [27] considers NSCs, i.e. tanks, vessels, pipes, etc. as nonbuilding structures for which dedicated standards are recommended.

Karamanos and coworkers [30], proposed performance criteria for liquid storage tanks and piping systems subjected to seismic loading that are useful for a quantitative definition of limit states for NSCs. In agreement with the above framework, both the definition of EDPs and thresholds adopted for primary structures and NSCs are summarized in Table 3.1. Thus, the seismic inputs correspond to DBE and SSE for both primary and NSCs and are linked to a probability of exceeding 10% and 5% in 50% in 50 years, respectively, with an importance factor of 1.5. In particular, for both storage tanks and pipes the thresholds are defined by strain thresholds, whilst the performance of the primary structure is governed in terms of interstorey drift [27]. More precisely, the assumed minimum compressive strain ε_{Cu} for piping reads $\varepsilon_{Cu} = 0.5 \cdot (t/D) - 0.0025 + 3000 \cdot (\sigma_h/E)^2$; provided an internal pressure of 20 bar, see Subsection 3.2.1, it reaches a value of 1.6%.

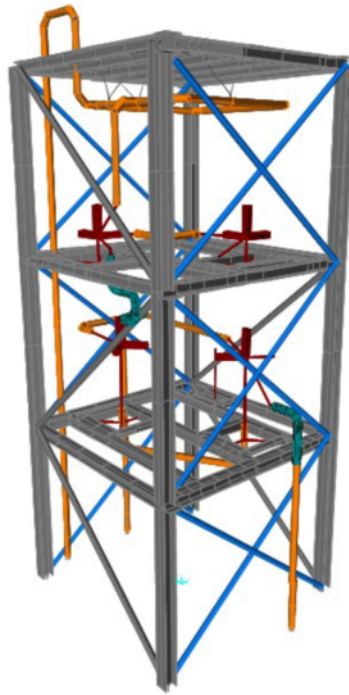
Table 3.1: Limit states and performance objectives for industrial plants.

Limit States	$P_{V/R}$	T_R	PGA	Primary structure			NSC		
				Performance requirement	Performance requirement	Performance requirement	Tank	EDP	Piping
DBE Design Basis	10 %	1200	0.56	All components devoted to normal operations remain functional. Damage to structure is limited. Structures do not experience permanent drift.	Performance threshold	Post-earthquake damage state in which the NSCs remain fully functional.	Performance threshold	EDP	Performance threshold
					EDP	Max tensile local strain	$\varepsilon < \varepsilon_y$	Tensile strain	$\varepsilon_t < \varepsilon_y$
SSE Safe Shutdown	5%	2475	0.69	Minor yielding or buckling of braces. Transient drift that causes minor nonstructural damage. Negligible permanent drift.	Storey drift	Post-earthquake damage state in which NSCs can be damaged, but they allow for a safe closure	$\varepsilon < \varepsilon_y$	Compressive strain	Compressive strain
					Storey drift	Max tensile local strain	$\varepsilon_y < \varepsilon < 0.5\%$	Tensile strain	$\varepsilon_y < \varepsilon_t < 0.5\%$
Safe Shutdown	5%	2475	0.69	Minor yielding or buckling of braces. Transient drift that causes minor nonstructural damage. Negligible permanent drift.	Storey drift	Post-earthquake damage state in which NSCs can be damaged, but they allow for a safe closure	$\varepsilon_y < \varepsilon < 0.5\%$	Compressive strain	Compressive strain
					Storey drift	Max tensile local strain	$\varepsilon_{C_u} = 0.5 \cdot (t/D) - 0.0025 + 3000 \cdot (\sigma_h/E)^2$	Compressive strain	$\varepsilon_y < \varepsilon_c < \varepsilon_{C_u}$

3.3.3 FE model of the primary steel structure

To comply with numerical, physical and control requirements of the experimental test campaign, a low fidelity (LF) FE model of the SPIF mock-up depicted in Figure 3.1 was deemed necessary. In particular, the LF model was initially adopted for the selection of the seismic input amplitude to be assigned to the shaking table; during the experimental campaign, then, the FE model was utilized to monitor and predict the response of the system, updated from run to run via SI. It was used in the post-processing phase, eventually, to better interpret the physical phenomena that governed the dynamic behaviour of the coupled system, e.g. the TMD effects presented in Figure 3.9(c), etc. Thus, the SAP2000 software [31] was considered adequate for the complexity of the test-case structure. Linear elastic elements for the SPIF #2 structure have been generally adopted. Thus, columns and beams have been modeled by means of Euler-Bernoulli elements; floor levels diaphragms have been considered extreme flexible and, therefore, no-rigid constraints for each floor level were applied. In particular, in-plane and out-of-plane stiffness contribution was only provided by the grid location of secondary beams, as depicted in Figure 3.4b-c. Masses of cabinets and tanks were represented by lumped masses applied to *ad-hoc* stick models, discussed in depth in the following Subsection 3.3.4 and reported in Figure 3.5d-e. A special attention was paid to the base joints. More precisely, to reproduce joint stiffness, a rotational link was adopted and calibrated against previously test run of the SPIF project; thus, an initial rotational stiffness $S_{j,ini} = 22.5 \text{ MNm/rad}$ was estimated through the FE Idea StatiCA@software [32]. Following 5.2.2.5(2) of EN 1993-1-8 [33], we have evaluated $S_{threshold} = 109.94 \text{ MNm/rad}$, through Eq. (5.2d) that discriminates between rigid and semirigid joints. As $S_{j,ini}/S_{threshold} = 0.21$, we clearly deal with a semirigid joint. Moreover, the joint can be classified as a partial strength joint.

The Young's modulus was calibrated by means of previous tests and a value of 220 GPa has been assumed. Along this line, thanks to the rational fraction polynomials (RFPs) identification technique applied to the data collected from the previous experimental campaign, a damping ratio with a value of 4.5% was introduced as suggested by European [34] and nuclear standards. Nonlinearities were introduced to model the bracing system through nonlinear tension only elements. Figure 3.5a reports a sketch of the FE model of the BF configuration.



(a) SAP2000 global FE Model

3.3.4 FE models of the piping systems and secondary elements

To reduce the computational burden required by the analysis, the piping system and secondary elements, such as vertical and horizontal tanks, were the object of a thorough parametric analysis. Several techniques are available for modeling both the inherent flexibility and stiffness of piping system layouts, see e.g. [30], that includes different components such as pipes, elbows, tee-joints and bolted flange joints (BFJs).

According to [10], tee-joints and straight pipes were modelled by means of beam elements with reduced stiffness' values through the adoption of flexibility factors. Instead, elbows, highly susceptible to ovalization and warping, were modeled with straight beam elements when it was possible to ensure a sufficient cut-off length. When this condition was not met, they were modeled with shell elements as depicted in Figure 3.5b-c for piping system Position #1 and #6, respectively. Furthermore, BFJs were modelled by means of beam elements with springs at the ends calibrated against previous tests' data [26]. More precisely, following UNI EN1591-1 [35], the axial stiffness of 816 MN/m , the

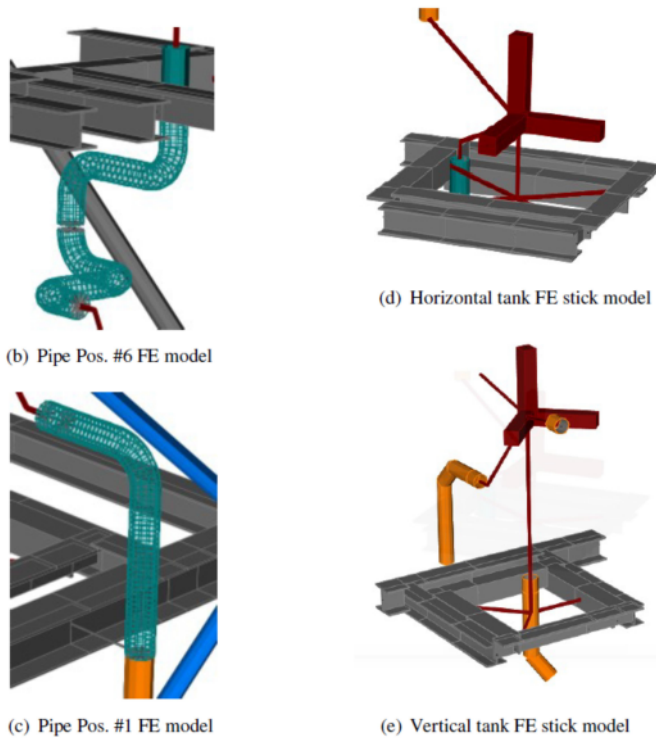


Figure 3.5: The SPIF #2 global FE model with relevant FE models of secondary components.

transversal stiffness of 313 MN/m and the rotational stiffness of 1.98 MNm/rad were estimated. For completeness, we have compared those stiffnesses to the ones of a 1 m length DN100 pipe, as in the monitored Pos.#1 configuration. Relevant ratios with regard to both rotational and shear stiffness entail that the BFJ is a semi-rigid joint; conversely, it is a pinned joint as far as the axial stiffness is concerned.

Moreover, tanks were modelled with simplified stick models, calibrated against previous outcomes of the SPIF campaign and validated through comparison of results from [36]. Following [34], both to consider the global seismic response and seismic action effects on the supporting structure due to the installed tanks, two modelling assumptions were made: (i) the particulate content moves together with the tank shell; (ii) the effective mass to consider is evaluated as 80% of the total mass of the granular content and is applied in correspondence of the tanks' centre of gravity. Therefore, in order to take into account both translational and rotational characteristics of the components, an equivalent

stiffness for each considered degree of freedom has been evaluated and applied separately for each direction, as clearly depicted in Figure 3.5d-e for both horizontal and vertical tanks.

At the very least, the predictive capabilities of the aforementioned LF model have been checked with results stemming from the experimental frequencies and mode shapes. In particular, to perform system identification (SI) in order to calibrate and validate the numerical model, we applied the rational fraction polynomials (RFP) to the experimental frequency response functions (FRFs) provided by seismic inputs.

More specifically, we compute the matrix of FRFs obtained from seismic excitation signals of the shaking table and the corresponding response signals of the whole structure, processed at a sampling rate of 256 Hz . Then, to define the system's transfer function, we perform the ratios between outputs *vs* input in the frequency domain. In particular, we assume that the response to the input applied at location q is measured at location p , and thus, the related FRF reads,

$$H_{pq}(\omega) = \frac{Y_p(\omega)}{F_q(\omega)} = \frac{\sum_{k=0}^n \beta_k (i\omega)^k}{\sum_{h=0}^m \alpha_h (i\omega)^k} \quad (3.1)$$

For a generic single input-multiple output case, Eqn. 3.1 can be expressed in a matrix form in the unknown α_h and β_k terms as,

$$\sum_{h=0}^m \left([\alpha_h] (i\omega)^k \right) \cdot [H(\omega)] = \sum_{k=0}^n \left([\beta_k] (i\omega)^k \right) \quad (3.2)$$

Therefore, the unknown coefficients ($\alpha_h, h = 0, \dots, m$) and ($\beta_k, k = 0, \dots, n$) of both the numerator and denominator polynomials can be determined in a least-squared-error sense, by curve fitting the aforementioned analytical form to measurement data.

As demonstrated by [37], the denominator of the RFP holds the information about poles or eigenvalues of the structure, whilst the numerator embodies the information about the mode shapes or eigenvectors of the dynamic system. Consequently, frequencies, mode shapes and damping ratios of the BF structure were evaluated. Section 3.5.1 collects both results and insights of the SI procedure.

3.4 Input Selection, Shake Table Test Programme and Sensor Layout

This section is particularly important because the performance of both the primary structure and NSC were established by means of several limit states. These states need to be clearly reached and exceeded by specific seismic excitation levels, and in this case, synthetic ground motions. Moreover, the shake table test program that incorporates different levels of input is presented and discussed.

3.4.1 Input Selection

In order to excite NSCs characterized by vibration periods of $T = 0.10 - 0.20$ sec and possibly couple the secondary components to the primary BF structure, the accelerogram TH #8 resulted to be inadequate due to its spectral properties shown in Figure 3.2a. As a result, a synthetic ground motion model (GMM) was employed also with purpose of reproducing the spectral variability of recorded accelerograms. Along these lines, the GMM proposed by [22] was assumed. The stochastic and site-located adopted GMM is based on a modulated and filtered white-noise process that incorporates both temporal and spectral non-stationarities, i.e.,

$$a_g(t) = q(t, \hat{\alpha}) \left[\frac{1}{\sigma_f(t)} \int_{-\infty}^t h(t - \tau, \lambda(\tau)) \omega(\tau) d\tau \right] \quad (3.3)$$

where $\omega(\tau)$ is a white-noise process and $\hat{\alpha}$ is defined by means of

$$\hat{\alpha} = \arg \min_{\alpha} \left(|I_a(t_{45}) - \hat{I}_a(t_{45})| + |(I_a(t_{95}) - \hat{I}_a(t_{95}))| \right) \quad (3.4)$$

More precisely, $q(t, \hat{\alpha})$ represents the modulating function that completely defines the temporal characteristics of the process. In addition, $h(t - \tau, \lambda(\tau)) = f(\omega_f, \zeta_f)$ is the impulse-response function (IRF) whose parameters thoroughly define the spectral characteristics of the process, with $\omega_f = \omega_{mid} + \dot{\omega}_{mid}(t - t_{mid})$ and ζ_f sets the filter damping ratio. With regard to Eqn. 3.4, $I_a()$ and $\hat{I}_a()$, define the reference and the simulated Arias intensity evaluated at 45% and 95% of the Arias intensity, respectively; additional information can be argued from Table 3.3 and [22].

An initial dataset of recorded accelerograms was drawn both by the INGV catalogue¹ and ITACA - Italian Accelerometric Archive Database

¹Istituto Nazionale di Geofisica e Vulcanologia, <http://terremoti.ingv.it/en>, accessed: 2020-08-01.

² - with the following requirements: *i*) fault to site distance $R > 10\text{km}$; *ii*) moment magnitude $M_w > 5.5$; *iii*) main shock seismic events only; *iv*) strong motion intensities expressed in terms of $PGA > 0.075g$.

Besides, to be consistent with the aforementioned TH #8 spectrum-compatible accelerogram employed for the MRF mock-up, the soil condition $V_{S,30} > 600\text{m/s}$ and normal fault mechanism criteria were adopted. Table 3.2 gathers the set of selected natural seismic records, while Figure 3.6 highlights a good variability of the selected records in terms of frequency and intensity content for both the NS and WE components.

Table 3.2: Recorded natural seismic events for the GMM's dataset.

Event Name	Date	M_w [-]	Station	Epicen.Dist. [km]	PGA [g]	$V_{S,30}$ [m/s]
EMSC-20161030-0000029	30/10/2016	6.50	AMT	26.40	0.522	670
EMSC-20161030-0000029	30/10/2016	6.50	CSC	14.90	0.165	698
EMSC-20161030-0000029	30/10/2016	6.50	MSC	39.20	0.092	652
EMSC-20161030-0000029	30/10/2016	6.50	MZ08	26.40	0.527	670
EMSC-20161030-0000029	30/10/2016	6.50	MZ51	25.90	0.947	638
EMSC-20161030-0000029	30/10/2016	6.50	TERO	46.60	0.121	912
EMSC-20160824-0000006	24/08/2016	6.00	CSC	18.30	0.105	698
EMSC-20160824-0000006	24/08/2016	6.00	MSC	21.30	0.104	652
EMSC-20160824-0000006	24/08/2016	6.00	MTR	19.40	0.079	689
EMSC-20160824-0000006	24/08/2016	6.00	TERO	31.50	0.084	912
IT-2009-0121	09/04/2009	5.40	AQP	11.80	0.076	836
IT-2009-0102	07/04/2009	5.50	AQG	14.60	0.133	696
IT-2009-0102	07/04/2009	5.50	AQP	13.20	0.092	836
IT-1998-0103	09/09/1998	5.60	LRG	12.50	0.242	603
IT-1998-0103	09/09/1998	5.60	LRS	18.00	0.162	1024
IT-1980-0012	23/11/1980	6.90	BSC	28.30	0.094	972
IT-1980-0012	23/11/1980	6.90	CLT	18.90	0.171	557

²Italian ACcelerometric Archive, <http://itaca.mi.ingv.it/>, accessed: 2020-08-01.

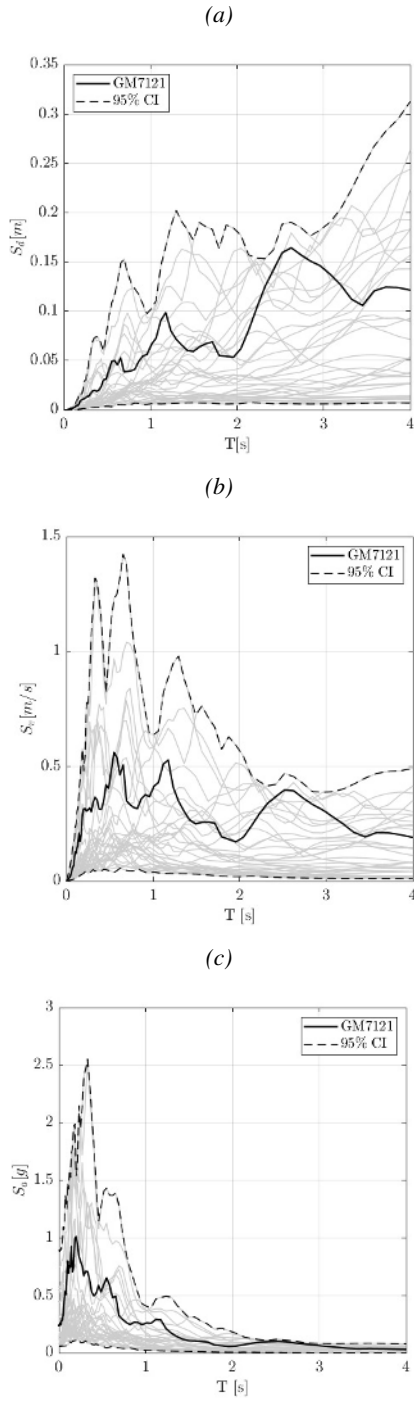


Figure 3.6: Selected records: (a) displacement; (b) velocity; and (c) acceleration response spectra. Grey lines refer to single records whereas black dashed lines indicate the corresponding 95% confidence interval.

In agreement with the aforementioned requirements, a severe seismic prone zone of central Italy was identified and selected in Palmoli (CH), by means of a deaggregation probabilistic seismic hazard analysis (DP-SHA). Owing to its low variability, a fixed value of frequency's change rate $\dot{\omega}_{mid} = -0.5 \text{ rad/s}^2$ was selected. Therefore, in order to calibrate the remaining five GMM parameters, i.e. I_a , D_{5-95} , t_{mid} , ω_{mid} , and ζ_f , against motion characteristics of the selected natural records, the relevant values of probability density distributions which are gathered in Table 3.3 to the dataset collected in Table 3.2 were selected.

In order to set the number of white -baseline- noise processes, 100 values of $\omega(\tau)$ were combined to 100 parameter realizations according to the distributions defined in Table 3.3, so expanded to a total of 10000 synthetic accelerograms. It was observed that 100 values of $\omega(\tau)$ were enough to ensure a convergence of the 75th percentile of $S_a(T_1)$ for $T_1 = 0.18$ sec and 0.36 sec, respectively. Among the ensemble, the records were also checked w.r.t. shaking table limits, frequency content and spectral acceleration in the period's range of interest for the BF structure and its secondary components. Finally, the aforementioned GM7121 record, depicted in Figure 3.2c, was selected and adopted for the test campaign.

3.4.2 Test programme

In the perspective of the PBEE approach discussed in Subsection 3.3.2 and defined in Tables 3.1 and 2.3, respectively, the experimental campaign included different PGA levels for the test structure. Besides, in view of both the tuning of the shaking table and the dynamic identification of the mock-up, the test campaign was further enriched with a series of tests with random excitation and low PGA levels, between 0.05 g and 0.15 g. Thus, the seismic excitation was then scaled w.r.t. the design PGA of 0.69g to 31%, 38%, 62%, 81%, 93%, 100% and 115%, respectively. The corresponding testing campaign and nomenclature is collected in Table 3.4. For instance, the first two runs were conducted with two random excitations, named RND #1 and RND #2. Successively, runs with PGA levels of 0.22g, 0.26g and 0.43g were carried out. Moreover, the most severe test was executed at the maximum value of 0.79g PGA. Eventually, a closing test with the random excitation RND #9 was performed.

3.4.3 Sensor layout

The sensor layout for the SPIF #2 test campaign remained basically unchanged from the first SPIF campaign. Therefore, for the sake of clarity, a careful reader is referred to Subsection 2.2 of [18]. Nonetheless,

both the main sensors and the different locations of the new strain gauges (SG) are depicted in Figure 3.4 and commented herein. Since the sensitivity analysis carried out on the FE SPIF #2 model predicted no uplift mechanism in the column bases, the relative SGs sensors and those installed on the uplift bars were removed. Conversely, w.r.t. the entire piping system, the T-joint in pipe Pos. #5 is endowed with SGs#16-#17, the elbows in pipe Pos. #1 with SG#20-#21, and the BFJ in Pos. #6 with SG#22 - #24 were accurately monitored. Moreover, three SGs sensors - SG#1 to #3- were installed on the support beam under Tank #1. To check the stress levels on the tanks' wall, SGs #4-#5 and #10-#11 were installed on the bottom plate of both Tank #1 and Tank #2. To monitor also the stress level and to check for instability, four SGs were installed on the bracing system at the 1st floor as depicted in Figure 3.4.

Table 3.3: Values of probabilities density distributions of GMM's parameters.

Model Parameters		Units	Distribution	μ	σ	Distribution Bounds
I_a	Arias Intensity	[m/s]	Log-normal	-0.46	0.51	$(0; +\infty)$
D_{5-95}	Time interval of 95% of the I_a	[s]	Log-normal	2.21	0.23	$(0; +\infty)$
t_{mid}	Time interval of 45% of the I_a	[s]	Log-normal	1.698	0.21	$(0; +\infty)$
$\omega_{mid}/2\pi$	Filter frequency at t_{mid}	[Hz]	Uniform	4.8	1	$[3.8; 5.8]$
ζ_f	Filter damping ratio	[-]	Uniform	0.35	0.1	$[0.25; 0.45]$

Table 3.4: Nomenclature. Characteristics and synthetic observations of the SPIF #2 Test Programme.

Label	Excitation	Limit State	Scaling Factor [%]	PGA Level [g]	Observations
RND #1	Random	DBE-OP	-	0.08	- Overall elastic linear behaviour
RND #2	Random	DBE-OP	-	0.08	- Overall elastic linear behaviour
GM7121#31%	Seismic	DBE-OP	31	0.22	- Overall elastic linear behaviour
GM7121#38%	Seismic	DBE-DL	38	0.26	- Overall elastic linear behaviour
GM7121#62%	Seismic	DBE-DL	62	0.43	- Relevant rotation of the cross beam under the vertical Tanks #1-#2 - Buckling of SE bracing system at 1 st floor
RND #3	Random	DBE-OP	-	0.08	- Overall elastic linear behaviour
RND #4	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour
GM7121#81%	Seismic	SSE-SD	81	0.56	- Buckling of bracing system at 1 st floor - Severe stresses on bottom wall of Tank #1-#2
RND #5	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour
GM7121#81%-2	Seismic	SSE-SD	81	0.56	- Severe stresses on bottom wall of Tank #1 - Buckling of bracing system at 1 st floor
RND #6	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour
GM7121#93%	Seismic	SSE-SD	93	0.64	- Relevant rotation of the cross beam under the vertical Tanks #1-#2 - Buckling of bracing system at 1 st floor - SG #7 plasticization at bottom wall of Tank #2
RND #7	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour
GM7121#100%	Seismic	SSE-NC	100	0.69	- Significant displacement of LVDT at Pipe POS #6 - Buckling of bracing system at 1 st floor '- Relevant rotation of the cross beam under the vertical Tank #2 and warping of the web of fin plate FP #1-#2
RND #8	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour
GM7121#115%	Seismic	SSE-NC	115	0.79	- Cracking on previously detail FP #1 and #2 significantly increased - Yielding of LVDT at Pipe POS #6 - Severe stresses on elbow Pipe POS #6 - Buckling of bracing system at 1 st floor
RND #9	Random	DBE-OP	-	0.14	- Overall elastic linear behaviour

3.5 Data Analysis and Insight

Both the system identification (SI) of the SPIF #2 structure and relevant observations drawn from the experimental campaign are presented and discussed hereinafter. In particular, the huge amount of collected data provides insights both on the primary structure and nonstructural components performance. Besides, a special focus is given to the dynamic interaction between the primary structure and the secondary components, but also between the structure, the components and various representations of the seismic input.

3.5.1 System ID of frequencies, mode shapes and damping ratios of the primary structure

To estimate the dynamic properties of the experimental mock-up, the RFP method, mentioned in Subsection 3.3.4, has been employed. The RFP technique overcomes many of the numerical analysis problems associated to parameter estimation of structures, such as damping ratio estimation, processing computational burden, etc.; moreover, it allows for a quite accurate identification of poles, zeros and resonances. For the sake of brevity, Table 3.5 summarizes the main results in terms of eigenvalues -frequencies -; while Figure 3.7a-b collects frequencies, damping ratios and relevant mode shapes.

Table 3.5: Identified frequencies by means of the RFP's signal analysis on the GMM7121 excitation for both the BF seismic and orthogonal direction. The horizontal line separates DBE from SSE seismic inputs. Also Δ [%] variations between current and initial frequencies are reported.

BF - Seismic Direction							BF - Orthogonal Direction						
Input	f_{n1} [Hz]	Δ [%]	f_{n2} [Hz]	Δ [%]	f_{n3} [Hz]	Δ [%]	Input	f_{n1} [Hz]	Δ [%]	f_{n2} [Hz]	Δ [%]	f_{n3} [Hz]	Δ [%]
GMM7121#31%	4.30	-	6.83	-	7.84	-	GMM7121#31%	4.58	-	-	-	7.29	-
GMM7121#38%	4.50	-4.68	6.59	3.48	7.74	1.28	GMM7121#38%	4.08	10.9	-	-	7.85	-7.68
GMM7121#62%	3.65	15.08	6.07	11.07	7.70	1.70	GMM7121#62%	4.97	-8.51	6.24	-	-	9.42
GMM7121#81%	3.85	10.42	6.24	8.65	7.83	0.09	GMM7121#81%	4.83	-5.46	6.40	-2.56	-	9.64
GMM7121#81%-2	3.90	9.36	6.19	9.28	7.71	1.58	GMM7121#81%-2	4.31	5.89	6.37	-2.08	-	9.53
GMM7121#93%	3.80	11.65	6.21	8.98	7.55	3.64	GMM7121#93%	3.26	28.8	6.84	-9.62	-	9.79
GMM7121#100%	3.58	16.87	6.36	6.88	7.62	2.74	GMM7121#100%	4.35	5.02	6.56	-5.12	-	8.79
GMM7121#115%	3.79	11.94	6.27	8.23	7.78	0.72	GMM7121#115%	4.13	9.48	6.19	0.80	-	9.48
													6.78

From Table 3.5, one can observe a slight decrease of frequency values in the DBE range, i.e. until GM7121#81%. Conversely, a frequency jump from the DBE to the SSE range, i.e. from 4.30 Hz to 3.79 Hz for f_{n1} , can be clearly noticed. In addition, identified frequencies of the BF orthogonal system w.r.t. the BF seismic system have proven to be fairly consistent and indicate a limited amount of damage. Moreover, some frequencies in the BF orthogonal directions have not been identified due to the lack of power in the signals. Nonetheless, the reliability of the identified values is confirmed by the values of the coherence function.

Figure 3.7a depicts the natural frequencies of the identified modes which clearly show a decrease after each seismic test; an exception is represented by the test run GM7121#62%, where the bracing system was subjected to elastic buckling with a subsequent stiffness recovery. The identified damping ratios reported in the same figure correspond to the first three modes of vibration, - 1-L, 2-L and 3-L-, and range between 5 and 16%. These high damping values are mainly due to the contribution of granular material stored in vertical Tanks #1 and #2, as stated also in [7]. On the other hand, the damping ratios of the modes associated with the deformation of the primary steel structure -modes 1-T+To- are mostly in the range 2.5 to 4.5%. Moreover, based on magnitude and phase of FRFs, the mode shapes of the tested structure have been derived and reported in Figure 3.7b. They correspond to the first three longitudinal modes (1-L, 2-L, 3-L) and the first coupled transverse-torsional mode (1-T+To). The modes 1-L and 1-T+To mainly contribute to the deformation of the primary structure; while the modes 2-L and 3-L represent the dynamic response due to coupling interactions between NSCs, i.e. the vertical Tanks #1 and #2, and the primary steel structure. Both for brevity and a better comprehension of Subsection 3.5.4, the main frequencies of Tank#1 and Tank#2 were identified. They correspond to ~ 6.2 Hz and ~ 7.1 Hz, respectively; these figures were largely influenced, indeed, by the kinematics of the granular content.

3.5.2 Performance of the Primary Structure

Reduce obvious stuff about design.worthwhile The primary structure depicted in Figure 3.1 was subjected to the dynamic inputs collected in Table 3.4. A careful reader can argue from Table 3.4 that the main structure exhibited an elastic behaviour up to a PGA level of 0.43g. Subsequently, secondary crossbeams experienced an inelastic behaviour followed by elastic buckling of the bracing system at the 1st floor. Later, significant web warping and cracking of the aforementioned FP#1-#2 connection happened, see Figure 3.3b.

For clarity, Figure 3.8a depicts the deformed shape of a brace at

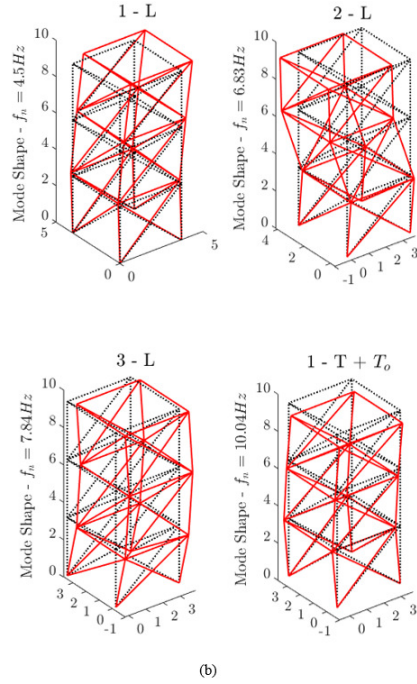
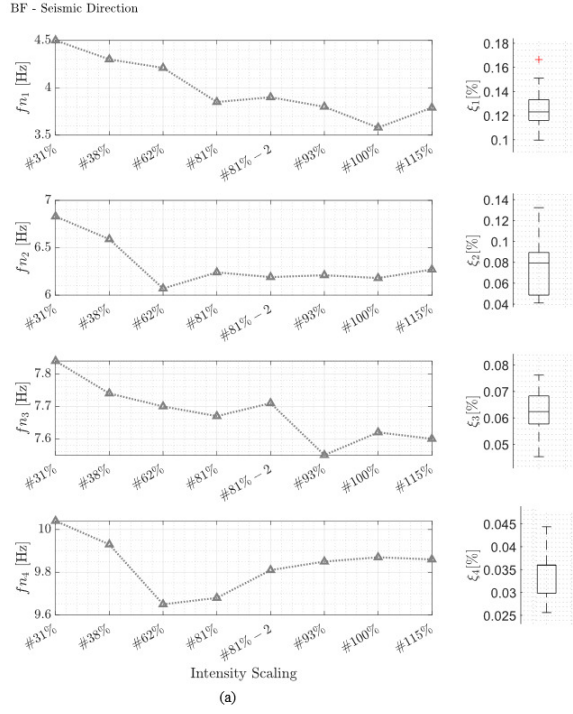


Figure 3.7: a) Relevant frequency variations for the BF mock-up. Damping ratio box-plots are reported with whiskers referring to the 10th and 90th-quantile; b) mode shapes relevant to frequencies $f_n = 4.50, 6.83, 7.84$ and 10.04 Hz of Table 3.5.

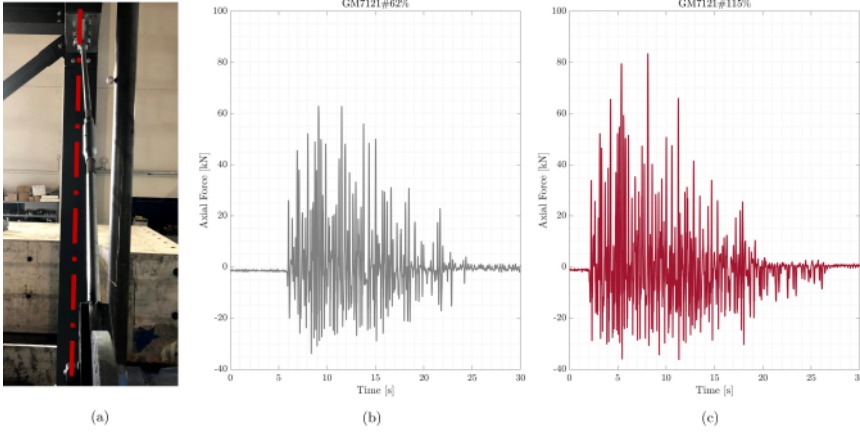


Figure 3.8: a) Photo of a buckled brace at GM7121#62%; b) axial forces time history at GM7121#62% and (c) at GM7121#115%.

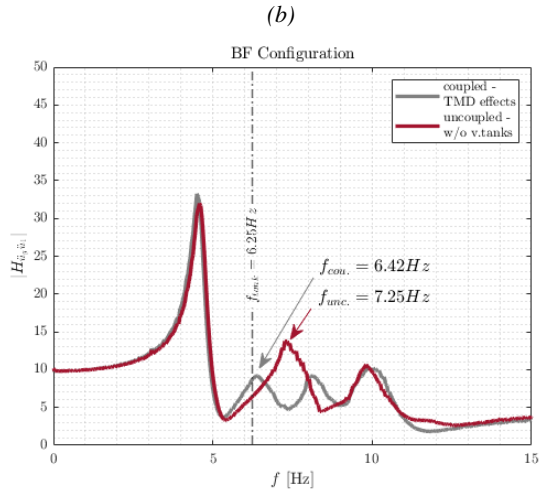
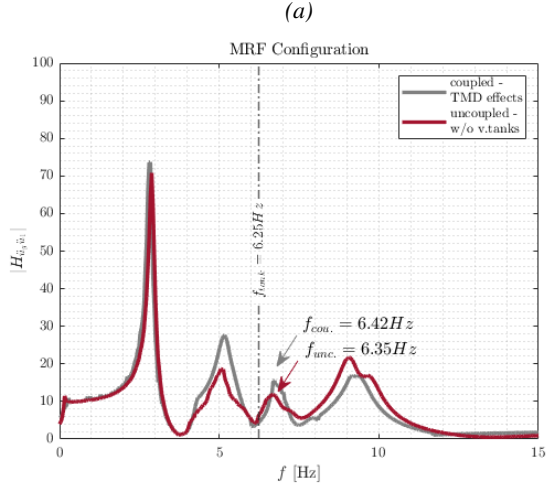
the 1st level buckled at about 20 kN for the excitation GM7121#62%. The average axial forces revealed by SG#14-15, depicted in Figure 3.4a, corresponding to the excitations GM7121#62% and GM7121#115% are shown in Figure 3.8b and Figure 3.8c, respectively. One can notice the asymmetric behaviour of braces due to elastic buckling at about 20 kN. The large values exhibited by braces in compression, e.g. about 38 kN are mainly affected by the preloading of about 15 kN initially applied to braces. Despite the high acceleration values that exceeded the NC limit state PGA values and, the consequent damage suffered by braces, they exhibited a favourable behaviour as foreseen in the design phases.

3.5.3 Behaviour of Secondary Components

Vertical Tanks As anticipated in Subsection 3.4.3, both vertical and horizontal tanks were monitored. In particular, being vertical, we focused our attention on vertical Tank #1 - with sensor A#7 at the base and A#8 at the top of the tank -, see Figure 3.4b. As a result, the top acceleration increase appears quasi-linear with the PGA level; whilst the amplification of the base acceleration was almost constant for each run except for: i) the run GM7121#62% where buckling of the bracing system occurred: ii) the runs GM7121#100% and GM7121#115% where buckling broadened and localized non-linearities at the fin plate connections intensified. Both the effects of brace buckling and stress peak values beyond the yielding threshold of 388 MPa and 245 MPa of tanks, respectively, indicate a localized plasticization of braces and tank base connections and justify the aforementioned behaviour. Moreover, a com-

parison of recorded acceleration data between sensors installed at floor level and at vertical tanks' bottom revealed some coupling effects, as described in depth in Subsection 3.5.4 by experimental processed data. As noticed during the BF shaking table tests, the structural response was significantly reduced w.r.t. the previous MRF campaign, despite the adopted higher acceleration spectra input.

To better understand the above-mentioned phenomenon, both Tables 3.6 and 3.7 together with Figure 3.9(c) have been prepared. Both table quantify the mass ratio $\mu = m_{Tanks}/m_{tot}$ and the modal mass ratio μ_{mod} of the heaviest tanks, i.e. the vertical tanks located at the first floor, including Tank #1. Therefore, for brevity, the entailing coupling analysis is restricted at the first floor of MRF and BF configuration, respectively. Moreover, let's consider Figure 3.9(c) that presents the FRFs for the MRF and BF configuration, respectively. In particular, grey lines stand for numerical simulations results related to the coupled system; red lines, instead, for numerical simulations without the 2 vertical tanks, installed at the first floor, whilst the vertical dash-dotted line indicates the natural frequency f_{tank} of the vertical Tank #1. One can clearly grasp two effects: (i) in Figure 3.9(c)(b) related to the BF, the tuned mass damper (TMD) effect, that reduces the FRF amplitude at 7.25 Hz with the appearance of two lower adjacent peaks; (ii) the location of f_{tank} which is in an antiresonance condition for the MRF configuration, see Figure 3.9(c)(a), and in a resonance condition for the BF configuration, see Figure 3.9(c)(b), with clear dynamic effects. One can look at De Angelis et al. [38] for additional considerations on TMDs with large masses. In such instances, the interplay between the primary structure, the vertical tanks and the seismic excitation can be better understood from Figures 3.12÷3.14



(c) FRFs for the (a) MRF and (b) BF configurations: grey lines stand for numerical simulations results for the coupled system; red lines for numerical simulations without the 2 vertical tanks, installed at the 1st level.

Table 3.6: Mass values and μ ratios.

Mock-up Masses			
		MRF	BF
Frame	[ton]	$\simeq 19$	$\simeq 19$
2 Vert. Tanks	[ton]	6	6
μ	-	0.32	0.32

Table 3.7: Modal masses and μ_{mod} ratios.

Modal Values			
		MRF	BF
Modal frequency	[Hz]	6.42	6.83
$M_{coupled}$	[ton]	$\simeq 0.95$	$\simeq 8.55$
$M_{w/out v.Tanks}$	[ton]	$\simeq 0.59$	$\simeq 2.34$
μ_{mod}	-	0.51	0.82

Pipe connections: BFJs With regard to pipe connections, the BFJ category consists of a DN100-PN16 joint with loose flange and collar on one side and solid flanges on the opposite; M16x8 bolts and 2 mm aramid fiber gasket with NBR binder. For clarity, a schematic cross section of the water pressure ($\simeq 20$ bar) Pos.#1 BFJ is depicted in Figure 3.10a, whilst a photo of the monitored BFJ is reported in Figure 3.10b. More precisely, to evaluate maximum values of flange opening and possible leakage, both SGs and LVDTs were installed. The leakage was also monitored with the use of optical fibers deployed in Pos.#4 BFJ, i.e. one of the four pressurized pipe branches. A design tightening torque of 40 Nm was applied following the procedure described in [35]. In particular and in seismic condition, that tightening torque ensures a minimum gasket pressure $Q_{s,min}^L$ equal to 6 MPa that corresponds to a tightening class $L_{0,01}$.

In order to track leakage phenomena, previsions were made based on the analytical predictive model proposed in [39]. The model is based on the [35] framework; however, since it takes into account also the interaction with shear forces, it results to be more accurate and reliable than the standards [35]. This is confirmed in the experimental work documented in [40].

In detail, the analytical predictive model results in an interaction shear F_s - axial forces F_{RI} leakage domain as shown in Figure 3.11. It has been evaluated with an internal pressure of 20 bar and a friction

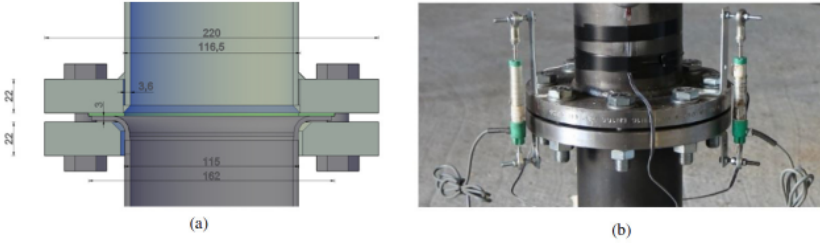


Figure 3.10: Pos.#1 BFJ: (a) sketch of the cross section; (b) photo of the monitored BFJ on site.

coefficient $\mu = 0.15$. More precisely, F_{RI} defines the equivalent resultant axial force at the load condition stage I , evaluated as the joint contribution of the applied axial force and bending moment to the BFJ, i.e.

$$F_{RI} = F_{AI} + \frac{4}{\phi} \cdot M_{AI} \quad (3.5)$$

The resulting shear F_s - axial F_{RI} leakage domains for both the DBE and SSE scenario are depicted in Figure 5.7a and Figure 5.7b, respectively. More precisely, for each scenario two interaction domains were set: (i) a domain that takes into account scatter effects due to the employed tightening technique method, according to Annex C and Annex F of [35], depicted by a continuous black line; (ii) a domain that does not take into account tightening scatter effects w.o. safety factors and depicted by a dashed black line.

In order to predict leakage of the entire set of BFJs, including the critical ones, i.e. Pos.#1, #4 and #5 that were instrumented, the FE model described in Subsection 3.3.4 was employed. The aforementioned figures collect leakage predictions for all installed BFJs where the instrumented BFJs are highlighted in red. A careful reader can note that at DBE conditions BFJs do not leak. Conversely, in the SSE scenario leakage of BFJs is not expected, since all predictions are inside the leakage domain w.o. safety factors, as shown in Figure 5.7b. These predictions were confirmed, indeed, both by visual inspection and data analysis provided by the test campaign.

In conclusion, Table 3.8 gathers EDPs values and observed damage, recorded during the experimental campaign, for the two limit states, thoroughly described in the previous Subsection 3.3.2. As a careful reader can notice, the recorded structural EDP values are far from the thresholds presented in Table 3.1. The reason for this is twofold: involved safety factors in the frame design; activation of large TMD effects, as mentioned in Subsection 3.5.3, that significantly reduced storey drifts. In-

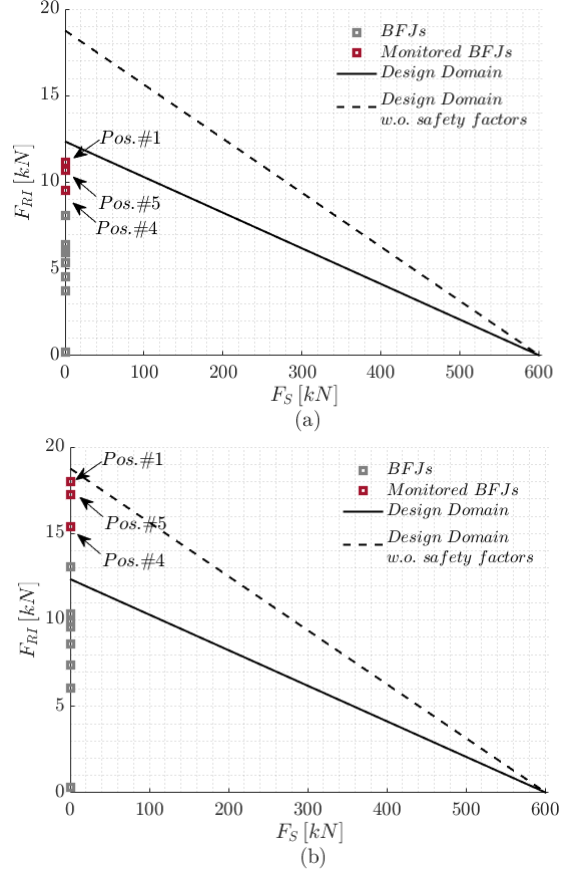


Figure 3.11: Leakage predictions and interaction domains with and w.o. safety factors for (a) DBE and (b) SSE condition.

stead, recorded EDPs and observed damage for NSCs are in good agreement with prescriptions of Table 3.1 of Subsection 3.3.2.

3.5.4 Correlations between input, primary structure and non-structural components

The results of the comprehensive shake table test campaign on both the initial MRF and the BF configuration of the SPIF mock-up highlight the importance of understanding the correlation between: (i) the seismic input and the demand parameters of the structure; but also between (ii) the seismic input and the demand parameters of the NSCs; besides, when coupling effects are significant, informative directions and strengths of the relationships between (iii) seismic demand parameters of NSCs and the demand parameters of the primary structure can be

found. To gain insights on potential correlations, several structural response quantities, NSCs demand parameters and seismic intensity measures were processed. Herein, main data findings are investigated for the two tested configurations, i.e. MRF and BF, respectively, in terms of both spectral and non-spectral demand parameters.

Seismic input vs primary structure

The investigated structural response quantities of interests (QoI) depicted in Figure 3.12a and b are floor acceleration spectra evaluated at the fundamental period T_1 of the system and, maximum drifts recorded by markers at the last floor, respectively. In the X axis, we selected the PGA as informative IM of the input, among other quantities [12]. According to observations made during the experimental tests and summarized in [18] for the MRF and in Table 3.4 for the BF - where at a PGA level of 0.43g brace buckling occurred -, inelasticity was mainly observed in the steel structure; conversely, it was limited in the NSCs. As a result, the correlations observed are clearly non linear. This is also evident looking at the forthcoming Figures 3.13 and 3.14.

In particular, see Figure 3.12a, although the BF was more severely excited by the synthetic accelerogram of Figure 3.2, the structural response $Sa_{floor}(T_1)$ was more severe for the MRF, due to the favourable TMD effect exerted on the BF as explained in Subsection 3.5.3. Similarly, as expected, Figure 3.12b reveals the same trend, in terms of non-spectral maximum top displacements $\max|D_{top}|$; as a result, $\max|D_{top}|$ values of BF exceed those of the MRF.

Seismic input vs NSCs

When NSCs are considered, see, in this respect, Figure 3.13, it is deemed necessary to consider as QoI for the relevant demand parameters the maximum floor spectral acceleration $Sa_{floor}(T_{NSC})$ evaluated at the identified period $T_{NSC} \simeq 0.16s$ of the vertical tanks, and the maximum relative drift ratio of vertical tanks at the first floor, $\max|D_{R,Tank\#1}|$. This latter QoI is evaluated according to $D_{R,Tank\#1} = (\max D_{top,Tank\#1} - \max D_{base,Tank\#1}) / H_{Tank\#1} \cdot 100$, where $\max D_{top/base,Tank\#1}$ stands for the maximum displacement recorded during each test, by the installed markers at the top and at the base level of Tank#1. With regard to the seismic input, along the indications of De Biasio et al. [12], we adopt the innovative equipment relative average spectral acceleration, namely $E - ASA_R(f_1)$. More precisely, $E - ASA_R(f_1)$ is defined as the average spectral pseudo-acceleration over the dominant-frequencies interval of the primary structure,

$$E - ASA_R(f_1) = \frac{1}{f_1 (X_f - 1)} \int_{f_1}^{X_f f_1} S_a(f, \xi) df \quad (3.6)$$

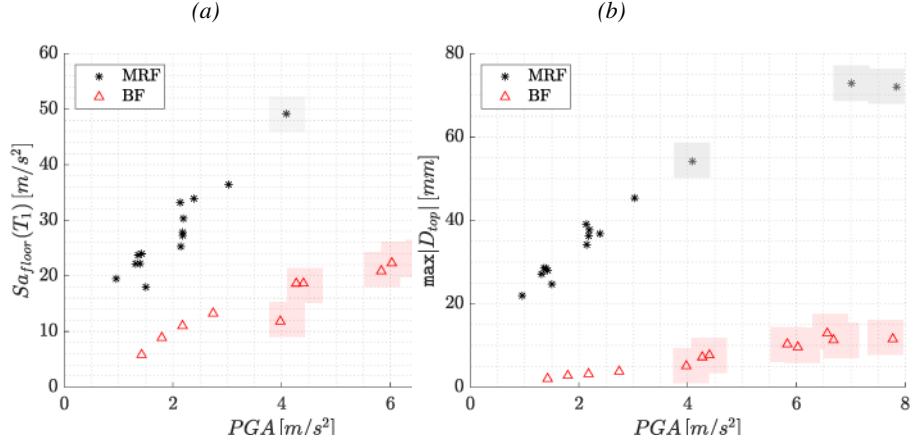


Figure 3.12: Seismic input vs primary structure correlation data: (a) PGA vs $Sa_{floor}(T_1)$ as spectral QoI; (b) PGA vs $\max|D_{top}|$ as non-spectral QoI. Collections of * and $\triangle$ data represent results for MRF and BF configuration, respectively. Shaded dots indicate processed data after inelastic behaviour occurred.

where X_f is a factor accounting for the drop of the fundamental frequency f_1 of the primary structure, S_a is the spectral pseudo-acceleration and ξ defines the structural damping ratio. The suffix R indicates the chosen percentage of drop of the fundamental frequency ($X_f = 1 - (R/100)$). Anew, as the location of f_{tank} is in an antiresonance condition for the MRF configuration, see Figure 3.9(c)a and in a resonance condition for the BF configuration, Figure 3.13a highlights the greater response in terms of $Sa_{floor}(T_{NSC})$ for the BF configuration. Along the same line, $\max|D_{R,Tank\#1}|$ values of BF exceed those of the MRF.

Primary structure vs NSCs

Anew, we deal with NSCs and primary structure characteristics. Therefore, a meaningful QoI of interest for the primary structure is $Sa_{floor}(T_1)$. We assume instead as QoI for the NSCs both the spectral shear base $S_{V,Tank\#1}$ and the non-spectral maximum relative drift ratio $\max|D_{R,Tank\#1}|$ of Tank #1. In particular, $S_{V,Tank\#1}$ is evaluated as $S_{V,Tank\#1} = m_{Tank\#1} \cdot Sa_{floor}(T_{Tank\#1})$, with a clear meaning of the terms. As $S_{V,Tank\#1}$ is proportional to $Sa_{floor}(T_{NSC})$, the same trend observed in Figure 3.13a is expected. As a consequence, also the maximum $|D_{R,Tank\#1}|$ is generally more intense for the BF configuration at equal spectral acceleration levels.

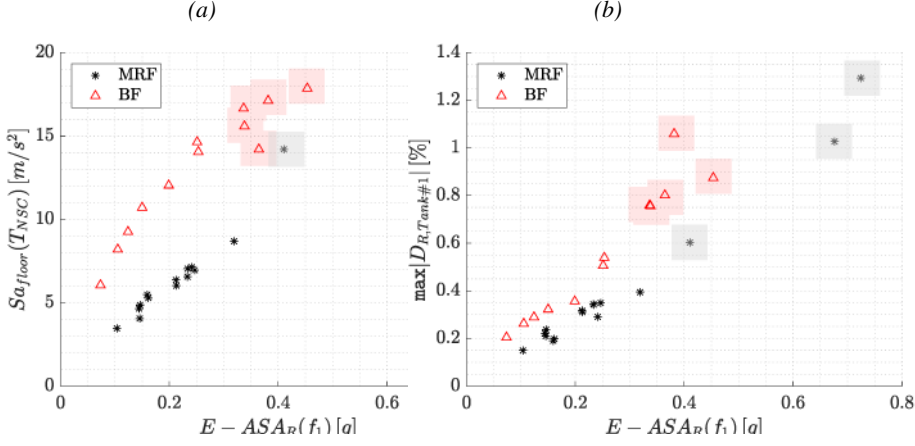


Figure 3.13: Seismic input vs NSCs correlation data: $E - ASA_R(f_1)$ vs (a) $Sa_{floor}(T_{NSC})$; (b) $E - ASA_R(f_1)$ vs $\max|D_{R,Tank\#1}|$. Collections of * and $\triangle$ data represent results for MRF and BF configuration, respectively. Shaded dots indicate processed data after inelastic behaviour occurred.

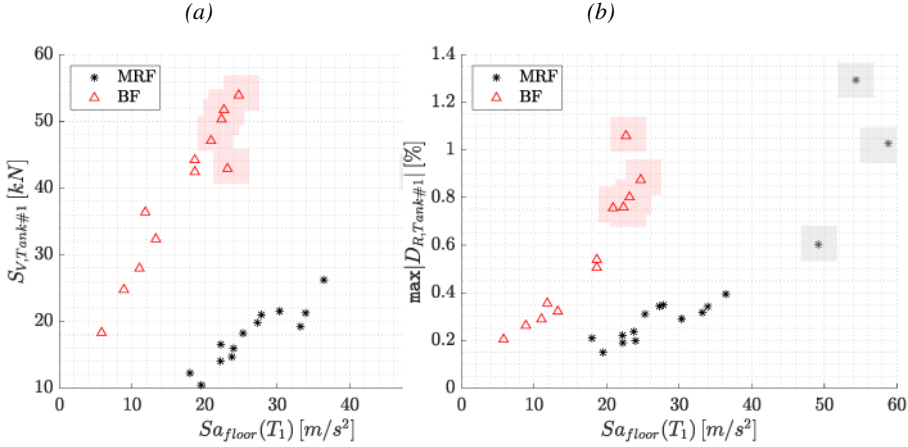


Figure 3.14: Primary structure vs NSCs correlation data: (a) $Sa_{floor}(T_1)$ vs $S_{V,Tank\#1}$; (b) $Sa_{floor}(T_1)$ vs $\max|D_{R,Tank\#1}|$. Collections of * and $\triangle$ data represent results for MRF and BF configuration, respectively. Shaded dots indicate processed data after inelastic behaviour occurred.

Table 3.8: Experimental EDP values and observed damage.

Primary structure				NSCs			
Limit State	PGA	Observed damage	EDP	Observed damage	Tank	Observed damage	Piping
DBE	0.56	Minor buckling of the bracing system at the 1 st floor	Storey drift 0.35%	Sever stresses on the bottom wall of Tank #1		Max tensile local strain 0.11%	Tensile strain Compressive strain 0.08% 0.07%
SSE	0.69	Buckling of the bracing system at the 1 st floor. Warping in fin plate connections FP #1-#2	Storey drift 0.4%	Yielding on the bottom wall of Tank #1		Max tensile local strain 0.18%	Tensile strain Compressive strain 0.14% 0.12%

3.6 Conclusions and outlooks

The SPIF project, developed inside the European H2020 SPIF research framework, focused on the investigation of the seismic behaviour of an archetype multi-storey moment resisting frame (MRF) structure equipped with complex secondary industrial components by means of shaking table tests. The experimental campaign was comprehensive and successfully carried out at the EUCENTRE Laboratory of Pavia and the tests clearly pointed out the inadequacy of floor crossbeams and routine design of fin plate connections. The existing SPIF MRF mock-up was converted into a braced frame (BF) configuration, by installing a dedicated bracing system. The mock-up was tested by means of a shaking table with several earthquake levels within a PBEE perspective. Thus, the present paper proposed limit states and performance objectives for a primary BF structure and NSCs; and to excite the vibration periods of the NSCs, i.e. the heaviest vertical tanks, a synthetic record derived by a site-based ground motion model was selected; as a result, the complex interaction between the primary BF structure and the NSCs was enhanced; and the experimental EDPs values were compared with reference thresholds.

As a result, the SPIF#2 testing program was able (i) to highlight the efficiency of the BF configuration; (ii) to experimentally verify the effectiveness of the proposed PBEE limit states and thresholds; (iii) to provide data about the dynamic interaction between the primary steel BF and the NSCs; (iv) at the very least, to interpret the causes of these interactions. More precisely, the primary steel structure designed at the near collapse (NC) limit state exhibited almost a linear behaviour until the foreseen buckling of the bracing system at the first level. With regard to the NSCs performance, leakage phenomena at the monitored bolted flange joints did not occur, as predicted by analytical formulations based on design codes. Conversely, due to the behaviour of the heavy vertical tanks on secondary beams, the strong dynamic interaction between primary structure and NSCs was evident. In this respect, experimental relationships between several structural QoI, NSCs response parameters and PGA were determined. They showed that: (i) vertical tanks at the first floor acted as tuned mass dampers (TMDs) for the BF, and thus, the relevant base shear at the first floor was limited; this trend was also confirmed by frequency response functions; (ii) owing to dynamic properties, the favourable TMD effect was not relevant for the MRF configuration; (iii) owing to the flexible behavior of the secondary beams, vertical tanks induced warping of the web and crack propagations in the same beams and fin plate connections. In summary, a proper

experimental test campaign on an archetype braced steel frame showed: i) the effectiveness of a proper installed bracing system; ii) the high influence of NSCs on the primary structure due to dynamic interaction. To enhance understanding of these complex systems and to improve current standards, low and high fidelity models calibrated on experimental data are needed for reliability evaluations.

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4. Fiber Bragg Grating sensors performance in Bolted Flange Joints of piping systems under seismic conditions

This chapter contains the post-print of

Paolacci, F., Quinci, G., **Nardin, C.**, Vezzari, V., Marino, A., Ciucci, M. (2021). Bolted flange joints equipped with FBG sensors in industrial piping systems subjected to seismic loads, *Journal of Loss Prevention in the Process Industries*, Volume 72, <https://doi.org/10.1016/j.jlp.2021.104576>.

as published in the Journal of *Journal of Loss Prevention in the Process Industries*, differing from the published paper only in terms of layout and formatting.

Author contributions. **F. Paolacci:** Supervision, Writing – Review. **G. Quinci:** Formal analysis, Software, Data Curation, Writing – Original Draft. **C. Nardin:** Formal analysis, Data Curation, Writing - Original Draft. **V. Vezzari:** Methodology, Investigation, Writing - Original Draft. **A. Marino:** Funding Acquisition. **M. Ciucci:** Funding Acquisition.

Summary

The vulnerability of major-hazard industrial plants to natural hazards has been recognized as an emergent issue whose importance is underlined by the Sendai Framework, established immediately after the Tohoku earthquake of 2011, in Japan. Hence, seismic risk analysis is of paramount importance as testified by the intense research activity that characterized the last years. In this respect, structural health monitoring can represent a valuable tool able to strongly help the decision-making phase. Along this main vein, optical fibers (OFs) represent a class of sensors able to both monitor critical conditions, as leakage of hazardous material, and activate safety barriers, if any. More precisely, optical fibers represent an economic solution, whose characteristics appear particularly suitable for dangerous environments like major-hazard plants. However, investigations relevant to their use for seismic monitoring of chemical/petrochemical plants are rather limited, especially when subject to strong dynamic excitations.

As a result, this paper deals with the analysis of optical fiber Bragg gratings (FBGs) applied to bolted flange joints (BFJ) under cyclic loadings. More precisely, two experimental programs, i.e., a cyclic test on a single BFJ and a series of shaking table tests on BFJs of a multicomponent system, demonstrated the effectiveness of the proposed monitoring systems in detecting hazardous conditions and, thus, their potential use in conjunction with safety barriers.

4.1 Introduction

4.1.1 Background and Motivation

The impact of a natural disaster on a facility storing or processing dangerous substances can result in the release of hazardous substances with possible severe off-site consequences through toxic-release, fire or explosion scenarios. Accidents triggered by a natural hazard, which involve release of dangerous substances are commonly referred to as NaTech events [1]. These include releases from fixed chemical installations and spills from oil and gas pipelines. One of the main problems of NaTech accidents is the simultaneous occurrence of a natural and a technological disaster; both require simultaneous efforts to address a situation where rescue lines designed for disaster mitigation are likely not available, as they may have been damaged by the natural disaster. In addition, hazardous substances releases may occur from single or multiple sources, resulting in multiple chains of accidents in the same installation, or in different plants; this requires emergency-management resources that can be engaged in responding to the natural disaster in other situations [2]. Despite of a recognized growing of research activities, that results in more strict regulations for the design of industrial activities, NaTech accidents remain an emerging threat [3]. The Directive 2012/18/EU of the European Parliament, implemented by European member states, recognized the relevance of NaTech events. Therefore, Lanzano et al. [4] presented a large database of earthquake-induced damages for steel and non-steel pipelines. The database can be adopted for the definition of fragility curves commonly implemented for the assessment of Na-Tech risks. Along this vein, the use of smart sensors-based technology integrated with existing safety sensors, generally present in major-hazard industrial plants can represents an interesting mitigation strategy option. In particular, to reduce the risk in case of major accidents and control the consequences of seismic NaTech events, both the use of *ad hoc* monitoring and control systems [5] and the implemen-

tation of smart technologies -sensors, actuators and innovative systems for seismic protection- can be pursued. For this purpose, various types of sensors can be used, which are often integrated into specific systems for structural monitoring and identification of the release of dangerous substances [6]. Sensors currently available on the market and employable for this purpose, can be broadly divided into: a) sensors for deformation measurements; b) sensors for acceleration measurements; c) sensors for temperature measurements [5]. The first group includes the more traditional sensors, capable of measuring deformation in a point of the structure, also in real time. They are resistive in nature and generally have a high measurement sensitivity. Nonetheless, they exhibit the disadvantage of being installed in specific points of the structure where major deformations are expected; therefore, their installation requires a preliminary structural study. Strain gauges and linear potentiometers belong to this family. They generally require a permanent in-situ acquisition system, capable of continuously reading deformations. In the recent past, different experimental tests campaigns demonstrated the reliability of traditional sensors in detecting loss of containment (LoC) events during progression of physical damage [7], [8]. However, the use of this class of sensors requires a large amount of data to training a trustworthy relation between seismic damage and hazardous material release [9]. In order to detect the release of hazardous substances through automatic check valves, sensors for acceleration measurements can be usefully employed. They are able to detect both seismic P and S waves and, therefore, they can exploit, the time between the arrival of S and P waves. As a result, they can promptly trigger the actuation systems of safety valves, etc. [10]. The micro-electro-mechanical-system (MEMS) technology can reduce the costs of these sensors, and nowadays allows to produce accelerometers contained in a single chip. Moreover, these sensors can operate in a wireless environment, e.g. within networks known as radio frequency identification tags (RFIDs), [11]. These systems exhibit a remarkable low battery consumption with a guaranteed life of at least 10 years. These sensors generally provide a favorable integration into early-warning systems; it is well known that early-warning systems exhibit a series of problems related to both signal arrival time and the so-called "false alarms". These issues are still under investigation due to economic losses related to false alarms [12]. An alternative to the aforementioned sensors, which are widely used in chemical industries for leakage detection, is based on temperature measurements associated with substance release [13]. Optical fibers belong to this sensor family; they have been often used for releases of dangerous substances in pipes of refineries or in pipelines devoted to the transportation of gas and

oil [14], [15]. Since these systems are generally continuous, it is not required to determine a specific LoC position in advance, but it is sufficient to install FBGs along the entire line to be monitored [16]. The cost of optical fibers is rather limited, while the acquisition system is still the most expensive component of the measurement system chain. For the sake of completeness, we mention other interesting solutions, like the acoustic emission technique (AET), [17]. The AET represents an important tool employed in non-destructive analysis techniques (NDT), it is commonly used to locate potential cracks in elements subjected to external loads. The sensor is generally based on piezoelectricity and is integrated with a preamplifier. This sensing system can also be implemented with remote sensing techniques by means of wireless technologies. For instance, to monitor cracks in welded piping systems endowed with elbows and tee joints, Sayginer et al. [18] successfully employed AET.

4.1.2 Scope

With the objective to examine new industrial facilities endowed with structural sensors to improve safety performance and to upgrade safety conditions of existing industrial plants also retrofitted with sensor solutions, we consider the implementation of smart sensors in view of simultaneous monitoring and control of natural and man-made adverse events. Accordingly, this paper analyzes the feasibility of FBG grating for monitoring and leakage detection of BFJs. For this purpose, two experimental tests have been carried out: i) a cyclic bending test of a full-scale BFJ; ii) a more demanding one, i.e. a shaking table test on a full-scale complex industrial multi-storey frame structure with process equipment and pipes. Both tests demonstrate the feasibility of FBG sensors in leakage detection in structural BFJs, and thus, the potential risk reduction and plant control exerted by these sensors under seismic loading. In this respect, the paper is subdivided in different sections. In view of a better understanding of piping system and desired goals for a monitoring active system capable to prevent extreme consequences due to hazardous material release under seismic loading, Section 4.2 focuses on seismic performance of those kind of systems. Section 4.3 instead, provides an overview of the use of optical fiber sensors and FBGs for seismic monitoring of piping components and BFJs. For the sake of clarity, the cyclic bending tests of the aforementioned BFJ and the relevant experimental investigation is described in Section 4.4. Conversely, Section 4.5 introduces a more complex case study based on shaking table tests and the BFJ performance monitored with FBGs. Finally, conclusions are summarized in Section 4.6 with further perspectives.

4.2 Related work

With regard to seismic safety of industrial piping systems and LoC detection, we begin underlying that in petrochemical plants, kilometers of pipes are installed to transfer raw and refined materials, e.g., fluid and gas, and connect all components involved in the transformation process, i.e. tanks, distillation columns, furnaces, etc. Consequently, piping systems represent a key element for industrial plants and thus deserve a particular attention, especially in seismic prone-areas where consequences of possible release of hazard materials should be accounted for in view of possible severe hazardous events, like fire, pollution, etc. [19]. The impact of 1999 Turkey earthquake at Itzmit [20] or the 2008 Wenchuan earthquake in China [21] clearly demonstrated these effects. The current design approach to piping systems in chemical plants against earthquakes is still strongly based on the allowable design method, instead of the most modern performance-based approach [22]. One of the main reasons is the scarcity of information about the definition of limit states for pipes and the structural modeling, which have not yet been treated in a satisfactorily manner. Moreover, most of the seismic Codes and Standards, (EN1998:4 2006 [23], EN13480 2002 [24], ASME B31.3 2004 [25]) do not contain enough rules and information for the proper design of industrial piping systems in seismic-prone areas and related consequences due to release of hazardous material, gas or liquid. As a matter of fact, Eurocode 8 part 4 [23], the reference European seismic code for industrial components, is devoted to pipelines, but only of above ground and buried type, which differs from metallic piping systems for many aspects, and thus useless. Actually, the main European contribution to industrial piping systems design is primarily represented by the EN13480 [26], dedicated to metallic piping systems; however, the seismic problem is only partially treated and should be more deepened. Americans know-how on piping systems is instead very rich, especially in terms of design standardization and seismic design calculations, and the long list of Standards and Codes available represents an evident demonstration. More precisely, the main American standard is represented by the ASME B31.3 [27], but many other contributions and guidelines are also available [28],[29]. However, it is necessary to stress that ASME B31.3 does not contain explicit indication on seismic analysis of piping system, but rather refers to the American Standard for the seismic analysis of structures, i.e. ASCE7, which also includes clauses for non-structural elements like pipes [30]. In addition to the fuzziness in design rules of piping systems, most of the standards devoted to preventing hazardous material release are inadequate in case of Na-Tech events. In fact, ac-

cording to literature survey, many of the current common standards for leakage detection are exclusively related to operation conditions and integrity management of pipelines and not to refinery piping systems. Most of them are issued by the American Petroleum Institute (API) and the International Organization for Standardization (ISO) and are mainly focused on technology and in-situ inspections (DNV 2016 [31], API 2017 [32], ISO 2017a/b [33]-[34], ASTM 2017 [35]). The above framework clearly highlights the need of specific techniques to detect leakage in case of extreme events like earthquake. In sum, two main issues are still unresolved: i) a clear methodology for relating seismic damage of piping systems to gas or liquid release; ii) reliable technologies and techniques for leakage detection able to account for severe damage conditions caused by seismic action.

4.3 Use of optical fibers for seismic monitoring of Bolted Flange Joints

Fiber Bragg Grating (FBG) optical sensors have been installed on several BFJ loaded under different conditions, described in the following sections, to evaluate the seismic behavior of the joints and to monitor any loss of content. The results recorded by the FBG sensors were discussed and compared with those from traditional sensors. In order to better understand working conditions and leakage detection features of FBG optic fiber sensors, a brief overview and possible applications of this technologies in oil and gas industry is provided.

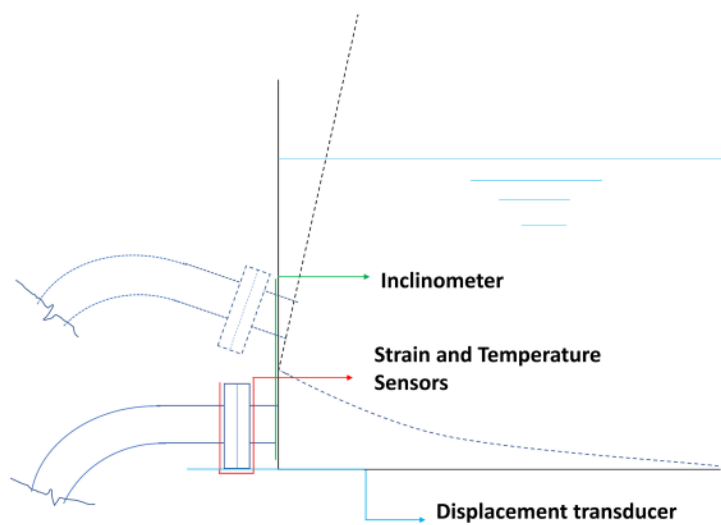
Characteristics and working conditions of FBG fiber sensors

FBG fiber optic sensors, which are based on interference phenomena caused by the reflection of waves by different parallel crystalline planes, are multifunction transducers capable of detecting, with high accuracy and at long distance, deformations, temperatures, length variations, dynamic pressures, geometry variations and accelerations along a single mode optical fiber bonded or embedded in the structure or on the component to be monitored. An FBG optic sensor is represented by a diffractive grating photo-etched with a UV laser along a single mode optical fiber of 125-micron diameter, whose property is to reflect an associated well-defined light wavelength, generated by a laser or SLED source. The low weight and dimensions of the FBG sensors means a minimal application impact, allowing their installation in the most critical part of a structure. When the light emitted by a laser source or a super luminescent LED (SLED) passes through the optical fiber, each grating inside reflects the light with a specific wavelength associated with it, expressed

in nm (nanometer), which varies according to the length of the grating subjected to traction or compression, induced by thermal or mechanical stress. Each wavelength variation associated with each individual FBG sensor is detected and converted in real time into temperature, deformation, vibration, pressure or geometry variation, by a dedicated optoelectronic control unit, known as an FBG interrogator. Therefore, by inscribing along a single mode optical fiber, a certain number of FBG gratings each having its own nominal wavelength, therefore uniquely identifiable, we obtain an FBG fiber optic backbone capable of measuring, in those points where the FBG diffractive gratings are present, temperature, deformation, vibration, pressure or geometry variation. Furthermore, since their total insensitive to oxidative and erosive phenomena, FBG sensors are able to work in the worst environmental conditions, in a wide temperature range (-180° to 300°), even if immersed in a liquid. This sensor is suitable for networking since it has a narrowband with a wavelength operating range and hence can be highly multiplexed. This nonconductive sensor can operate in electromagnetically noisy environments without any interference. An FBG sensor is made up of glass which is environmentally more stable and with a really high long-life time. Because of its low transmission loss, the sensor signal can be monitored from longer distances and with a high sensitivity ($\pm 1 \mu\epsilon$ for deformations; $\pm 0.1^{\circ}\text{C}$ for temperatures), making it suitable for remote sensing [36].

Usage of FBG fiber sensors for leakage detection of hazardous equipment in seismic areas

In order to identify possible applications of FBG fiber sensors, the most frequent seismic damage involving BFJ are here identified. Besides, the most suitable working conditions of these smart sensors are presented. In the case of unanchored storage tanks, a possible release mechanism can be identified in the junction area between the tank and the inlet and outlet pipes. The lifting mechanism and/or the sliding mechanism of the base, typical of this type of tanks, could in fact cause excessive movement between the tank and the pipes Figure 4.1.



(a)



(b)

Figure 4.1: (a) FBJ Sensor placement for the sliding/uplifting monitoring of unanchored storage tanks; (b) example of inlet/outlet pipes of a storage tank.

In this case, the movement between tank and pipeline can induce relative rotation between the two flanges of BFJ that can be monitored through an FBG sensor (red line in Figure 4.1-(a)). In this way it is possible to identify excessive rotations of the joint and to perform an intervention such as a possible closure of a safety valve or alerting the system for an immediate check of the hazardous condition. Also, FBG temperature sensors could be profitably used for leakage detection as already exploited in the literature [37]. Another useful employment of FBG sensors is to incorporate inclinometers in the measurement system to derive the uplift phenomenon and the consequent bending of the pipe, green line in Figure 4.1-(a), [38]. Finally, FBG displacement sensors could be used to detect the sliding of the tank base, which could correspond to an excessive axial force in the BFJ, blue line in Figure 4.1-(a). Another type of hazardous equipment widely used in hazardous plants is represented by vertical columns. For this structural archetype, the most sensitive fitting is the BFJ of pipes connected to the vessel at various heights, as well as the pipe typically placed at the top of the column (Figure 4.2-(a)). In both cases, the main damage mechanism to be monitored is the relative rotation of the two flanges of BFJ of the column-pipe system. In this respect, FBG strain sensors installed on the pipe flange joint can be profitably used. Indirect measure of the BFJ damage condition could be derived by using FBG inclinometers (Figure 4.2-(a)). FBG temperature sensors could be also installed in the most critical BFJs to detect possible leakage. A similar sensors pattern can be employed for elevated vertical vessels that suffer similar damage conditions. In addition, to avoid serious consequences following the structural collapse, it is always possible to use FBG strain sensors to monitor the yielding and post-yielding condition of the support columns at the base section, where the maximum stress is typically developed (Figure 4.2-(b)).

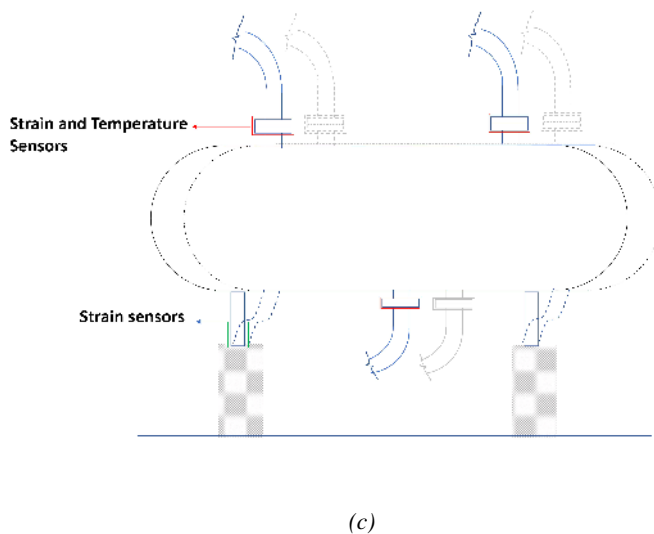
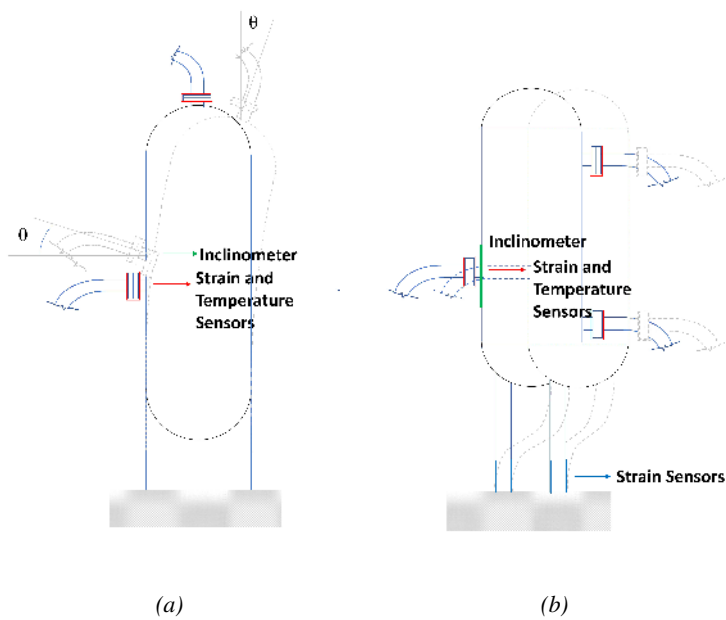


Figure 4.2: (a) FBJ sensor layout for columns; (b) FBJ sensor layout for vertical elevated vessels; (c) FBJ layout for horizontal pressure vessels.

For horizontal pressurized tanks, it may be useful to monitor the relative rotation in the flanges of the connected pipe BFJs (Figure 4.2-(c)). FBG strain sensors can also be used to check the stress level in the saddles, due to the oscillation of the vessel under the earthquake. The above framework clearly shows how the use of FBG sensors can be considered a valuable resource for acquiring useful data and reduce the consequences of a hazardous material release. In fact, the use of sensors for local monitoring of the most sensitive points allows the activation of safety barriers (e.g. shut-off valves) capable of limiting both the leakage of dangerous substances and containing possible domino effects. In order to show the applicability in practice of FBG sensors for monitoring bolted flange joints subjected to damages involving the release of hazardous material during an earthquake, the results of two different experimental tests will be presented and commented.

4.4 Experimental investigation on an isolated Bolted Flange Joint

4.4.1 Description of Case study #1

For the investigation of optical fibers as structural monitoring system and leakage detection of bolted flange joints, two relevant case studies are herein considered. The first one is composed of a coupled tank-piping system, which employs a smart sensor based on AET technology [18]. In particular, the piping network is made by API 5 L X52 steel, equivalent to the European P355 steel. Besides, it encompasses 8" (outer diameter: 219.08 mm; thickness: 8.18 mm) and 6" (outer diameter: 168.28 mm; thickness: 7.11 mm) schedule 40 pipes. Furthermore, the piping network includes several critical components, mainly two elbows, a bolted flange joint, and a Tee joint. The layout of the piping system and the main dimensions are given in Figure 4.3, where the bolted flange joint, considered for the experimental tests, is circled in red. This joint is considered critical because of the unanchored conditions of the tank that can have some sliding and thus stress in bending and axial conditions of the flanged connection. Details on the bolted joint components are provided in Section 4.5.

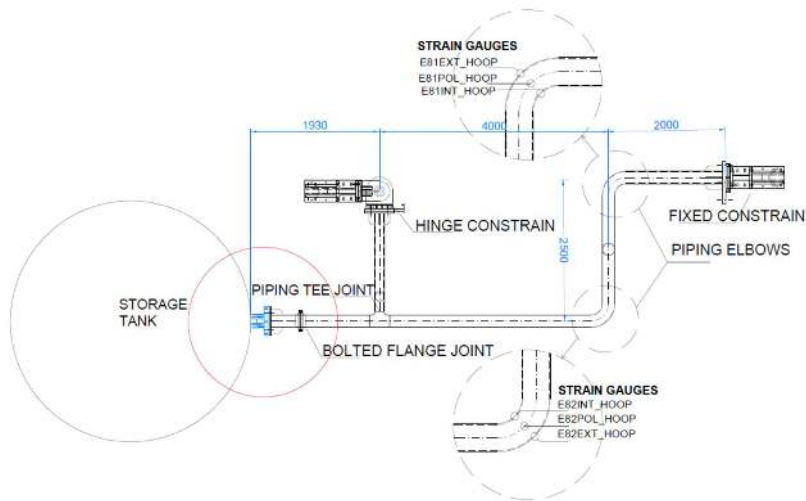


Figure 4.3: Case study #1: tank-piping system and secondary components, (after Sayginer 2020 [18])

4.4.2 Description of the mock-up and the sensors layout

The specimen consists of a portion of piping which also includes a standard flanged joint, designed according to EN 1092-1 (2007) [39], whose gasket has been selected according to EN 1514-2 (2005) [40]. The dimensions of the specimen and flange are shown in Figure 4. The pipe has a diameter of 219.1 mm, a thickness of 8.18 mm and a total height of 1.608 m. The flange plates have an external diameter of 375 mm and are connected to each other by $\phi 30$ mm bolts and welded to the pipe with welds of adequate strength. Figures 4.5 shows the sensors layout used for the experimental test. It consists of a series of LVDTs for measuring the relative displacements between the two flanges of the joint. There are also strain gauges (SG) to measure both longitudinal and circumferential local deformations. To measure the horizontal displacements at the top and at the base of the column (sliding), two wire potentiometers have been installed as illustrated in Figure 4.5-(a). Similarly, other two wire potentiometers placed in vertical position were used to monitor the vertical displacements of the ground anchor system (to the right and to the left of the specimen) Figure 4.5-(a). The rotation of the base was calculated as the ratio between the sum of the vertical displacements and the distance between the two potentiometers.

In addition to traditional sensors, a fiber optic sensors system has been installed with twofold purpose: a) measuring both the mechanical quantities measured by traditional sensors to validate their reliability (US

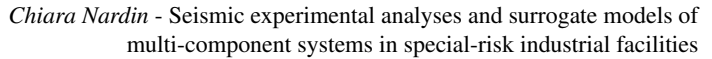


Figure 4.4: Main dimensions of specimen and flange joint.

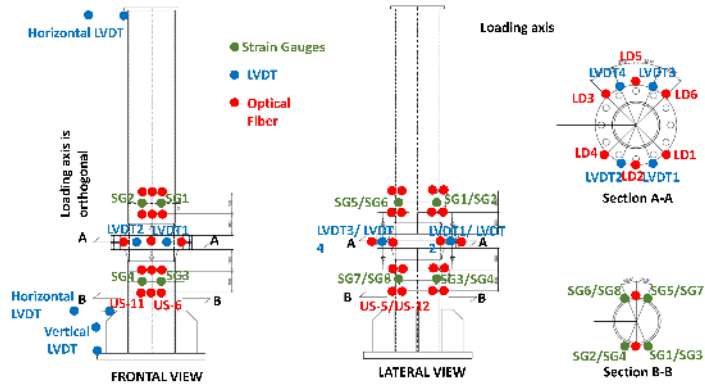
type of Figure 4.5-(a)), b) measuring temperature variations (LD type of Figure 4.5-(a)). The latter are generated by pressure variations caused to the Joules Thomson phenomenon [13], which therefore indicate conditions of material release (leakage). For the application of Fluids Leakage Detection monitoring on the experimental flanged test pipe, one leakage detector has been installed around the two flanges, which is composed by six FBG-based mechanical transducers (US) and four FBG-based thermal transducers (LD), able to detect any leakage caused by a relative rotation of the flanges or by a local failure of the pipe.

The leakage detector fiber optic terminal was then connected to a channel of the interrogator unit, able to read with high accuracy and sensitivity, deformations and temperature variations up to 2.5 kHz sampling rate for each of the FBG sensors installed along the optical fiber. The interrogator unit, thanks to the very low signal loss rate, can be installed at a long distance from the FBG sensors, maintaining accuracy, sensitivity and safety of the measurement. To validate the system dedicated to leakage detection (LD sensors), a test setup was developed, as shown in Figure 4.6, consisting of a support in composite material instrumented with four FBG sensors encapsulated inside a teflon capillary to ensure the mechanical decoupling of the FBG sensor and allow only thermal sampling. To test the capacity of the LD sensors to detect leakage, a radial-direction hole was artificially created on the pipe from which the pressurized liquid was instantly released. In Figure 4.6-(a) it can be observed the moment when the temperature variation corresponds to the instantaneous loss of the liquid off the hole highlighting the generation

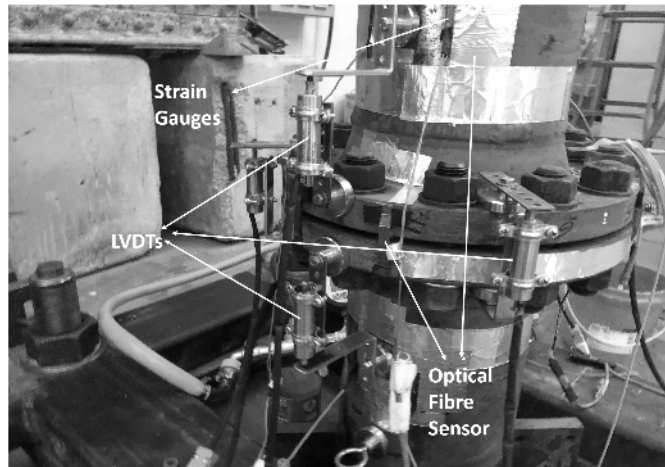
of the Joule -Thomson effect (Figure 4.6-(b)).

4.4.3 Description of the test setup

The experimental setup illustrated in Figure 4.7-(a) was conceived with the idea of creating a cantilever scheme, fixed at the base and subjected to an imposed displacements history at the top. This has allowed to easily determine step-by-step the static behavior of the structure, also in the nonlinear field, and thus to detect the structural damage. The displacements history was applied by using a 250 kN MTS hydraulic actuator with a stroke of 25 cm. The characterization of the monotonic behavior of the structure was derived by using a quasi-static displacement history with a velocity equal to 0.02 mm per second. This allowed to follow up the evolution of the local and global response of the structure.

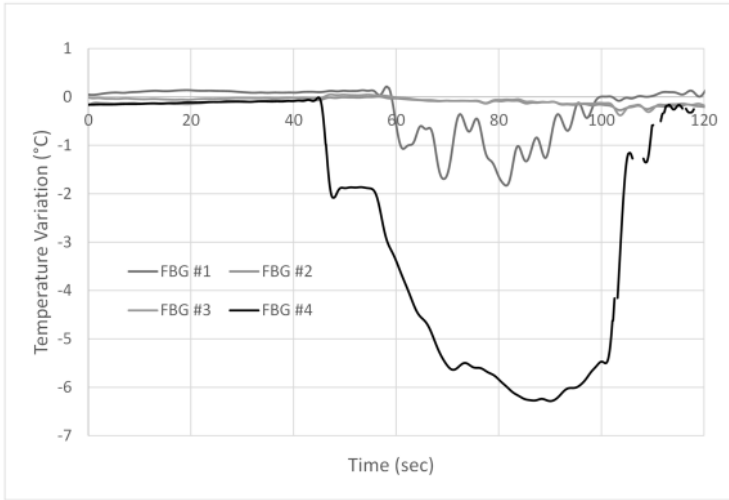


(a)

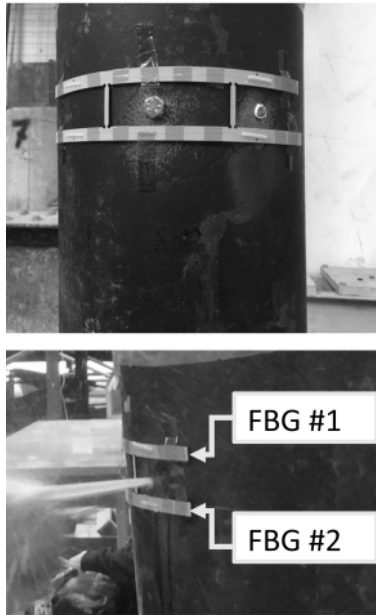


(b)

Figure 4.5: (a) Sensor layout for the flange monitoring (b) Sensors installed on the specimen.

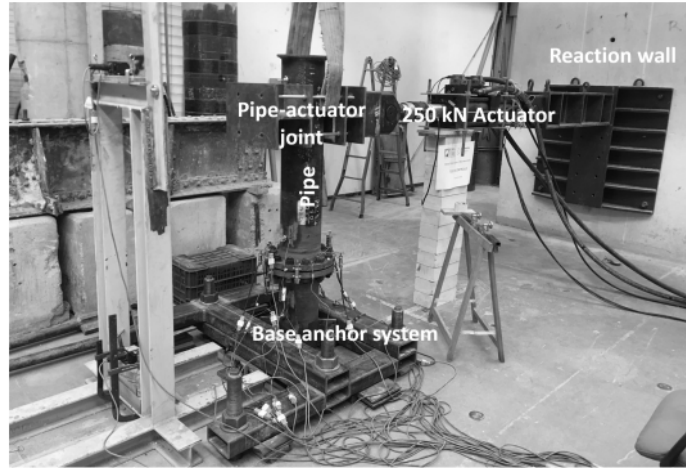


(a)



(b)

Figure 4.6: (a) Joule-Thomson effect during the validation test, (b) test setup of the validation test.



(a)



(b)

Figure 4.7: (a) Experimental setup (b) Liquid pressure system.

An external access point for the liquid was located at the pipe bottom, Figure 4.7-(b), and was used for the pressurization of the system pumping water up to the desired pressure level. In the examined case, the maximum applied pressure was equal to 30 bar. To facilitate the pressurization and de-pressurization of the pipe, a water outlet point has been created at the top. This allowed to check the full liquid conditions in the pipe and facilitate the emptying phase after each test. In order to assure fixed conditions at the pipe bottom, a series of steel plates and profiles were used as illustrated in Figure 4.7-(a). However, this system did not guarantee the expected rigid behaviour. Consequently, the ground anchorage system was also monitored to derive the rigid horizontal displacement generated, during the test, by the bottom joint rotation. This component was used to derive the net elastic/plastic displacement at the top.

4.4.4 Results

Figure 4.8 shows the displacements history with and without the contribution of the rigid displacement of the pipe, where it is possible to observe the finite deformability of ground anchorage system. Figure 4.9 illustrates the force-displacement relationship of the pipe. Firstly, it is possible to observe the yielding condition of the pipe, which occurred for a displacement of 42 mm, beyond which the force remained constant, demonstrating the full plasticization of the bottom pipe section. This behaviour is confirmed by the local deformations of the pipe measured by the strain-gauges along the longitudinal direction, Figure 10a. The onset of the yielding conditions occurred for a deformation of 0.2

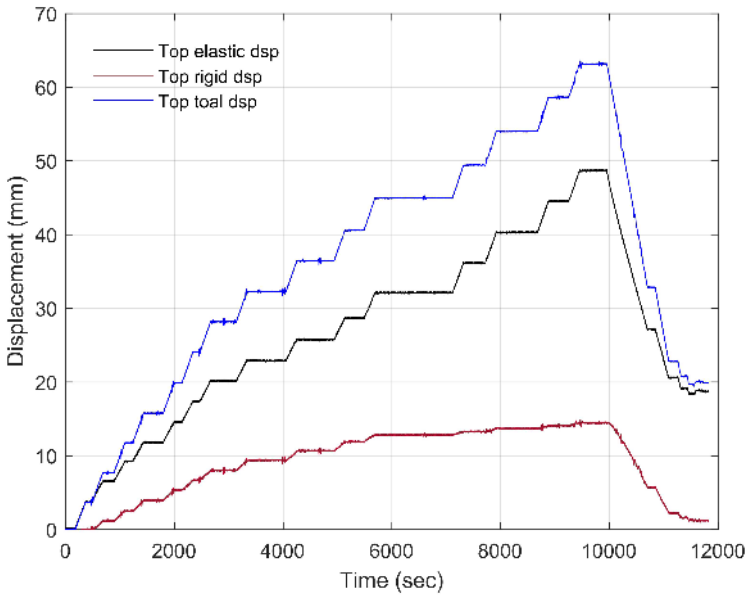


Figure 4.8: Top displacements history.

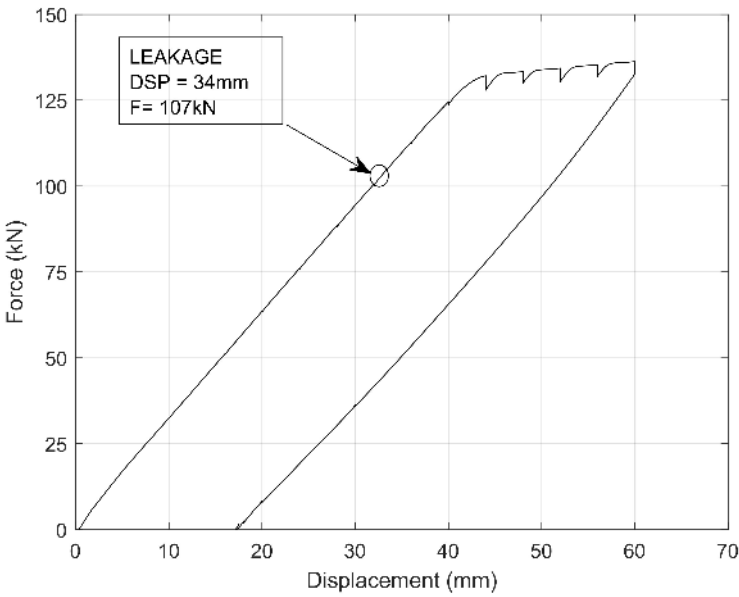
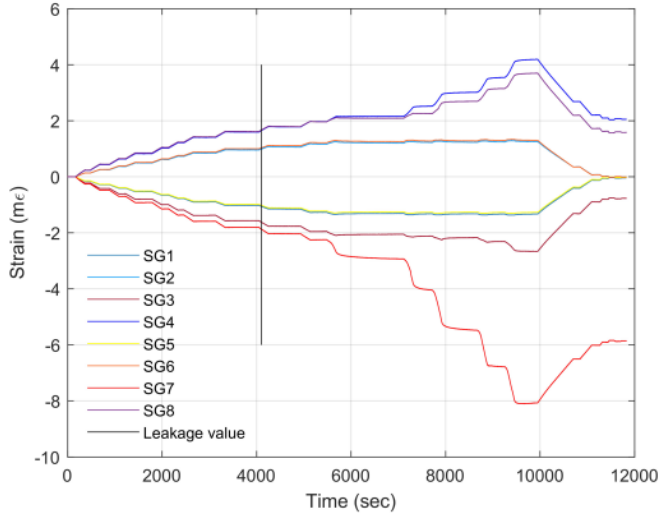
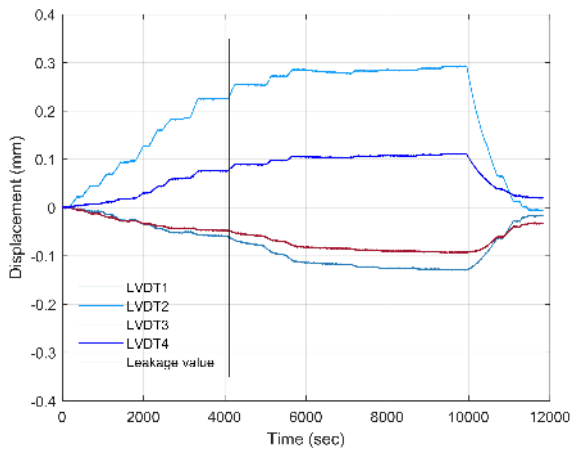


Figure 4.9: Force-displacement relationship.



(a)



(b)

Figure 4.10: (a) Deformation evolution measured by strain-gauges; (b) relative displacement histories of the flange joint measured by LVDTs.

To better understand the behaviour of the flange in both leakage and plasticization conditions, Figures 10b and 11 show, respectively, the relative displacement and rotation between the two flanges of the joint. Moreover, Figure 4.11 shows the corresponding moment-rotation relationship. The relative rotation was defined according to Reza et al (2014) [8], Section 3.4 Eqns.(1)-(2). In particular, the rotation is defined as the sum of two rotations: the relative rotation between the two flanges and the rotations of the pipe section adjacent to the flange. See Reza et al (2014) [8] for reference. The relative rotation corresponding to the leakage condition was equal to 0.015 rad, as shown in Figure 4.11, whereas the corresponding moment is about 82.67 kNm. This value is fully compatible with the predictive model proposed by Bursi et al (2017) [41], Section 4.2 Eqn. (8). The referred mechanical model is based on the framework of the EN 1591-1, (2013) [42]. More precisely, the model considers the BFJ as composed of three main components: bolts, flanges and gasket. For each component, it evaluates the tensile/compressive forces involved, considering also the resulting forces due to internal pressure and external applied load conditions. Additionally, all these forces are referred to a generic design load condition. However, it is defined as an enhanced model since it takes into account also interactions with shear force. Results of its application are shown in Figure 4.12. As can be seen the estimated maximum equivalent traction applied to the BFJ is 1038.5 kN, corresponding to an equivalent leakage moment of $M_{leak} = 83.07$ kNm. The experimental data reveals to be in good agreement with the provisions of the analytical model, derived in Section 4.2 of Bursi et al. (2017) [41]. In fact, as shown in Figure 4.12, the experimental point lies outside the safety domain for leakage prevention, meaning the occurrence of a leakage scenario. The results recorded by traditional sensors have been compared with the results recorded by FBG optic fibers system. Figure 4.13-(a) and Figure 4.13-(b) show, respectively, the deformation and the relative displacement of the flanges. As can be appreciated, the trend of the plot and the values recorded are really close to the value recorded by traditional sensors and shown in Figure 4.10-(a) and Figure 4.10-(b). The FBG sensor US-5 have recorded a peak value of deformation of about $4.8 \text{ } m\epsilon$ that are fully comparable with the peak value of $4.2 \text{ } m\epsilon$ recorded by SG4 strain gauge. Similarly, the maximum relative displacement of the flange recorded by LVDT2 is very close to that recorded by the LD-6 FBG sensor, Figure 4.13-(b).

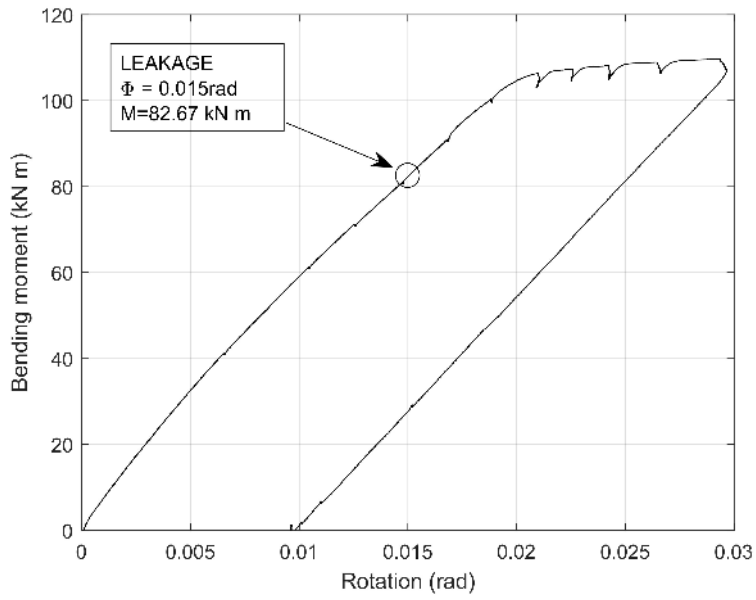


Figure 4.11: Moment – Rotation relationship of the BFJ.

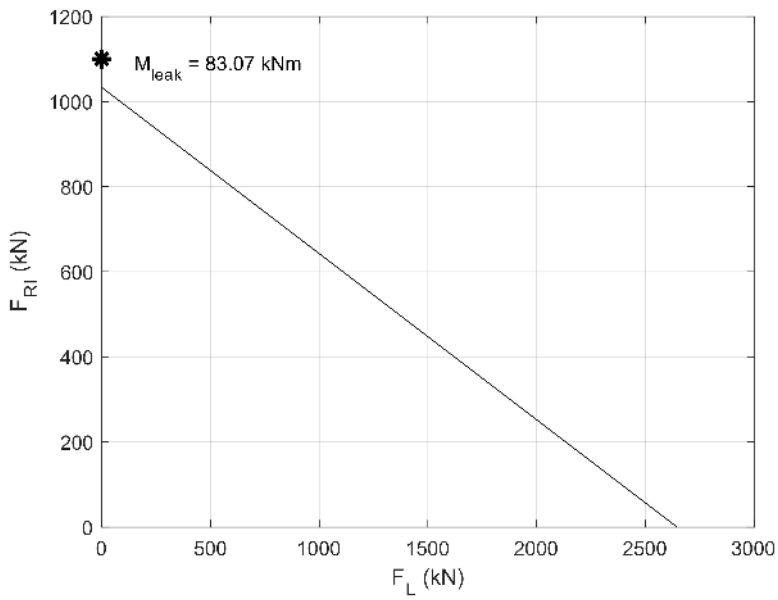
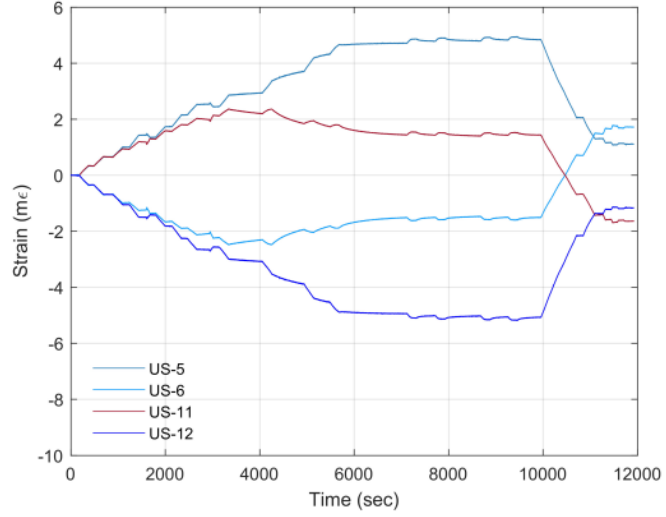
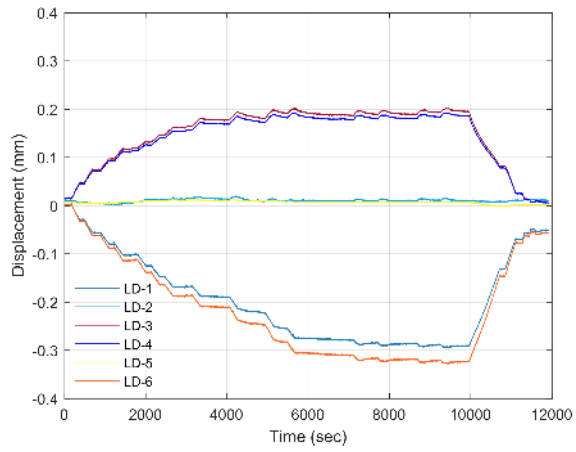


Figure 4.12: Leakage domain of the BFJ (after Bursi et al., [41]).



(a)



(b)

Figure 4.13: (a) Deformation measured by FBGs (b) Relative displacements of the flange joint measured by FBGs.

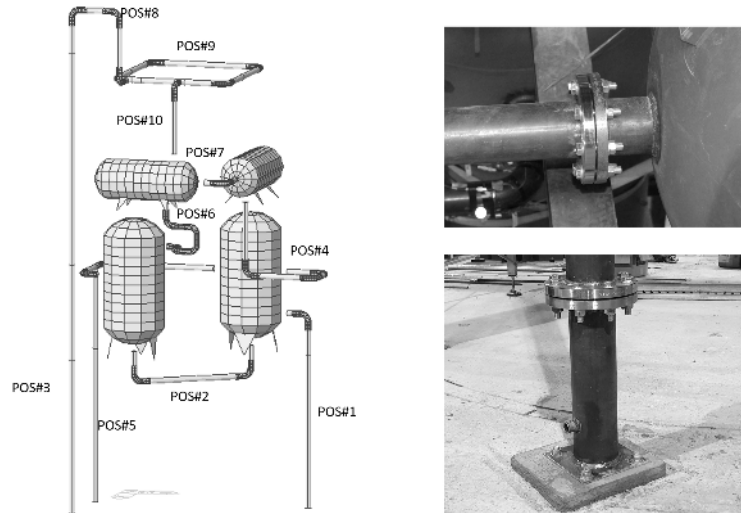
4.5 Experimental investigation on Bolted Flange Joints part of an industrial plant substructure

4.5.1 Description of Case study #2

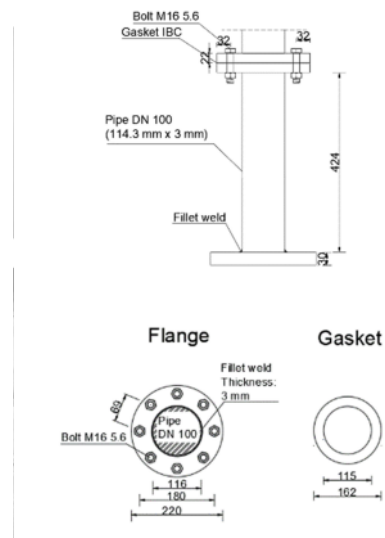
The second case study regards an industrial multi-storey frame structure with process equipment and pipes, which has been tested in a wide experimental campaign at the Eucentre laboratory in Pavia (Italy),[43]. The full-scale mock-up consists of a primary steel structure supporting several process units, i.e. the secondary elements.



Figure 4.14: Case study #2: Mock-up photographs at the EUCENTRE Laboratory (after Butenweg et al 2020, [43]).



(a)



(b)

Figure 4.15: (a) Piping layout and (b) flange joints details.

The main structure was conceived as a three-storey steel frame with flexible diaphragms made of crossbeams hinged to the frame beams. Figure 14 shows some views of the test structure on the shaking table. The structure is 3.7 m x 3.7 m in plan with a storey height of 3.1 m, which leads to a total height of 9.3 m. Two vertical and two horizontal tanks were installed at the first and second floor, respectively. Dimensions of the storage tanks and a schematic of the piping layout, together with some pictures of the flange joints are illustrated in Figure 15. More details on the structure and the experimental campaign results can be found in Butenweg et al (2020), [43]. The efficiency and robustness of fiber optical sensors have been tested also during the monitoring of two BFJs during a shaking table campaign performed on a typical industrial plant. The primary steel structure of the tested prototype industrial plant is a three-storey moment resisting frame with flexible diaphragm made of crossbeams. The piping layout is designed with DN 100 elements with 3.6 mm of thickness. Besides, test structure was equipped with a complex system of sensors which includes FBG fiber optics sensors for leakage detection, strain-gauges SG, LVDT and accelerometers, to study the dynamic behavior of the structure and its dynamic interaction with secondary elements (pipes and tanks). The shaking table test consists in applying an artificial generated accelerogram by increasing the PGA level up to 0.69g. Two critical BFJs (Pos. #4 and Pos. #6, Figure 4.15), belonging to a pipe connecting the vertical tanks at first floor to the horizontal tanks at second floor, have been monitored with both optic fibers GS4LD sensors and traditional linear potentiometers, in order to measure the flange opening and detect the leakage during the seismic test. Three SGs and three LVDTs have also been installed around the BFJs at 120° to each other, as shown in Figure 16, with the aim to evaluate damage conditions of the pipe.

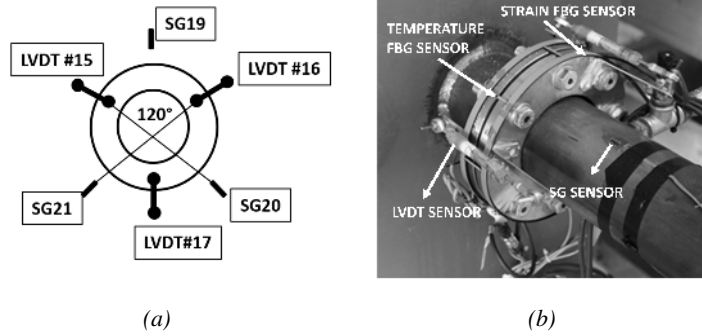
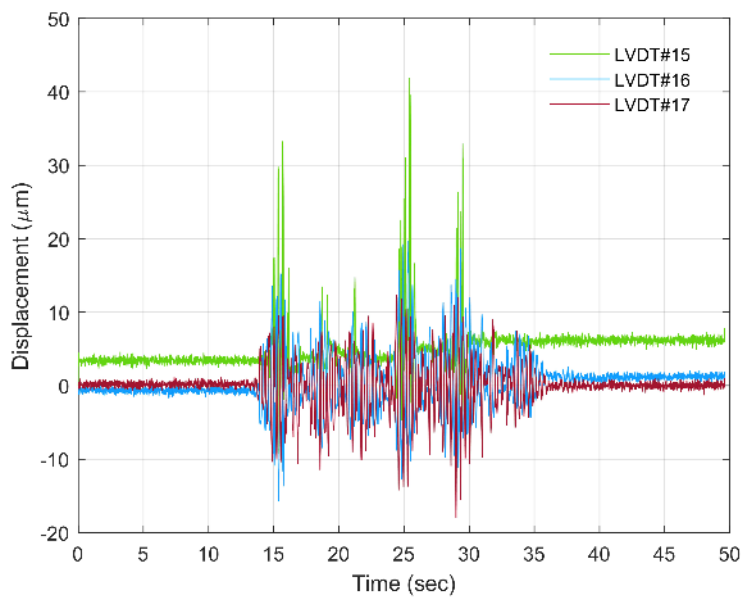
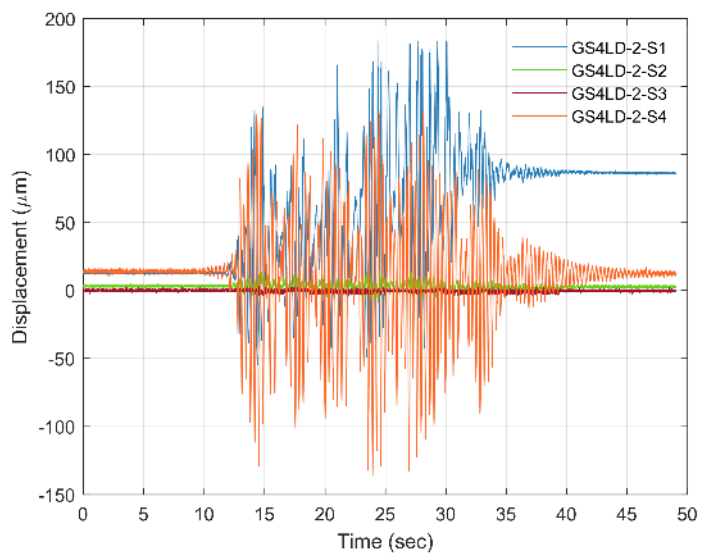


Figure 4.16: (a) Traditional sensors Layout (b) GS4LD optic fibers based Leakage Detection transducer installed on one of the two pressurized vessels mounted on the EUCENTRE shaking platform in Pavia.

For the application of the optical fibers, a transducer dedicated to monitoring liquid leaks was developed and engineered using both the non-regular coupling of the flanges control method and the measurement of temperature. The structure of the integrated transducer for GS4LD leakage detection is composed by two semi-sectors in composite material constituting the support on which the optical fiber travels, equipped with 4 FBG strain sensors and 4 FBG temperature sensors. Taking advantage of the technical and operative metrological features of fiber optic sensors, briefly reported in Section 4.4, both of the two GS4LD transducers, equipped with n. 8 FBG sensors each, were connected to one of the four channels of the GS4X interrogator positioned 60 meters away, without any risk of signal dispersion. By increasing the sampling of strain sensors up to 2.5 kHz, the GS4LD transducer can also be used for vibration monitoring. Figure 4.16-(b) shows the layout of the installed FBG system. As can be appreciated in Figure 4.17-(a), the T-H function results of the LVDT sensors show a maximum flange opening value equal to $42\text{ }\mu\text{m}$ recorded by LVDT #17. The latter have also recorded a residual plastic deformation corresponding to a displacement of about $7\text{ }\mu\text{m}$. This confirms the over conservatism of the pipe design for typical industrial pipe-rack, as already demonstrated in [43]. In order to reduce interference from unwanted noise due to power lines or power supply ripple, a finite impulse response (FIR) low-pass filter with cutoff frequency of 30 Hz was applied to recorded signals. In particular, the FIR filter uses a least-squares approximation to compute the filter coefficients and then smooths the impulse response with a Hamming window.



(a)



(b)

Figure 4.17: (a) Time history relations of flange opening recorded by LVDTs; (b) time history relations for flange opening recorded by FBG sensors.

Concerning FBG optical sensors results, while GS4LD-2-S2 and S3 have shown results very close to the LVDT sensors in terms of trend and peak value, GS4LD-2-S1 and S4 have shown a maximum flange opening value of about $160\ \mu\text{m}$. In particular, GS4LD-2-S1 have recorded a residual plastic deformation, corresponding to a displacement of about $90\ \mu\text{m}$ in the BFJ, as showed in Figure 4.17-(b). However, no leakage detection has been observed during all the test, as confirmed by the data coming from the FBG temperature sensors.

4.6 Summary and conclusions

The use of FBG optic fibers has been largely recognized to be effective in monitoring civil and industrial structures subjected to seismic loading conditions, thanks to the versatile employment of optical fiber systems both in distributed and localized structural configurations. Besides, applications in oil and gas industry are rapidly growing. However, exploitation of FBG sensors for monitoring hazardous conditions in Seveso industrial plants is still rather limited. This is mainly due to the scarce and limited experience in managing seismic damage conditions for hazardous plants, with a special attention to the identification of related consequences to both people and environment. In this respect, two important projects have been recently devoted to these aspects, whose main results were described above. The first one (MSMART) aimed at studying the use of Smart Sensors and, in particular, FGB sensors for the control of disastrous events in major-hazard industrial plants caused by earthquakes. The second one (SPIF) developed a realistic simulation through seismic shake table tests of an industrial multi-storey steel frame structure endowed with secondary process equipment and pipes. In view of a full implementation of the EU Seveso III directive, the results of these two tests were used as benchmark for the validation of FBG optical sensors in predicting potentially hazardous events. Both tests were focused on bolted flange joints (BFJs), identified in the literature as particularly vulnerable in terms of loss of containment (LoC) of hazardous material. The following conclusion can be drawn:

- FBG sensors used as traditional sensors for detection of critical strain and stress conditions appears fully credible. In fact, the comparison of the response with long-established sensors, i.e. strain gauges, linear displacement transducers, LVDTs, etc. demonstrated their reliability in monitoring structural quantities of interest.
- FBG sensors can also be profitably used for the assessment of BFJ and, in particular, for leakage detection in seismic conditions. Based on the Joule-Thomson effect, it is always possible to detect

LoC events using temperature sensors. The pressure drop due to the LOC event generates a temperature decrease easily detectable by an FBG sensor. The test on a single BFJ fully showed these capabilities.

- The use of FBG optical fiber sensors in monitoring BFJ, also in dynamic conditions, has been tested in a real-scale seismic simulation event on a prototype industrial plant. To the best of authors' knowledge, these experiments represent the first application of FBG sensors in realistic dynamic tests for BFJ monitoring system and leakage detection.
- Simplified installation of FBG sensors on BFJ and possible use of acquisition systems far away from on site condition environment, as demonstrated in the shake table test at EUCENTRE, render the proposed sensor systems especially suitable for hazardous environments, where the interaction with external elements needs to be limited.

Suitable sensor layouts for different equipment have been proposed based on the experience gained by damage survey after past seismic events. However, given the variety of situations that can be encountered in oil and gas industry, the identification of further advantages and limitations in the use of FBG for structural integrity and LoC of BFJ under seismic conditions requires further studies. Finally, to fully validate the use of FBG-based optical sensors, especially as leakage detectors, additional test programs on BFJ are needed.

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5. High fidelity model and seismic risk analysis of a vertical tank

This chapter contains the pre-print of

Quinci, G., **Nardin, C.**, Paolacci, F., Bursi, O.S. (2022).
Modelling issues in seismic risk analysis of Non-Structural
Components installed on industrial plant subsystems, *Bulletin
Earthquake Engineering*.

as in preparation for the submission in the Journal of *Bulletin Earth-
quake Engineering*, differing only in terms of layout and formatting.

Author contributions. **G. Quinci:** Methodology, Formal analysis, Data Curation, Writing - Original Draft. **C. Nardin:** Methodology, Formal analysis, Software, Data Curation, Writing - Original Draft. **F. Paolacci:** Supervision, Writing – Review. **O.s. Bursi:** Supervision, Writing – Review.

Summary

The seismic risk assessment of industrial facilities, equipped with complex components, is still an open issue. In recent decades, various analytical approaches for Performance-Based Earthquake Engineering (PBEE) have been developed, but its accuracy in the case of those complex critical infrastructures, such as nuclear and non-nuclear industrial plants, is under investigation. The goal of the proposed study is to examine, in a risk assessment framework, the critical aspects related to (i) the modelling of those industrial structures together with their process equipment considering the non-trivial dynamic interaction; (ii) the definition of reliable limit states and appropriate Engineering Demand Parameters (EDPs); (iii) the selection of proper seismic records for fragility analysis. To this purpose, an innovative algorithm for accelerogram selection is used. On these premises, two different configurations, i.e. a moment resisting frame (MRF) and a braced frame (BF) fully equipped with non-structural components (NSCs), recently tested by means of shaking table, are considered case studies. Vulnerability assessment of a critical NSC, i.e. the vertical tank installed on the mock-up, was carried out and its mean annual frequency (MAF) of exceedance for both configurations was evaluated.

5.1 Introduction

5.1.1 Background and Motivation

The seismic behaviour of support structures in major-hazard industrial plants (e.g. refineries) is of paramount importance as undoubtedly demonstrated by many contributions present in the literature, where researchers deeply analyzed the main issues concerning the analysis and component design of industrial plant substructures equipped with process components (pipes, tanks, etc.). Notwithstanding, several aspects are still unresolved, resulting in codes and standards often dated and inadequate, especially regarding consequences on humans and the environment. For example, [1] concluded that both European [2] and American [3] standards provide the same poor indications for assessing the seismic response of refinery piping systems, where support structure (rack) and components (pipes) are present. In particular, an allowable stress design approach is preferred, even though modern literature suggests a performance-based earthquake design (PBED) approach. The main obstacle to the widespread adoption of PBED is the need for accurate non-linear models, which are computationally expensive and require specific and high-standard skills. A few studies can be found in the literature where an explicit performance-based approach has been used for the seismic analysis of these special structures. For example, in [4], the application of the performance-based approach is illustrated through complete non-linear seismic analyses of two petrochemical piping systems. The authors concluded that a general over-conservatism in designing piping systems is typically adopted, even though it is neither economic nor rational. Moreover, the definition of performance parameters and limit states has a certain degree of complexity, given the scarcity of experimental data. An interesting attempt to rationalize this aspect can be found in [5]; however, the lack of experimental results on the local behaviour of industrial components still represents a critical aspect on which limited contributions are present in the literature ([6]). A significant contribution is provided by the SPIF experimental campaigns, [7] and [8], where two different rack configurations equipped with pipes and tanks are tested through a shaking table test and the local behaviour of Non-structural Components (NSCs) are deeply analyzed. Furthermore, in [9], the authors investigated the behaviour of non-standard bolted flange joints endowed with plastic deformation capacity, demonstrating that the release of elastic behaviour in case of strong earthquakes can be beneficial, also in the presence of potential release of hazardous materials. Furthermore, [10] used the hybrid test techniques to investigate a full-scale petrochemical pipe under strong

motions, demonstrating, also, in this case, that refinery pipes, which are designed according to the current codes, presents a clear over-strength. Along this vein, the identification of proper non-linear analysis methods for correctly evaluating limit states exceeding conditions represents another delicate issue. Dynamic integration of a structure, where explicit non-linear behaviour of primary and secondary structures are considered, is the most reliable analysis method; however, simplified approaches, for example, the use of static analysis or the use of floor spectra, are preferred. Nonetheless, these latter are not problem-free. For example, [11] demonstrated that the standard non-linear static analysis can not be used to assess the pipe-rack response and that the current standards' behaviour factor is improper. This derives essentially from the presence of non-building characteristics of this kind of structure [12]. The absence of a rigid floor and the dynamic interaction between primary (support) and secondary (components) structures are among the leading causes of these discrepancies [13]. In this respect, the type of analysis method becomes crucial. For example, [14] focused on developing floor acceleration spectra for an industrial concentrically braced frame (CBF) structure supporting a liquid storage tank. The authors demonstrated that in most cases, code-compliant analytical approaches over-predict the results with respect to (w.r.t.) analyses that explicitly take into account the interaction between the supporting structure and the installed component. A practical approach for the derivation of floor spectra has been proposed by [15], where simplified formulas derived from non-linear SDOF and MDOF models were proposed, which can accurately predict the spectral peak. These approaches were successfully adopted by [14] to estimate floor spectra for an industrial CBF structure, accounting for non-linearities. Special attention should also be paid to the consequences that a failure of supported components (e.g. pipe flanges and elbows, tanks, etc.) can induce on humans and the environment. [16] analyzed the problem in view of a quantitative seismic risk analysis (QRA) of major-hazard process plants. They highlighted that one of the critical points that caused a limited diffusion of QRA techniques is given by limited knowledge of the seismic behaviour of critical units and components, among which the support structures play a crucial role. Release of hazardous materials from secondary elements (pipes, tanks, etc.), even not directly considered by the current standards, could generate uncontrollable disasters, dramatically amplified by the domino effect [17]. Its relation with seismic damage represents the main problem in evaluating loss of containment (LOC), which is still under discussion ([18]). Therefore, it is clear the need for additional data coming from experimental campaigns where actual seismic conditions on support structures endowed

with secondary elements can be reproduced. Finally, a still open issue concerning units like industrial structures supporting critical equipment in seismic-prone areas is represented by a clear definition of limit states and related seismic action demands. While seismic hazard conditions-limit states dichotomy (including consequences) is clearly identified for civil structures, almost nothing can be found in the literature for hazardous industrial equipment. A shy attempt to imitate the consolidated approaches coming from the nuclear industry can be found in [10], where a typical refinery piping system was analyzed, referring to Design Basis Earthquake (DBE) and Safe Shutdown Earthquake (SSE). The first corresponds to the typical Safety Life Condition and is representative of a damage limit state, whereas the second is related to near collapse conditions. However, these results should be extended to other types of structures, with the identification of the return period of the seismic action corresponding to a real DBE and SSE condition. From the above picture, it is evident that a large number of unresolved issues need to be investigated, which are mainly related to the lack of data. Furthermore, really poor contributions concerning the seismic risk assessment of the NSC can be found in the literature [19]. Consequently, the SPIF project [7] and [8] has been conceived to fill this gap by testing a full-scale industrial plant substructure equipped with process components under real seismic conditions in order to identify the seismic behaviour and the dynamic interaction of the elements. Given the experimental results of the cited campaigns, this work aims to carry out the seismic risk assessment of a common NSC, i.e. vertical tank, showing the issues correlated to the modelling that need to be, on one side, reliable in order to catch the seismic behaviour of this particular structure and, on the other side, quick in order to satisfy the requirement of a risk assessment.

5.1.2 Scope

Along this vein, the present study examines the challenges faced in modelling a 3-storey steel structure equipped with process components and subjected to an intensive experimental campaign on a shake table. In particular, the need to construct a sufficiently efficient and reliable finite element model is highlighted, balancing the computational burden required for a fragility analysis. Finally, the seismic risk assessment for a typical industrial NSC, represented in this case by a vertical tank, is carried out through the use of an innovative methodology for the stochastic selection of seismic records using the algorithm proposed and described by the Authors in [20].

The paper consists of 6 sections; Section 2 provides a brief overview of the SPIF project and its main results. Following this, Section 3 and Sec-

tion 4 provide an overview of the modelling problems encountered by the Authors, the solutions adopted during the modelling phases and the validation of FE models. Section 5, on the contrary, presents the seismic risk assessment framework for the NSC of the described case study. Finally, Section 6 summarises and draws conclusions and final considerations.

5.2 Description of the cases study

Within the European research framework Horizon2020, the SPIF project, i.e. *Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities*, was conceived and realized through extensive shaking table test campaigns on real scale industrial archetype substructures, [21]. More precisely, the objective of the SPIF project was the investigation of the dynamic interaction between the main steel structure and the installed process components or non-structural components (NSCs) in a performance-based earthquake perspective as well as the evaluation of the seismic performances of the two most widespread design configuration for industrial plants. The two different configurations, i.e. the moment resistant frame (MRF) and the braced frame (BF), were tested by a uniaxial shaking table with several PGA's levels of a scaled spectrum-compatible record and a synthetic ground motions *ad-hoc* selected for the experimental campaign, reaching the final intensity of 0.71g and 0.79g, respectively. An interested reader may find more information on the seismic input selection and the carried out test programme in [7] for the MRF configuration and [8] for the BF.

The tested mock-up is depicted in Fig. 5.1a-d. It consists of a three-level primary bay steel structure supporting horizontal and vertical tanks, piping installations with racks and several bolted flange joints (BFJs). The dimensions of the designed structure are 3.7m x 3.7m in plan with a storey height of 3.1m. The horizontal load-bearing system of the steel structure consisted of either two MRFs or two BFs in the direction of the seismic excitation, according to the tested configuration, and two braced frames realized with circular cross-section bracings in the transverse direction. A crossbeams system is adopted as a flexible diaphragm, fastened to the main frame beams by simple bolted connections with web stiffeners. These crossbeams served, in turn, as bearing supports for installing the NSCs. More precisely, four unpressurized tanks made of steel S235 – JRG2 with a yield strength of 235 N/mm² were installed as reported in Fig. 5.1a-d: i) two vertical tanks on the first floor, i.e. Tank #1 and #2; ii) and two horizontal tanks on the second floor, i.e. Tank #3 and #4, respectively. To simulate the liquid typically stored in such tanks and to protect the shaking table against liquid release, they were

filled with granular material with a density equal to water. Thus, to reflect typical configurations in industrial plants, tanks were mutually connected through a piping system consisting of nine $DN100$ pipes, with a 100mm diameter and a 3.6mm thickness, whose overall layout consists of straight branches, elbows, BFJs and tee joints made of $S235$ with a yield strength of 235 N/mm^2 .

Testing results demonstrated a clear, dynamic interaction between the primary steel structure and the secondary process components in both configurations.

More precisely, in the MRF case, the primary steel structure remained undamaged for safe shutdown earthquake (SSE) levels mainly due to its design and inherent flexibility; whilst in the BF case, the main structure exhibited almost a linear behaviour until the foreseen buckling of the bracing system. However, due to the use of common practice solutions for NSC components, damage appeared in web plates of fin plate connections between main members and supporting floor crossbeams at the first level. In fact, due to the intense rocking of the vertical tanks #1 and #2, fin plate connections experienced warping of the crossbeam's web and cracks in the transition zone from beam to web.

Concerning the NSCs performance, leakage phenomena at the monitored BFJs did not occur, as predicted by analytical formulations based on design codes, in any of the MRF and BF cases. Moreover, the vertical tanks at the first level experienced intense rocking phenomena in both MRF and BF configurations. Conversely, in terms of interactions with the main structure, they resulted in a different seismic performance. Owing to dynamic properties, in the BF configuration, the vertical tanks acted as tuned mass dampers (TMDs), and thus the relevant base shear at the first floor was limited; instead, in the MRF configuration, the favourable TMD effect was not relevant, as collected data and further studies revealed. Overall, it was shown that a proper experimental test campaign could: i) highlight complex interactions and potential damage in industrial substructures; ii) reveal the efficiency of different system configurations.



Figure 5.1: Photos of the SPIF's mock-up @EUCENTRE, Pavia: a) the MRF configuration; b) the two installed vertical tanks; c) details of the BFJ between vertical tank and piping system at 1st level; d) the BF configuration.

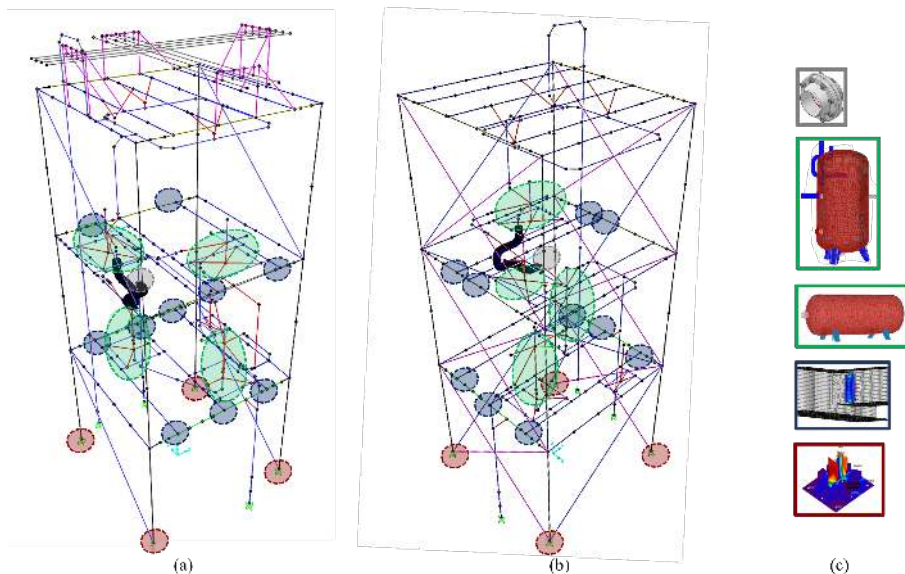


Figure 5.2: FE global models in SAP2000®: a) the global MRF model; b) the global BF model; c) local HF models and their locations in a) and b).

5.3 Multi-Storey frame and equipment modelling issues

To better understand the complex dynamics of the system, FE models of SPIF mock-ups were deemed necessary. The rationale behind developing these models was an optimal balance between computational demand time and reliability in assessing SPIF's experimental campaigns.

To do so, global models for the MRF and BF configurations, calibrated on local parametric studies on what was estimated as the utmost relevant components for those kinds of facilities, were implemented in SAP2000 software [22]. In particular, the following components were the object of a thorough, in-depth investigation: (i) for the main structure - base joints and fin plates connections; (ii) about piping system-elbows and BFJs; (iii) for NSCs - vertical and horizontal tanks.

5.3.1 Main Steel Frame Structure

Linear elastic elements for MRF and BF SPIF structures have generally been adopted. Thus, columns and beams have been modelled by Euler-Bernoulli elements; floor levels diaphragms have been considered flexible, and, therefore, no-rigid constraints for each floor level were applied. In particular, in-plane and out-of-plane stiffness contribution was only provided by the grid location of secondary beams, as depicted in Figure 5.2. Young's modulus of 220 GPa has been assumed, follow-

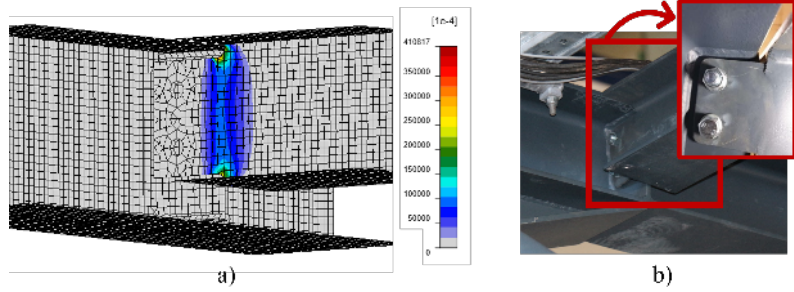


Figure 5.3: Fin plate: a) CBFEM strain analysis results; b) photos of the crack initiation on the mock-up.

ing the results of the model updating procedure described in detail in Section 5.4.1. Thus, thanks to the rational fraction polynomials (RFPs) identification technique applied to the gathered data of the experimental campaigns, a damping ratio with a value of 4.5% was introduced as suggested by European [23] and nuclear standards [24]. Further information on system identification can be found in [7] and [8]. Moreover, as testing results demonstrated, fin plate connections between main members and secondary beams were critical seismic details. Therefore, in order to reproduce the dynamic of the shaking table test campaigns and calibrate the moment-rotation relationship for local flexural plastic hinges, a component-based finite element model (CBFEM) analysis of the detail was carried out with the software IdeaStatica [25].

Results of the CBFEM analysis confirm severe stresses and significative strain levels in correspondence with the fin plate notch in the exact location where they were noticed during the experimental campaign. Figure 5.3a highlights strain values exceeding 12‰ yield threshold, thus explaining cracks initiation and propagation photographed in Figure 5.3b.

The moment-rotation curve evaluated by [25] was then applied to the flexural plastic hinge definition in SAP2000.

Furthermore, due to its complexity and relevance to the global response of the SAP2000 models, also base joint connections were thoroughly examined by referring to a CBFE local model developed with IdeaStatica [25].

Therefore, the rotational joint stiffness $S_{j,ini}$ the column base was derived, as reported in Figure 5.4a. Moreover, as reported in the graph, following §5.2.2.5(2) of EN 1993-1-8 [26], it was evaluated $S_{threshold} = 109.4 \text{ MNm/rad}$, through Eq. (5.2d) that discriminates between rigid and semi-rigid joints. Thus, it was possible to derive the equivalent rotational

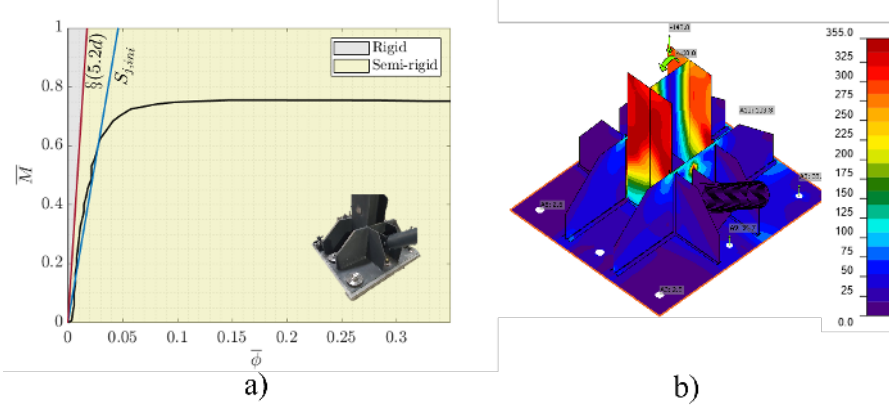


Figure 5.4: Base joint: a) semi-rigid joint classification, according to §5.2.2.5(2) of EN 1993-1-8 [26]; b) CBFEM of IDEA StatiCa® stress analysis results.

stiffness to apply to the base joint rotational link $k = S_{j,ini}L_c/EI_c$.

Finally, for the BF configuration, additional non-linearities were introduced to model the bracing system through nonlinear tension-only elements.

5.3.2 Piping systems and non structural components

To reduce the computational burden required by the analysis, piping systems and NSCs, such as vertical and horizontal tanks, were the object of dedicated parametric analysis. Several techniques are available for modelling both the inherent flexibility and stiffness of piping system layouts, see, e.g. [27], that includes different components such as pipes, elbows, tee-joints and bolted flange joints (BFJs).

According to [2], tee-joints and straight pipes were modelled by means of beam elements with reduced stiffness' values through the adoption of flexibility factors. Instead, elbows, highly susceptible to ovalization and warping, were modelled with straight beam elements when it was possible to ensure a sufficient cut-off length, see [28]. When this condition was not met, they were modelled with shell elements as depicted in Figure 5.2 for piping system Position #6.

Also, BFJs were the object of a comprehensive study due to their key-role related to loss of containment (LoC). In more detail, the examined BFJs category consists of DN100-PN16 joints with a loose flange on one side and solid on the opposite, sealed by 8 bolts M16 and a 2 mm aramid fibre gasket with NBR binder. To ensure leakage prevention, a minimum gasket pressure $Q_{s,min}^L$ equal to 6 MPa is assigned, corresponding to $L_{0,01}$ as a tightening class.

A refined FE model was developed in ANSYS [29] to evaluate ultimate resistance and sealing performances. The model, depicted in Figure 5.5b, takes into account contacts and material non-linearities through an appropriate non-linear pressure-closure curve obtained in compliance with [30]. Moreover, a refined meshing strategies was adopted: Table 5.5a and Figure 5.5b summarize the salient features. In particular, pipes, flanges and bolts were discretized by quadratic brick elements SOLID186, due to irregularity geometry issues. Whilst for gasket INTER195 element was adopted, since it is specifically designed for those kinds of flexible materials where primary deformation is confined into the gasket through-thickness direction.

MESH SIZE				ELEMENT
a.	Pipes	Hexahedral	5 mm	SOLID186
b.	Flanges	Tetrahedral	5 mm	SOLID186
c.	Gasket	Hexahedral	1 mm	INTER195
d.	Bolts - i.i.z.*	Tetrahedral	2 mm	SOLID186
e.	Bolts - o.i.z.*	Hexahedral	2 mm	SOLID186
f.	Welds	Tetrahedral	2 mm	SOLID186

* inside/outside influence zone

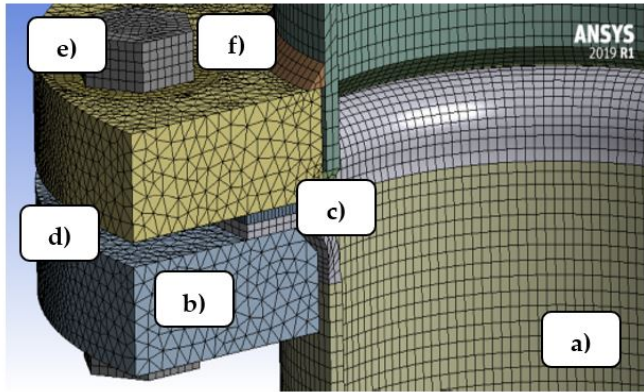
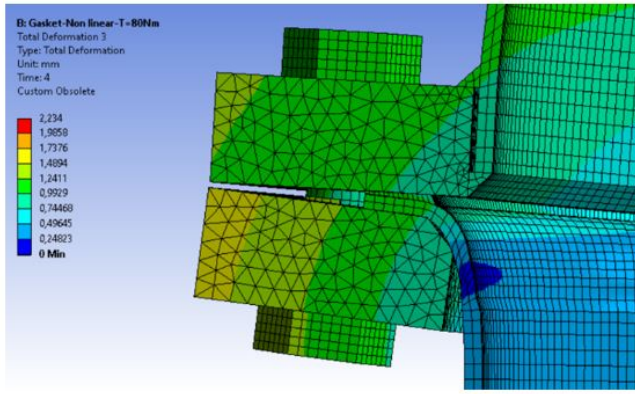


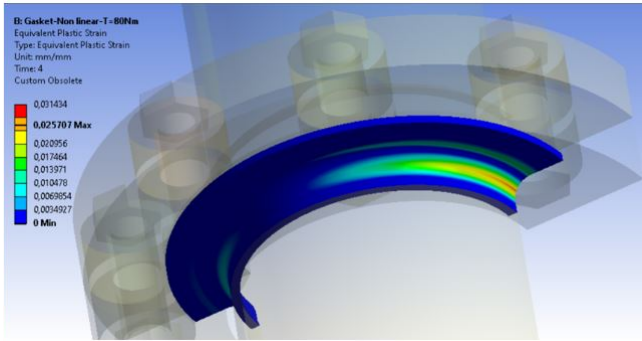
Figure 5.5: FE model discretization: (a) mesh size and element selected list; (b) mesh visualization and labels.

Results of simulations are reported in Fig. 5.6. In greater detail, Fig. 5.6a represents the ultimate deformation exhibited by the joint in seismic conditions; whilst Fig. 5.6b depicts ultimate strain levels. The analyses reveal a level of pressure distribution always less than the estimated $Q_{s,min}^L$, meaning no leakage could happen, as confirmed by visual

inspection along the test campaign. Moreover, as a careful reader could notice, two considerations emerge from Fig. 5.6a and b that explained why leakage was hindered: an i) significant rotation and an ii) higher slenderness of the collar that resulted in additional prying forces, thus inhibiting the onset of leakage. More precisely, the significant rotation of the loose flange, as can be noticed in Fig. 5.6a, generates a severe bending on the collar, leading to the formation of two plastic hinges. Thus, a plastic mechanism comparable to a T-Stub Mode 2 is developed. Hence, associated prying forces, due to the plastic mechanism, impose an additional gasket pressure at the external diameter: an extra-sealing performance, not considered by standard, is therefore triggered.



(a)



(b)

Figure 5.6: FE model results via ANSYS[®] of BFJ: a) deformation analysis; b) plastic strain analysis.

Moreover, an enhanced analytical predictive model for leakage, thoroughly illustrated in [31], was adopted to understand LoC scenarios deeply. In detail, the analytical model results in an interaction shear F_s

- axial forces F_{RI} leakage domain as shown in Figure 5.7. It has been evaluated with an internal pressure of 20 bar and a friction coefficient $\mu = 0.15$. More precisely, F_{RI} defines the equivalent resultant axial force at the load condition stage I , evaluated as the joint contribution of the applied axial force and bending moment to the BFJ. The resulting leakage domains for the SSE scenario of both the MRF and BF configurations are depicted in Figure 5.7a and Figure 5.7b, respectively. More precisely, for each configuration, two interaction domains were set: (i) a domain that takes into account scatter effects due to the employed tightening technique method, according to Annex C and Annex F of [32], depicted by a continuous black line; (ii) a domain that does not take into account tightening scatter effects w.o. safety factors and depicted by a dashed black line. Figure 5.7a and b collect predictions for all the installed BFJs on the two mock-up configurations and highlight in red the instrumented ones. Moreover, the initial tightening torque of 80 N·m applied in the MRF configuration was loosened to 40 N·m to simulate a possible maintenance issue's effect. Once again, the predictions confirmed the experimental evidence: no leakage happened.

Finally, according to [32], it is possible to estimate axial, transversal and rotational stiffness parameters to apply to the equivalent simplified mechanical model of the BFJ. More precisely, they were evaluated in compliance with

- Eq. (48) §5.2 of [26] for the axial stiffness:

$$K_a = 1 / (Z_F h_G (h_H + h_R) / E_{FI} + \dots) ;$$
- Tate and Rosenfeld formulation for the transversal stiffness $1/K_s = 10t^3 / 24E_b I_b + 4t / 3A_b G_b + 2tE_f / t^2 E_f^2$;
- by equilibrium considerations for the rotational stiffness: $K_r = M_{AI} / \phi_R = d_e d_{3e} K_a / 8$.

Last but not least, vertical and horizontal tanks have been thoroughly analyzed, given the crucial function they occupy in the system.

As reported in Fig. 5.8, both HF and LF models were developed. In particular, the LF results in a simplified stick model with an equivalent stiffness for each considered degree of freedom that has been evaluated and applied separately for each direction, as clearly depicted in Figure 5.8, based on modal characteristics and stiffness properties of the HF model. Concerning the HF, tanks were modelled by thick shell elements, assuming Mindlin-Reissner formulation. Moreover, as Figure 5.8a-c indicated, a refined mesh is adopted near critical discontinuities, such as nozzles, BFJs' connections and anchors. Steel grade class S235, with damping ratio 10%. To accurately evaluate the global seismic response and the seismic action effects on the supporting structure, it was assumed

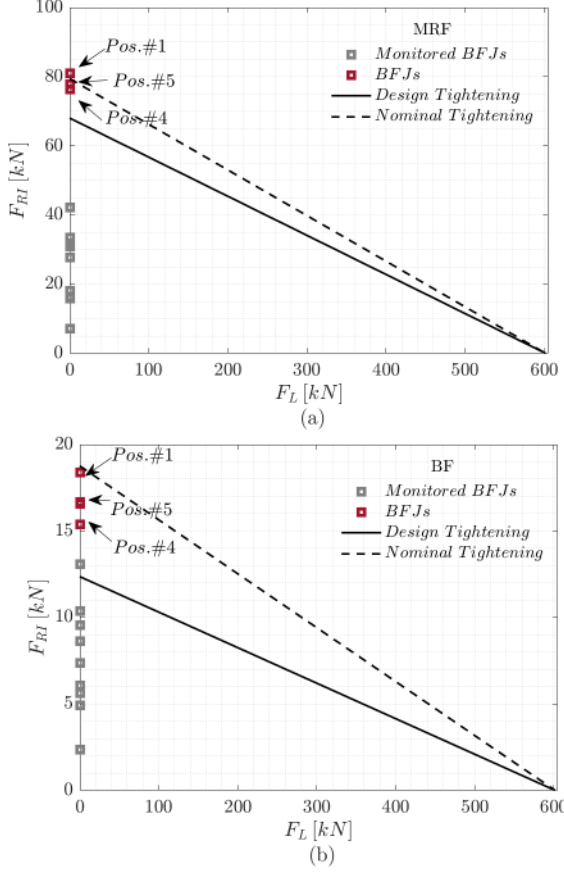


Figure 5.7: Leakage predictions and interaction domains with and w.o. safety factors for SSE in (a) MRF and (b) BF configurations.

that the particulate contents move together with the tank shell with an effective mass estimated through the modeling updating (MU) technique, thoroughly discussed in next Section 5.4.1. In more detail, 58% and 69% of participating mass w.r.t. the total mass of the insulated material were estimated by MU procedure for the MRF and the BF configurations, respectively.

Besides, to match the resulting stresses at base tanks footings, the residual masses, i.e. $m_{res} = m_{tot} - m_{eff}$ were applied at anchors levels. To capture also the effects on the shell due to the response of the granular material to the horizontal component of the seismic action, additional normal pressure on the wall $\Delta_{ph,s}$ was applied over the part of the wall in contact with the stored content, according to: $\Delta_{ph,s} = \alpha(z) \cdot \gamma \cdot \min(r_s^*, 3x) \cdot \cos\theta$; where x represents a vertical distance between

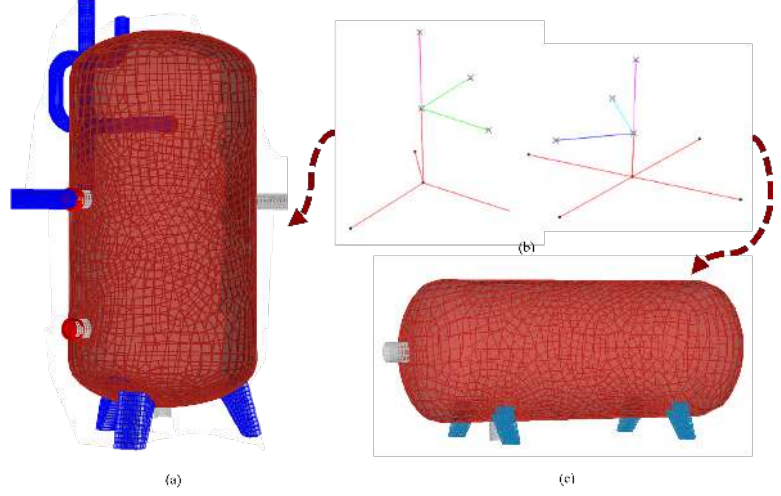


Figure 5.8: Local high fidelity models for (a) vertical and (c) horizontal tanks; (b) simplified stick models of the vertical and horizontal tanks.

a point on the tank's wall and the apex of the hopper; $\alpha(z)$ stands for the ratio of the response acceleration of the tank at a vertical distance z from the equivalent surface of the stored contents, to the acceleration of gravity; γ is the bulk unit weight of the stored granular material; $r_s^* = \min(h_b, d_c/2)$, where h_b is the overall height of the tank and d_c is the inside diameter; θ is the angle between the radial line to the point of interest on the wall tank and the direction of the horizontal component of the seismic action, according to §3.3(6-8), [33].

5.4 Numerical Simulations and Comparison with Experimental Results

The MRF and BF FE models were then calibrated through the Bayesian model updating technique by matching identified experimental frequencies and relevant mode shapes with the numerical ones. Then, validation was pursued by comparing the FE models' displacement and acceleration time histories with the experimentally acquired data.

5.4.1 Free vibration analyses and model calibration

In the last decade, Bayesian methods have become very popular in the area of finite element model updating (FEMU), thanks to their versatile formulation. Based on Bayes' theorem, the main concern of the approach is to determine the posterior *pdf*. Thus, the updating parameters θ of the FEM are obtained once the posterior *pdf* is solved, as illustrated

in Eqn. 5.1:

$$P(\theta|\mathbf{y}) \propto P(\mathbf{y}|\theta) \cdot P(\theta) \quad (5.1)$$

where $\mathbf{y}$ represents the experimental evidence. Parameters θ are regarded as random variables with prior distribution $pdf(\theta)$, which represents the *a priori* state of knowledge of the parameters. The posterior distribution $pdf(\theta|\mathbf{y})$ represents the state of knowledge on the parameters after data observations $\mathbf{y}$. According to Bayes' theorem, it can be seen as an update of the prior through Eqn.5.1, where $P(\mathbf{y}|\theta)$ is the likelihood of the updating process.

Generally, the complexity of the posterior pdf is proportional to the complexity of the targeted FEM and the number of updating parameters involved in the process. Moreover, an analytical solution of the posterior pdf is not always available, especially when the search space is relatively large and a significant number of parameters is involved. Therefore, using numerical techniques such as Monte Carlo (MC) algorithms is the most commonly employed practical approach. MC methods include Classic MC and Markov Chain MC methods [34], which both rely on the possibility of drawing samples from a target distribution and averaging along the paths. However, while classic MC methods draw samples independently from a fixed distribution, the latter MCMC produces a step-by-step random walk in the parameter domain. In this respect, the Metropolis-Hastings (MH) algorithm [35] is, in particular, the most widespread.

Based on the results of a sensitivity analysis, the following parameters were drawn crucial in the model updating process: θ_1 , steel structure Young modulus; θ_2 and θ_3 seismic activated mass per meter and effective height of the profile pressure of the insulated granular material for the vertical tanks; θ_4 base joint stiffness. For the sake of clarity, from θ_2 and θ_3 it can be derived the % of effective participating seismic mass as the ratio between the estimated experimental excited mass of the vertical tanks over the total mass insulated, as defined in EN 1998:8-4 [33], that resulted in 58% and 69% for the MRF and BF scenarios, respectively. As a careful reader can appreciate, a mismatch exists with the safety recommendation of EN 1998:8-4 §3.3(4), which assumes as effective mass the 80% of the total mass of the granular content applied in correspondence of the tanks' centre of gravity.

Before executing the MH algorithm for FEMU, prior distributions for the updating parameters are set. In particular, based on technical literature, for both the MRF and BF cases, θ_1 and θ_4 pdf s were assumed normally distributed. Instead, since no other recommendations, despite

what is suggested in EN 1998:8-4 are available, uniform distributions are selected for θ_2 and θ_3 . Distribution and parameters of the *pdfs* are collected in Tables 5.1 and 5.2 for the MRF and BF cases, respectively. An initial parameter guess is needed θ^0 . Then, according to the pseudo-code reported in 1, a candidate θ' state is generated, conditioned on the previous state of the sampler. This candidate can be either accepted or rejected: in an MH algorithm, the probability of accepting the proposal is the ratio $\beta = \text{pdf}(\theta'|\mathbf{y})/\text{pdf}(\theta^{(q-1)}|\mathbf{y})$ between the new posterior and the previous one. Thus, draw a random number u from a uniform distribution in $[0, 1]$: if $\beta > u$, the proposal is accepted, and the next state is set equal to the proposal; otherwise, the proposal is rejected, and the next state is set equal to the old one. This procedure continues until the sampling converges. After a burn-in time and by setting a huge N of order 10E4, the parameters converge, and the samples distribute according to the target posterior distribution.

Algorithm 1 MH scheme

- 1: start with θ^0 , the first set of parameters
- 2: **for** $q = 1, \dots, N$ **do**
- 3: draw θ' from a candidate density distribution $p_c(\theta^{(q-1)}, \theta')$
- 4: calculate the acceptance probability

$$a_c(\theta^{(q-1)}, \theta') = \min \left[\frac{p(\theta'|\mathbf{y}) \cdot p_c(\theta^{(q-1)}|\theta')}{p_c(\theta^{(q-1)}|\mathbf{y}) \cdot p_c(\theta'|\theta^{(q-1)})} \right]$$

- 5: draw u , a uniformly distributed random number
 - 6:
 - 7: **if** $u \leq a_c$ **then**
 - 8: $\theta^q = \theta'$
 - 9: **else**
 - 10: $\theta^q = \theta^{(q-1)}$
 - 11: **end if**
 - 12: **end for**
-

A remark concerning the likelihood: a normal distribution was assumed for updating the error between the experimental and numerical relevant frequencies; whilst a beta distribution with $\alpha = 5, \beta = 0.5$ for the error between experimental and FE mode shapes based on modal assurance criterion (MAC) was employed. Moreover, to deal with the underflow numerical issues, *log* formulation was adopted as reported in Eqn.5.2:

$$\begin{aligned}
P(\mathbf{y}|\theta) = & \sum_{i=1}^4 \log \mathcal{N} \left(\left(\frac{fn_i^{exp} - fn_i^{FEM}}{fn_i^{exp}} \right)^2, \mu_{fn}, \sigma_{fn}^2 \right) + \dots \\
& - \sum_{i=1}^4 \log \text{Beta} \left((1 - \text{diag}(\text{MAC}(\phi_i^{exp}, \phi_i^{FEM}))) , \alpha, \beta \right)
\end{aligned} \tag{5.2}$$

Table 5.1: Probabilities density distributions of the updating parameters for the MRF configuration: prior and posterior parameters and type of distribution for each θ_i .

			Prior PDF, Parameters	Posterior PDF, Parameters
θ_1	Young Modulus	[GPa]	Gaussian, $\mu = 210 - \sigma = 100$	Gaussian, $\mu = 220 - \sigma = 50$
θ_2	Part. mass of v.tanks	[ton/m]	Uniform, $a = 0 - b = 1.22$	Gaussian, $\mu = 0.78 - \sigma = 0.60$
θ_3	H. of pressure profile	[m]	Uniform, $a = 0 - b = 1.87$	Gaussian, $\mu = 0.25 - \sigma = 0.1$
θ_4	Base Joint stiffness	[MNm/rad]	Gaussian, $\mu = 20 - \sigma = 5$	Gaussian, $\mu = 22.5 - \sigma = 2.5$

Table 5.2: Probabilities density distributions of the updating parameters for the BF configuration: prior and posterior parameters and type of distribution for each θ_i .

			Prior PDF, Parameters	Posterior PDF, Parameters
θ_1	Young Modulus	[GPa]	Gaussian, $\mu = 220 - \sigma = 50$	Gaussian, $\mu = 219 - \sigma = 30$
θ_2	Part. mass of v.tanks	[ton/m]	Uniform, $a = 0 - b = 1.22$	Gaussian, $\mu = 0.85 - \sigma = 0.55$
θ_3	H. of pressure profile	[m]	Uniform, $a = 0 - b = 1.87$	Gaussian, $\mu = 0.35 - \sigma = 0.1$
θ_4	Base Joint stiffness	[MNm/rad]	Gaussian, $\mu = 22.5 - \sigma = 5$	Gaussian, $\mu = 28.5 - \sigma = 4.5$

Finally, the predictive capabilities of the FE mentioned above models have been checked with results stemming from the experimental frequencies. Tables 5.3a and 5.3b gather numerical FE and identified experimental frequencies for the MRF and BF mock-up, respectively. Thanks to the updating procedure, a good agreement for the most relevant frequencies of the system is achieved. In general, the results showed a very small percentage difference, except only the second mode for the MRF. In this case, the difference between numerical and experimental recorded was 14%: this is probably related to the resonance effect between the second mode of the structure ($\simeq 4.80$ Hz) and the main period of the components ($\simeq 4.50$ Hz).

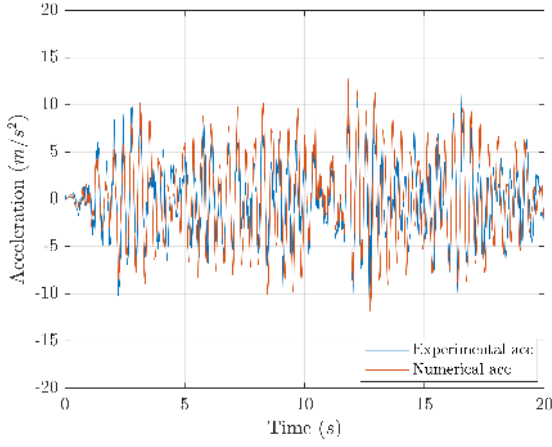
5.4.2 Seismic response analysis and results comparison

To assess the models' performance in capturing the structure's global behaviour and the dynamic interactions with NSCs, a comparison be-

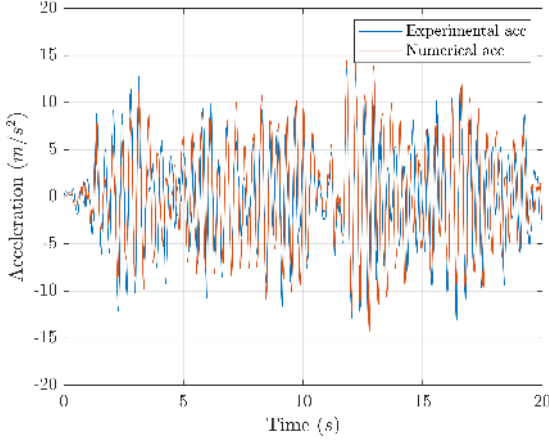
Table 5.3: Relevant model frequencies: numerical, i.e. evaluated through FE models; experimental, i.e. identified with rational fraction polynomials based on frequency response; Δ percentage as $(f_{n,exp} - f_{n,num}) / f_{n,exp} \cdot 100$ for (a) the MRF and (b) the BF configurations.

(a) MRF updated frequencies.				(b) BF updated frequencies.			
	Numerical [Hz]	Experimental [Hz]	$\Delta\%$ [-]		Numerical [Hz]	Experimental [Hz]	$\Delta\%$ [-]
f_1	2.81	2.88	2	f_1	4.54	4.50	-1
f_2	4.84	4.24	-14	f_2	6.36	6.83	7
f_3	6.73	6.38	-5	f_3	7.94	7.84	-1
f_4	9.18	8.95	-3	f_4	9.91	10.04	1

tween FEM results and those recorded during the experimental tests was conducted. In particular, to perform a seismic risk assessment for two of the key NSCs, namely the vertical tanks installed on the first floor, the Engineering Demand Parameter (EDP) is the acceleration recorded on the components. In detail, for numerical FE models, the acceleration is read at the stick model; while experimentally by the sensors placed on the tanks. For sake of brevity, in the follow, only the comparison of the results of the vertical Tank#1 are shown. Figures 5.9a and 5.9b show, for the MRF configuration, the numerical and experimental Time-Histories (THs) for the measured acceleration at the first floor and the one measured on the Tank#1, respectively. As can be appreciated, the model's ability to capture the general behaviour of the THs and, above all, the peak value of the acceleration, the parameter of interest for the fragility analysis, is significant. Similarly, Figures 5.10a and 5.10b, report the THs of in-plane and component accelerations in the case of the BF configuration. Again, a good agreement between the general and maximum trend is recorded. Moreover, a careful reader can notice from the comparison between Figure 5.9a and Figure 5.10a that the MRF primary structure is more excited than the BF configuration; whilst, the vertical tank in the BF configuration exhibits a higher peak acceleration value than that recorded in the MRF case, see Figure 5.9b and Figure 5.10b. As analysed by the Authors in more detail in [8], this highlights how the vertical tanks acted as tuned mass dampers (TMDs) in the BF configuration, effectively limiting the acceleration to the first floor and registering a higher peak acceleration themselves. Therefore, due to the dynamic properties, the favourable effect of the TMD was not relevant for the MRF configuration. The same behavior was observed for Tank#2 that shows good agreement between the experimental and numerical results as well.



(a)



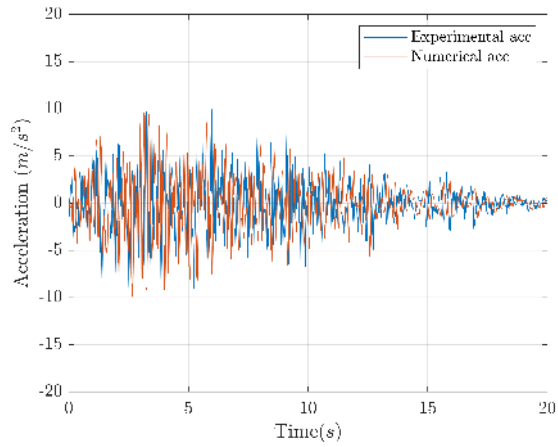
(b)

Figure 5.9: Numerical and experimental a) acceleration TH 1st floor MRF b) acceleration TH tank#1 floor MRF

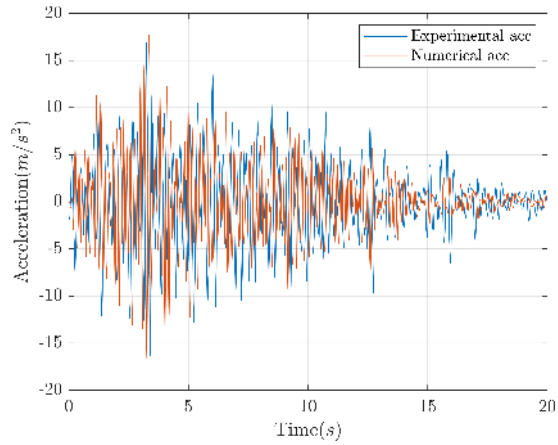
Finally, the relevant difference between the experimental and numerical results was measured by the normalized energy error (NEE) defined as:

$$NEE(x_n, x_e) = \frac{\sum_{i=1}^n x_{n,i}^2 - \sum_{i=1}^n x_{n,e}^2}{\sum_{i=1}^n x_{n,e}^2} \quad (5.3)$$

where x_i and x_e are the numerical and experimental acceleration values, respectively, and n is the length of the samples. Along this vein, Table 5.4 shows the values of the errors for both the configuration analyzed, with a focus on the acceleration TH functions for both Tank #1 and



(a)



(b)

Figure 5.10: Numerical and experimental a) acceleration TH 1st floor BF b) acceleration TH tank#1 floor BF

Tank #2. A careful reader can notice good accuracy for the prediction of the peak acceleration of the two tanks that show a 10% lower NEE.

Table 5.4: NEE values for the peak accelerations of Tank#1-#2 for (a) MRF configuration and (b) BF configuration.

MRF		BF	
	NEE		NEE
Tank #1	0.0748	Tank #1	0.0505
Tank #2	0.0653	Tank #2	0.0702
(a)		(b)	

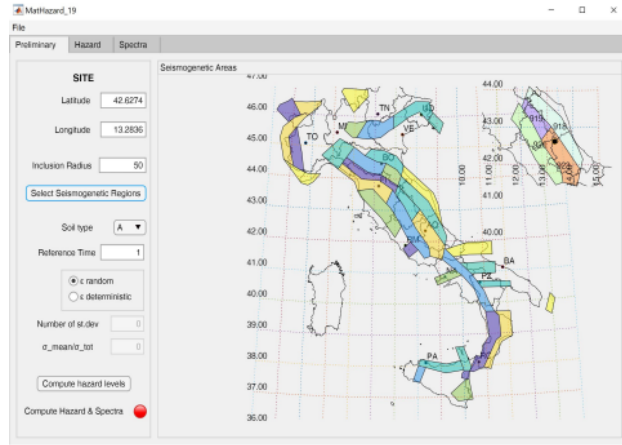
5.5 Seismic vulnerability analysis of industrial multi-storey frame structures

The risk assessment of the vertical tank#1 located ON THE first floor is carried out in this chapter, using the FEM models described in detail in the previous sections. A new algorithm for the records selection is herein adopted following the proposal of [20]. [20] showed the efficiency and robustness of the method applied to a 3D reinforced concrete building, allowing to avoid the criticisms related to the influence of the record to record variability on the vulnerability assessment and the choice of a reliable IM selection to represent the fragility curve. The following study aims to investigate the reliability of the method for an industrial Non Structural Component (NSC) where the dependence on the frequency content of the seismic input is very significant. In this regard, two different sets of 150 acceleration signals each, hereafter referred to as set a and set b, were selected and the risk assessment was evaluated with respect to two different limit states, in order to demonstrate the independence of the analysis from the selected set of signals. Along this vein, the following sections give a brief description of the algorithm, show the fragility and risk assessment analysis for the two sets of accelerograms and provide a final discussion of the results.

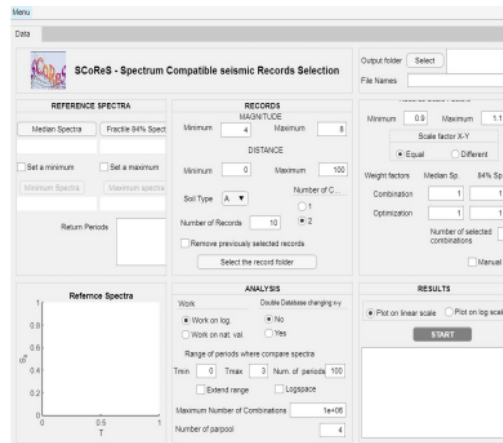
5.5.1 Input selection

The record selection was carried out with reference to the city of Amatrice, being infamous for the destructive 2016 Central Italy earthquake, [36]. Amatrice is located in Italy in the seismic zone 1, the most seismically vulnerable area according to national code [37], and is characterised by soil type B. Five return periods of 72, 224, 475, 975 and 2475 years, corresponding respectively to the probability of occurrence in 50 years of 50%, 20%, 10%, 5% and 2%, were assumed. Along this vein, a probabilistic seismic hazard analysis was carried out to derive both the hazard curves and the Uniform Hazard Spectrum (UHS). Two different

Matlab scripts, namely "Mathazard" and "SCoRes", were specifically developed for implementing the proposed methodology and used here. Figure 5.11 shows the dialogue box of the two applications.



(a)



(b)

Figure 5.11: Dialog box of the Matlab applications: a) MatHazard for PSHA, b) SCoRes for record selection

The idea of the applied method is to remove the dispersion of the record to record variability, fixing the dispersion ε from the Ground Motion Prediction Equation (GMPE). Along this vein, assuming that $\varepsilon=0$, we obtain the median uniform hazard spectrum (UHS) and assuming that $\varepsilon=0$, we get the fractile spectrum 84%. Finally, the algorithm allows to select the record to match the median UHS with the median spectrum of the accelerograms selected and to match the fractile spectrum 84% with

the median spectrum + one standard deviation of the records. More details of the proposed method can be found in [20] Figure 5.12 shows the generated UHS and the hazard curve developed considering ϵ completely random and deterministic ($\epsilon = 0, 1$). The hazard curves for $\epsilon = 0$ were adopted for the application of the proposed method.

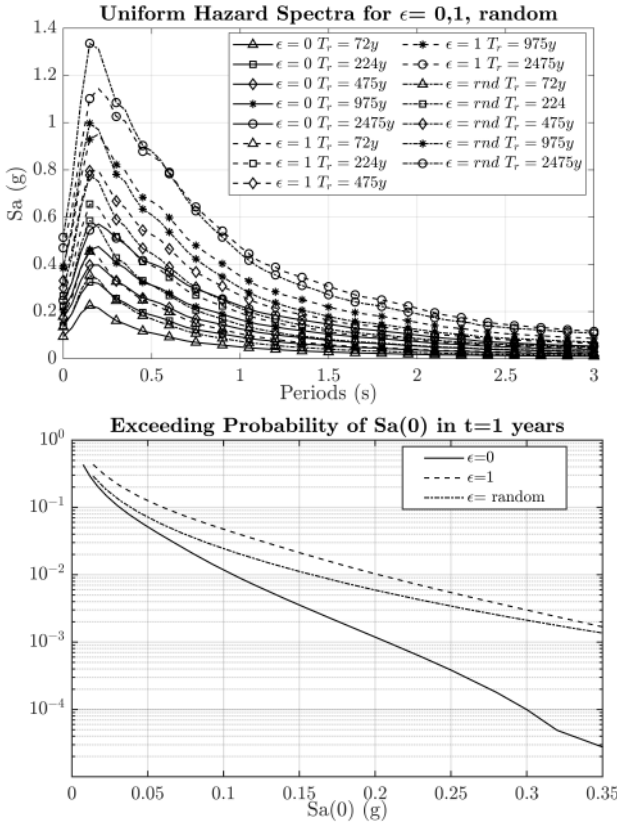


Figure 5.12: a) Target UHS for different return periods T_R , b) Seismic hazard curves of Amatrice(Italy)

The following parameters were taken into account when selecting the records from the database: magnitude range (4-8), fault-site distance range [0-100km], ground type B (according to the Italian Code). The spectra are compared in the range of periods [0, 3.0s] by using a hundred points. For each return period, a set of 30 natural records was selected using the SCoRes software. For this purpose, the weighting factors α and β were set to 1, both for the combination and for the optimisation. The scaling factors were constrained to a minimum value of 0.8 and a maximum value of 1.3, the maximum number of combinations was set to the

value $n_C = 5 \times 10^6$, whereas the other options were set to default values. Figure 5.13 shows the mean and the 84% fractile response spectra of the 30 scaled and unscaled records, for $T_R = 475 \text{ years}$, compared with the corresponding target UHS. The good agreement between the mean and dispersion of the five sets of records and the target spectra is evident. In particular, it can be noted that the already good agreement in the case of non-scaled accelerograms, is further improved by scaling the records along with the two principal directions by adopting the same scale factor F_s in the range 0.82-1.27. Table 5.5 shows the relevant records metadata for the return period $T_R = 457$ years for the set a. Once set a is formed, its records are deleted from the database and set b is selected using the same algorithm, ignoring the records already selected.

5.5.2 Fragility assessment

To perform a risk assessment for the vertical tank, first, the NSC's vulnerability is estimated. In this regard, two sets of 150 accelerograms each were selected using the methodology proposed in Section 5.5.1, to validate the robustness and effectiveness of the procedure. The fragility curve provides the probability that a fixed EDP, representing the structural response, exceeds a specific limit state threshold for a given value of Intensity Measure (IM), characterising the seismic demand. Peak Ground Acceleration (PGA) is chosen as IM in this study, whilst the peak acceleration level measured on the top of the tanks are adopted as EDP for fragility assessment. The rationale for picking EDP is strictly related to the availability of reliable and reference thresholds for the limit states, as thoroughly described below. Herein, two limit states are considered: the Design Basis Earthquake (DBE) and the Safe Shutdown Earthquake (SSE), as described in [8]. In particular, the tank bottom-wall strain value is selected as the reference EDP for both DBE and SSE limit states, in agreement with [6]. However, only acceleration data were available for both configurations since strain gauges were installed only on the BF case. Therefore, acceleration was correlated with the strain value thresholds for DBE and SSE limit states, as depicted in Figure 5.14, taking into account the experimental results described in [8] and resumed in Table 5.6. In particular, the following linear regression model was estimated $\varepsilon = p_1 * acc + p_2$, where acc is normalized by $\mu = 10.96$ and $\sigma = 4.139$ while for the p_i coefficients with 95% confidence bounds hold: $p_1 = 0.6441(0.5665, 0.7218)$ and $p_2 = 1.107(1.035, 1.179)$. A linear model was deemed accurate enough because of the following goodness of fit statistics: $SSE = 0.0274$ and $R^2 = 0.9891$ indexes.

Given the threshold values defined in Table 5.6, the fragility curve for each limit state is evaluated for each of the two sets of accelerograms.

Table 5.5: List of records selected for the hazard level $T_R = 475$ years. Set A

Event	Country	Year	Station	M	R _{JB}	F _s
Chi-Chi	Taiwan	1999	TCU056	7.62	10.5	1.26
Erzincan	Turkey	1992	ST207	6.60	76.0	1.27
Chuetsu-oki	Japan	2007	Joetsu Yanagishima paddocks	6.80	28.1	1.22
Norcia, Perugia	Italy	2016	Monte Cavallo	6.10	19.2	0.83
Loma Prieta	USA	1989	Gilroy Array #6	6.93	17.9	1.27
Ussita, Macerata	Italy	2016	Preci	5.90	8.80	0.83
Tabas Iran	Iran	1978	Dayhook	7.35	0.00	1.17
Mammoth Lakes-03	USA	1980	Convict Creek	5.91	2.70	0.91
APP. LUCANO	Italy	1998	LRG	5.60	6.60	0.84
Landers	USA	1992	Barstow	7.28	34.9	0.96
Kalamata	Greece	1986	ST163	5.90	11.0	1.26
Norcia, Perugia	Italy	2016	Poggio_Cancelli	6.10	36.5	1.15
Chi-Chi	Taiwan	1999	TCU053	7.62	6.00	0.83
Coalinga-01	USA	1983	Parkfield - Stone Corral 3E	6.36	32.8	1.19
Parkfield	USA	1966	Cholame - shandon Array #12	6.19	17.6	0.86
Mammoth Lakes-04	USA	1980	Convict Creek	5.70	1.40	1.27
Norcia, Perugia	Italy	2016	Matelica	6.10	46.2	0.82
San Fernando	USA	1971	Santa Felita Dam (Outlet)	6.61	24.7	0.82
Ano Liosia	Greece	1999	ST1100	6.00	16.0	1.28
Chi-Chi	Taiwan	1999	TCU015	7.62	49.8	1.10
Friuli Italy-01	Italy	1976	Tolmezzo	6.50	15.0	0.82
FRIULI EQK	Italy	1976	TLM1	6.40	21.7	0.82
Firuzabad	Iran	1974	ST3292	5.90	22.0	1.04
Norcia, Perugia	Italy	2016	Mascioni	6.10	40.0	0.87
Loma Prieta	USA	1989	San Jose - Santa Teresa Hills	6.93	14.2	1.23
Chi-Chi	Taiwan	1999	CHY019	7.62	50.0	0.96
Norcia, Perugia	Italy	2016	Mascioni	6.10	39.9	0.86
Ussita, Macerata	Italy	2016	Norcia	5.90	13.9	0.84
Loma Prieta	USA	1989	APEEL 10 - Skyline	6.93	41.7	1.26
Loma Prieta	USA	1989	Bear Valley #5 Callens Ranch	6.93	53.5	0.82

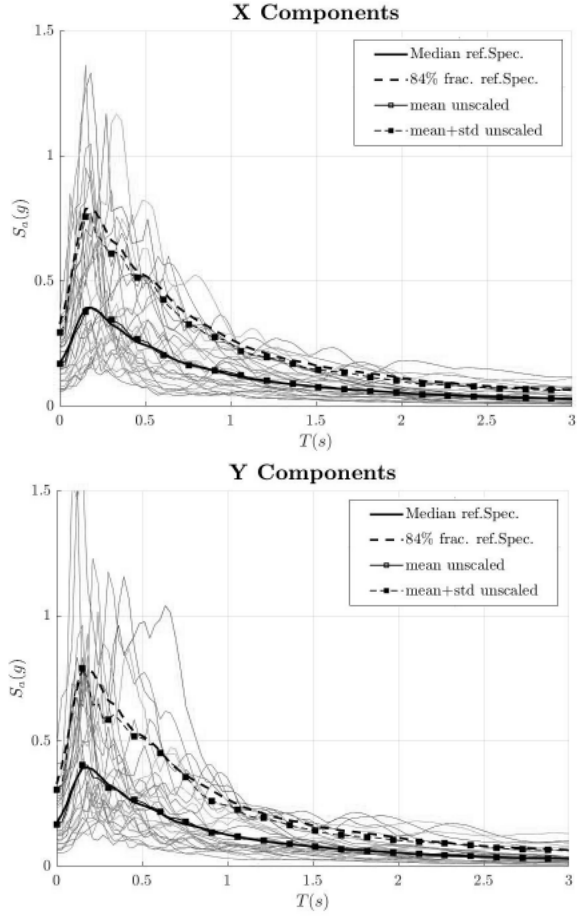


Figure 5.13: Spectra of the 30 unscaled accelerograms selected for the hazard level $T_R = 475$ years, spectra of their logarithmic mean and the mean + 1 std compared with the target spectra.

Table 5.6: Reference limit states

Limit state	Failure mode	Max tensile local strain (ε_T)	Max tank acc
DBE	Base plate fracture	$\varepsilon_y < \varepsilon$	$10m/s^2$
SSE	Base plate fracture	$\varepsilon_y < \varepsilon < 0.5\%$	$16m/s^2$

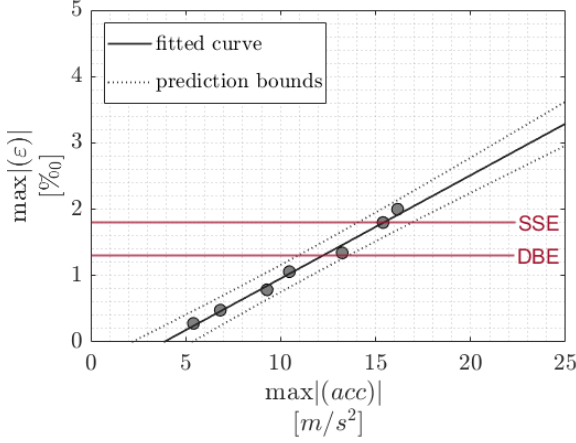


Figure 5.14: Maxima bottom-wall strain and acceleration experimental measures collected for Tank #1 and linear interpolation model with confidence bounds of 95%; $\max|(\varepsilon)| = \varepsilon_y$ threshold separating DBE from SSE limit state, according to [6].

Moreover, fragility curves are drawn for both BF and MRF configurations under the log-normal distribution assumption. Along this vein, the fragility curve is given by :

$$P(C|IM = y) = \Phi\left(\frac{\log(y/\theta)}{\beta}\right) \quad (5.4)$$

where $P(C|IM = y)$ is the probability that a ground motion with $IM = y$ will cause the structure to collapse; $\Phi(\cdot)$ is the standard normal cumulative distribution function (CDF); θ is the median of the fragility function (the IM level with 50% probability of collapse); and β is the standard deviation of $\log IM$. To evaluate the risk assessment of the tanks and to calculate the mean annual frequency (MAF) of exceeding the predefined limit states, a severe seismic zone is picked: Amatrice, Italy, as thoroughly described later on in Section 5.5.1. Figure 5.12 depicts the hazard curve of the site. [38] proposed to decouple the assessment of the seismic performance of a structure into two distinct sub-steps: (i) first, the Probabilistic Seismic Hazard Analysis (PSHA) as a feature of the site, then (ii) structural fragility, as, instead, a feature of the structure. Thus, if y is a parameter (or a vector of parameters) repre-

senting the seismic hazard, $\lambda(y)$ is the mean annual rate of exceedance of a given value y . Therefore, $P(C|IM = y)$ is the probability of exceeding a limit state C as a function of y and expressed as the marginal annual rate of exceedance of damage C , $\lambda(C)$, as follows:

$$\lambda(C) = \int P(C|IM = y) |d\lambda(y)| \quad (5.5)$$

5.5.3 Risk assessment and discussion of the results

Figure 5.15a and 5.16a depict the fragility curves, related to the DBE limit state, of the two vertical tanks for both the BF and MRF configurations. The curves refer to the two sets of selected accelerograms, i.e. set A and set B; at the same time, Figure 5.15b and 5.16b show the fragility assessment for the SSE limit state. A careful reader can notice that the vulnerability of the NSCs is higher in the BF configuration for both the limit states considered. These differences are more evident in the DBE limit state, while they are highlighted in the SSE limit state for PGA values higher than 0.2g, which is due to the higher EDP threshold that cannot be reached at lower PGA. These results are in good agreement with the experimental results described in [8], where the dynamic interaction between the primary structure and the secondary elements demonstrated a higher seismic response for the vertical tanks on the first floor of the BF configuration due to the TMD effects. Furthermore, considering the influence of the two sets for each configuration in the fragility assessment, it is worth noting that the fragility curves, especially for the DBE limit state, Figure 5.15a and 5.16a, are very close for the two selected sets. The differences in the SSE fragility curves can be reduced by increasing the number of selected accelerograms, although this also increases the computational time requested. Finally, integrating the fragility curves of the NSCs with the hazard curve of the site using Eq. 5.5, the MAF of occurrence was assessed for both limit states, and both accelerogram sets were considered here. Table 5.7(a) and Table 5.8(a) resume the value of MAF for the DBE limit states for both set A and set B, while the SSE results are shown in Table 5.7(b) and Table 5.8(b). The higher vulnerability of the tanks in the BF configuration is clearly readable in the DBE risk index, which shows twice the probability of the MRF configuration exceeding the threshold in one year. On the other hand, the difference in vulnerability between the two configurations does not carry over to the SSE risk index, where similar values are shown. The reason for this is related to the seismic hazard level of the site. As a matter of fact, as can be appreciated from Figure 5.12, the hazard curve shows a relatively low probability value for the site for PGA higher than 0.3g, where the

difference of the vulnerability for the SSE is more evident, pas shown in Figure 5.15b, so its combined contribution in the MAF evaluation is negligible. It emerges that risk indexes considering set A or set B reaches the same values for both the configuration analyzed. Therefore, the independence of the seismic risk assessment from the selected accelerogram sets is also verified in this NSCs application.

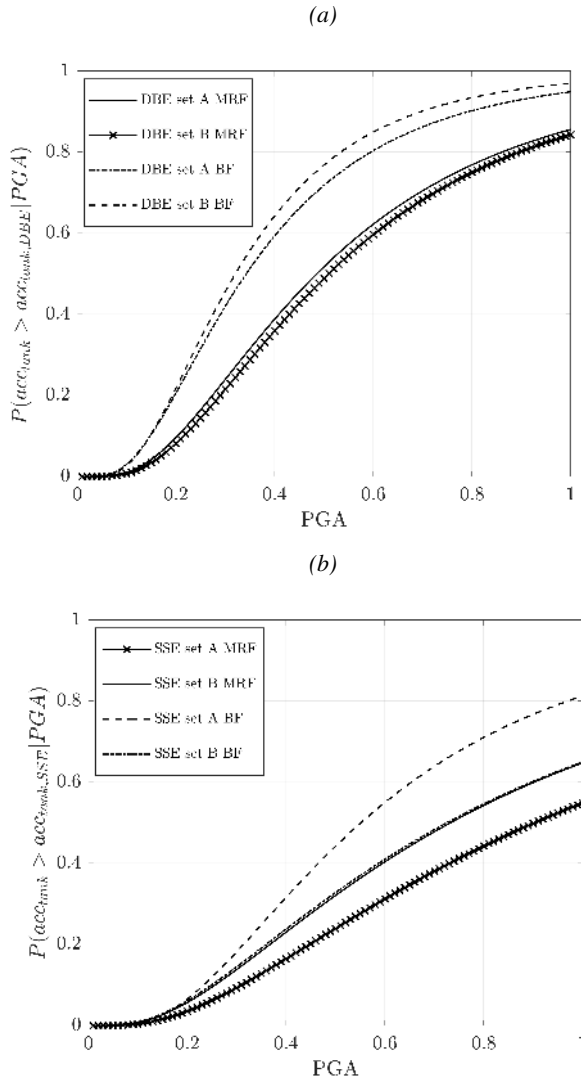


Figure 5.15: (a) fragility curves for DBE for MRF and BF set A and set B (b) fragility curves for SSE for MRF and BF set A and set B

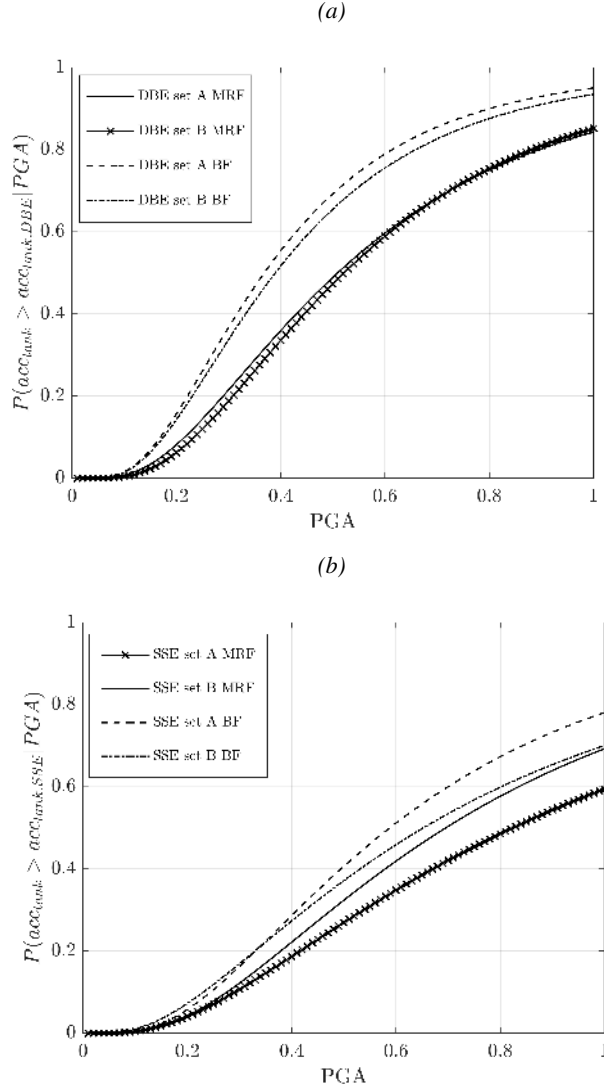


Figure 5.16: (a) fragility curves for DBE for MRF and BF set A and set B (b) fragility curves for SSE for MRF and BF set A and set B for Tank#2

5.6 Summary and Conclusions

This study aims to investigate the seismic risk assessment of a typical NSC, represented by a vertical tank, starting from what was observed during an extensive full-scale campaign conducted on a vibrating table. For this purpose, it was necessary to develop efficient finite element models that were, on the one hand, (i) reliable and, on the other hand, (ii)

DBE			SSE		
	Set A	Set B		Set A	Set B
MRF	0.00027	0.00024	MRF	0.00012	0.00014
BF	0.00054	0.00053	BF	0.00013	0.00015

Table 5.7: Mean annual frequency of exceedance for (a) at DBE and (b) SSE limit state conditions for Tank #1.

DBE			SSE		
	Set A	Set B		Set A	Set B
MRF	0.00022	0.00019	MRF	0.00012	0.00011
BF	0.00050	0.00051	BF	0.00010	0.00012

Table 5.8: Mean annual frequency of exceedance for (a) at DBE and (b) SSE limit state conditions for Tank #2.

accurate to the point of being able to capture the complex dynamic interactions between the main structure and components, which characterise this type of system.

In particular, attention was paid to a not secondary aspect represented by the computational burden: for a practical seismic risk assessment, a good trade-off between computational weight and calculation time must be found. This is so that acceptable results can be obtained relatively quickly. Furthermore, an innovative seismic record selection algorithm was used in this case study, which significantly reduces the age-old problem associated with record-to-record variability in selecting input sets for seismic risk assessment. Its efficiency was tested on an element, such as the first-floor vertical tank, which is highly dependent on the multimodal and high-frequency response of the system. For this purpose, two different sets of 150 accelerograms were selected. The results obtained from the analyses conducted on the two sets were then compared in terms of vulnerability and risk. Moreover, this study examined the design and performances of a 3D FE model validated against data acquired during shake table tests of the real mock-ups. The rationale behind implementing the models relies on finding an optimum between (i) accuracy and fidelity to the experimental data and (ii) the computational and time burden required by a single run of the FE model. On these premises, the strategy adopted in modelling involves the development of speedy and simplified global models. The simplification stems from a condensation of the degree of complexity of the problem through the separate development of (i) analytical models, such as for the BFJs' leakage domain;

(ii) stick models, such as for tanks; (iii) equivalent stiffnesses, such as for the base joint springs. In this fashion, the global model is enriched with information from local high-fidelity models. This reduces the overall calculation burden from, for example, seismic analysis of a single BFJ of $\approx 9h$, to an overall seismic simulation run time of the global model of only a few minutes. The time factor certainly plays a key role in the PBEE for the possibility or otherwise of conducting risk analyses. The built FE models revealed a good capability to catch the seismic behaviour of the NSCs, especially for the maximum acceleration value measured on the tank. As the optimal EDP for the seismic risk analysis, these maxima acceleration values were chosen. In addition, the seismic risk assessment has allowed highlighting the influence of the dynamic interaction between the primary structure and the installed equipment, as already the evidence of the experimental campaign revealed. Therefore, owing to the explored dynamical properties and the TMD effects, the results demonstrated the higher vulnerability of the tank located on the BF structure instead of the one in the MRF. Finally, applying the new methodology for selecting a set of ground motions demonstrated its efficiency by resulting in close indexes for vulnerability and risk assessment. Therefore, the methodology ensures the independence of risk evaluations from the adopted set of accelerograms: an undoubted advantage for designers. To enhance the understanding of these complex systems, further analyses on other NSCs are needed for reliability evaluations. In this respect, surrogate modelling and machine learning techniques could be applied to mitigate even more the required computational time.

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6. Metamodel state-dependent demand fragility curves for non-structural components in special risk industrial facilities

This chapter contains the pre-print of

Nardin, C., Marelli, S., Bursi, O.S. (2022). Metamodel state-dependent demand fragility curves for non-structural components in special risk industrial facilities, *Journal of Earthquake Engineering & Structural Dynamics*

as in preparation for the Journal of *Earthquake Engineering & Structural Dynamics*.

Author contributions. **C. Nardin:** Methodology, Formal analysis, Data Curation, Software, Writing - Original Draft. **S. Marelli:** Supervision, Methodology, Software, Writing – Review. **O.S. Bursi:** Supervision, Writing – Review.

Summary

Seismic fragility assessment of industrial plants and process equipment has gained momentum in recent years. This is reflected in the growing number of studies, and the increase of community-funded research projects and experimental campaigns on the matter. However, the problem's complexity and its inherent modelling, coupled with a general scarcity of available data and the model's limited capability, concurred in limiting or developing risk assessment methods based on meaningful reduced and quick-to-run models. On this background, an innovative method for developing state-dependent fragility functions is proposed. This new methodology combines limited data, both experimental and from refined FE models, with the power of well-known metamodeling techniques, i.e. the polynomial chaos expansion technique (PCE), combined with the bootstrap method. The method is first validated on a simplified and inexpensive-to-run MDoF system endowed with Bouc-Wen hysteresis. Then, it is applied to a more realistic industrial case of a vertical tank that proved critical during an experimental campaign on a shake table of a complex full-scale BF steel substructure equipped with process components.

Therefore, empirical fragility curves were derived based on the large number of outputs generated from the bootstrap and PCE combination. The best probability distribution was then inferred by employing the maximum likelihood estimation method and the Akaike criterion both for the undamaged and damaged structure, and the classical lognormal distribution assumption for the definition of fragility curves was verified.

6.1 Introduction

6.1.1 Background and motivation

Assessing structural and non-structural component vulnerability to earthquakes is a key step in modern probabilistic seismic risk assessment [1]. The PEER Performance-Based Earthquake Engineering (PBEE) framework has particularly gained momentum in this field, thanks to its inherently versatile formulation. Its strength, indeed, relies on simple yet effective implementation of total probability theorem, which allows to decouple and then combine the output of probabilistic seismic hazard analysis (PSHA) with fragility, damage, and loss analysis.

To this end, the fragility analyses step offers the critical link between seismic hazard and structural modelling, since it estimates conditional probability of exceeding or attaining a certain damage state (DS) given an intensity measure (IM) of earthquake motion. Initially introduced for nuclear safety evaluation [2], fragility curves are nowadays widely used, ranging from loss estimation [3] to assessment of collapse risk [4], from resilience quantification at individual to community scale [5]-[6], etc. In recent years, several novel methodological contributions to fragility analysis have been made, including the development of multivariate fragility functions [7], the introduction of seismic fragility analysis based on a combination of artificial ground motions and surrogate modeling [8] and the consideration of both state and time-dependent fragility [9].

Nonetheless, the majority of the past research focus was committed to characterizing the damage transition from a pristine state, i.e. no seismic damage, to a severer damage state for structures subjected to a single ground motion [7]-[9]-[10]. Instead, much less research attention was devoted to latest state-dependent fragility modelling, which can capture: (i) the damage accumulation due to sequential seismic events and (ii) enables fragility estimations of structures with different initial damage states. As a matter of fact, the compounding effect of damage accumulation and disruption caused by earthquakes in Wenchuan (2008), Tohoku (2011) and central Italy (2016) has clearly highlighted how accurately

capturing the effect of such irreversible damage accumulation under sequence of seismic events is instead crucial to reliable risk assessment of existing structures, so as to provide better inform decision making strategies. In this respect, recent researches as in [11] and [12] deeply investigated the performance of RC frames in terms of derived fragility curves that consider or not the effects of sequential seismic events and damage accumulation. The results demonstrated clearly a substantial difference in seismic losses assessment. Similar conclusions were reached by [13] for fixed and base-isolated steel frame structures.

Thus, a lot of effort has been devoted to investigate consistent procedures for selecting ground motions for event sequences, as in [13]. Drawing on the lessons of prior work and literature, Shokrabadi et al. [13] provide recommendations for selecting record pairs for use in sequential response history analysis and seismic performance assessment. In particular, the authors recommend the use of a seismic sequence (SS) of main-shock (MS) and after-shock (AS) record pairs, because they naturally capture and preserve within-sequence correlations. Besides, the use of SS-MS-AS record sets is preferred, since it places a focus on documentation of aftershock ground motions following major events as ground motion databases continue to expand and develop. However, to adequately populate an SS-MS-AS record set, ground motion models may serve the purpose, see [14]-[15].

On the other hand, advances in non-linear structural response simulations have yielded more accurate modelling of complex and multi-mode systems, allowing to gain better insights into critical issues and performance behaviour of structure and installed non-structural components (NSCs). A more thorough discussion of these issues is reported in [16]. Along this vein, significant efforts have been made in recent years to further investigate the coupling effects between main structure and NSCs, both numerically in [17]-[18] and experimentally in [19]-[20]. An important milestone may be found in NIST GCR 17-917-44, *Seismic Analysis, Design, and Installation of Nonstructural Components – Background and Recommendations for Future Work*, [21] that summarizes a year-long study undertaken to collect and document the body of available knowledge related to the seismic performance of NSCs for civil and, especially, industrial structures. Once again, NSCs for industrial plants were identified as a top priority in seismic risk assessment, since they account for the majority of direct property losses due to earthquake damage, [17]-[22]. As a matter of fact, costly and disruptive non-structural damage is much more widespread and can result in additional economic losses from functionality and business interruptions. However, performing non-linear dynamic analyses for those complex systems that

account for coupling interactions with NSCs is challenging and still limited by available computational resources. In addition, the computation of fragility analysis requires a large number of non-linear time history analyses (NLTHA). It follows that computational cost of NLTHA limits *de facto* the total number of simulations. This is a classical problem in simulation based uncertainty quantification (UQ), which is tackled by replacing the computationally expensive NLTHA of FE model with an equivalent surrogate model, as performed recently by [1]–[7]–[8]–[23]. Among surrogate models’ families, polynomial chaos expansions (PCE) [24], Kriging, or Gaussian process modeling [8], support vector machines, artificial neural networks [1]–[7] became the most popular in civil engineering domain, since usually they provide more than just an approximation to the model. For example, they might additionally yield analytical estimates of the response moments of the model, sensitivity indices or confidence levels for their predictions. In this respect, Marelli and Sudret [25] leverage combination of the bootstrap method and sparse regression to introduce local error estimation in the standard PCE predictor, thus calling the approach *bPCE*. The Authors showcased the effectiveness of this approach on both a simple truss and a complex realistic frame structure problem, by efficiently estimating the probability of failure of the systems within a reliability context.

6.1.2 Scope and core contribution

On these premises, this paper presents a novel method for deriving state-dependent fragility curves of industrial systems components. This approach blends data from a reduced number of complex and expensive sequential NLTH analyses with cutting-edge metamodeling techniques drawn from a strictly UQ context. In detail, the paper is organized as follows. Section 6.2 presents the heuristics and the theoretical background of the novel framework for fragility assessment. Then, in Section 6.3, two case studies are presented: an inexpensive-to-evaluate multi-degree-of-freedom (MDoF) system, adopted as a benchmark for the framework, and an industrial braced frame multi-component system endowed with process equipment, i.e. pipes, bolted-flange joints, vertical and horizontal tanks, subjected to a comprehensive shake table tests campaign. Thus, Section 6.4 deals with the computational FE and numerical models description. Then, in Section 6.5 the seismic state-dependent fragility assessment is carried out. More precisely, state-dependent fragility functions derived *via* Monte Carlo analyses and the bootstrapped-PCE approach are discussed and compared for the benchmark MDoF case study. Instead, for the industrial multi-component system, FE models and data gathered during the experimental tests cam-

paign, yield to evaluate *via* bootstrapped-PCE state-dependent fragility curves for a critical non-structural component, i.e. a vertical tank. The paper closes with the Section 6.6, where a summary of the main findings and future perspectives are reported.

6.2 Seismic fragility analyses framework

6.2.1 Theoretical background

Limited by the amount of time as well as by the amount of information, risk-informed assessments are commonly carried out on the basis of simplified and quick to evaluate models. However, these strategies have proven to lead to neither economic nor rational design strategies above all in special risk industrial plants, as highlighted in the recent NIST GCR 17-917-44 report [21]. On these premises, in this Section, the heuristics of a novel framework for fragility and, thus, risk-informed decision-making based on a combination between experimental data and surrogate models is outlined.

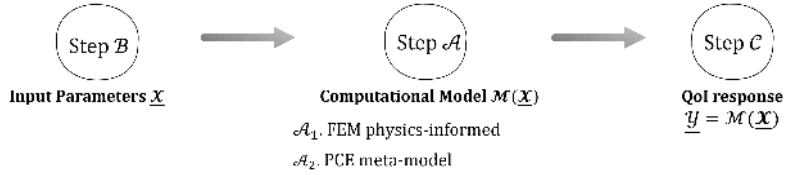


Figure 6.1: Steps of the global theoretical framework for UQ.

The theoretical backbone lies in the global uncertainty framework developed by [26]-[27] and sketched in Figure 6.1. According to this scheme, the following steps are all that is required to handle any UQ problem:

- $\mathcal{A}$: definition of a computational model $\mathcal{M}(\underline{X})$;
- $\mathcal{B}$: description of the input parameters $\underline{X}$;
- $\mathcal{C}$: propagation of the uncertainties through analyses and processing of QoIs

where $\mathcal{M}(\underline{X})$ could be numerical, analytical or FE model.

In particular, to derive state-dependent fragility curves, this scheme is applied twice: $\mathcal{B} \rightarrow \mathcal{A}_1 \rightarrow \mathcal{C}$; then $\mathcal{B} \rightarrow \mathcal{A}_2 \rightarrow \mathcal{C}$.

Figures 6.2-6.3 report the key aspects in a diagram-like manner. The difference between the two schemes relies on the computational model $\mathcal{A}$ that is assumed: in the former case, it is a FE model; whilst in the latter, it is a metamodel. In more detail, to derive reliable state-dependent

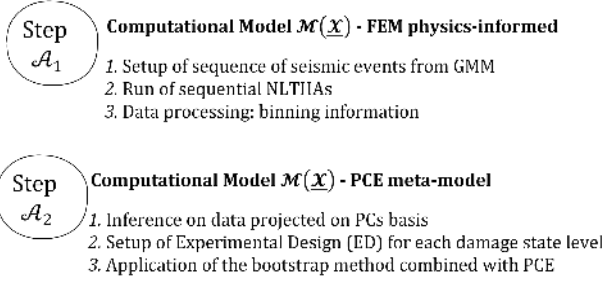


Figure 6.2: Heuristics $\mathcal{A}_{1-2}$: substeps for computational model, (i) FEM physics-informed and (ii) PCE meta-model.

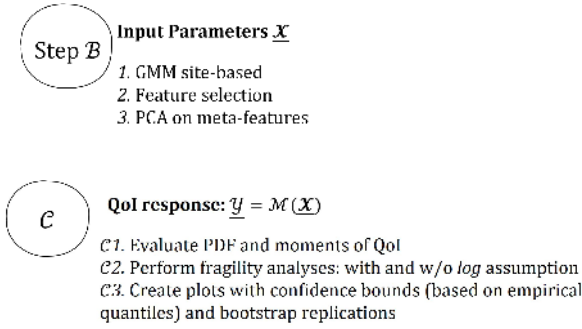


Figure 6.3: Heuristics $\mathcal{B} - \mathcal{C}$: substeps for input parameter description and post-processing of responses UQ forward analyses.

fragility curves for complex MDoF systems, the first scheme, i.e. $\mathcal{B} \rightarrow \mathcal{A}_1 \rightarrow \mathcal{C}$, is needed to populate the experimental design (ED) space for the latter scheme that involves PCE surrogate models. Indeed, the cost of simulations effectively hinders the direct application of the first method, thus leading to surrogate models. In this way, a combination of physics-informed insights with metamodel-derived data is achieved: a trade-off is reached between the physical complexity of the problem and the computational power demanded. Therefore, to effectively populate ED for each damage state level defined inside the fragility analysis, sequential non-linear time history analyses (sNLTHA) are performed. In this manner, it is possible to take into account both damage accumulation and ageing effects on the under-study system. A time history chain of several earthquakes is set up to create the sequence of events. In this respect, a ground motion model is deemed necessary due to the huge amount of seismic events involved. Thus, as schematically stated in Figure 6.2, sequence time histories QoI response of the FE model are binned, ac-

cording to the damage metric states, into simulations that overpassed or not limit states thresholds, given different initial limit state conditions. Sections 6.3 and 6.4 fully clarify this thoroughly. Once ED are defined, PCE metamodels for each initial damage state levels are created. In addition, to get local metamodel error estimates, bootstrap resampling is adopted. Finally, for the sake of clarity, steps $\mathcal{B}$, $\mathcal{A}_\infty$ and $\mathcal{C}$ are discussed in detail in the next Subsections 6.2.2, 6.2.3 and 6.2.4.

6.2.2 Step $\mathcal{B}$ - Input Parameters

The input to apply to the model $\mathcal{A}_1$ consists in a sequential chain of seismic events. To implement the chain and to populate the huge ensemble of seismic sequences, a site-based GMM is therefore adopted. Besides, in the second phase $\mathcal{A}_2$ of surrogate modelling, to deal with the dimensionality burden, principal component analysis PCA is applied to the input, which is handled as a set of random variables described by their joint probability density function *pdf*.

Site-based ground motion model

Recent examples of site-based stochastic ground motion models include the wavelet-based model of Yamamoto and Baker [28] and the non-stationary filtered white-noise model for far-field ground motions (GMS) [29]-[30]. A variant of these models that relied on an ensemble of selected natural records from the Italian INGV and ITACA databases was adopted in [19]. Along this line, the same far-field site-based GMM has been adopted hereinafter. Briefly, the stochastic model is based on a modulated and filtered white-noise process described by the Eqn. 6.1:

$$a_g(t) = q(t, \hat{\alpha}) \left[\frac{1}{\sigma_f(t)} \int_{-\infty}^t h(t - \tau, \lambda(\tau)) \omega(\tau) d\tau \right] \quad (6.1)$$

that incorporates both temporal - $q(t, \hat{\alpha})$, where

$\hat{\alpha} = \arg \min_{\alpha} \left(|I_a(t_{45}) - \hat{I}_a(t_{45})| + |I_a(t_{95}) - \hat{I}_a(t_{95})| \right)$ - and spectral non-stationarities, where $h(t - \tau, \lambda(\tau)) = f(\omega_f, \zeta_f)$ is the impulse-response function IRF.

The *pdf* of the GMM parameters, i.e. $\{I_a, D_{5-95}, t_{mid}, \omega_{mid}, \dot{\omega}_{mid}, \zeta_f\}$ gathered in Table 6.1, were inferred by the motion characteristics of an ensemble of selected natural records from the Italian INGV and ITACA databases, which fulfil the following assumptions: *i)* distance fault-site $R > 10\text{km}$; *ii)* moment magnitude $M_w > 5.5$; *iii)* main shock seismic events only; *iv)* strong motion intensities expressed in terms of $PGA > 0.075g$. The complete list of selected natural records and their characteristics can be found in [19]; and an ensemble of 1E4 GMS has been

generated.

Table 6.1: Probabilities density distributions of GMM's parameters.

Model Parameters		Units	Distribution	μ	σ	Distribution Bounds
I_a	Arias Intensity	[m/s]	Log-normal	-0.46	0.51	$(0; +\infty)$
D_{5-95}	Time interval of 95% of the I_a	[s]	Log-normal	2.21	0.23	$(0; +\infty)$
t_{mid}	Time interval of 45% of the I_a	[s]	Log-normal	1.698	0.21	$(0; +\infty)$
$\omega_{mid}/2\pi$	Filter frequency at t_{mid}	[Hz]	Uniform	4.8	1	[3.8; 5.8]
ζ_f	Filter damping ratio	[-]	Uniform	0.35	0.1	[0.25; 0.45]

* $\dot{\omega}_{mid}$, rate of change of frequency with t , is assumed constant and equal to -0.5

Features' selection through PCA

To define effective metamodels for the responses of QoIs, a reduced number of IM parameters (features) needs to be found; therefore a feature selection is performed herein. A wide range of time-, frequency-, spectral- and intensity-dependent IM parameters are summarized in Table 6.2, according to [31] and [32]. For each single ground motion, all $n_{IM} = 41$ parameters can be extracted. Thus, having N_{gms} ground motions, a matrix of $n_{IM} \times N_{gms}$ constructs the input domain. Ideally, having these features a ground motion record can be isolated, and the structural responses associated with it can be identified. Among the most popular PGA , PGV , PGD , $Sa(T)$, $Sv(T)$, $Sd(T)$, I_A IMs, also other novel and less widespread IMs are also introduced. Among the others, for instance, the Riddell–Garcia Intensity acceleration and velocity measures, which minimize dispersion of hysteretic energy-dissipation spectra of inelastic systems; the Fajfar Intensity, a compound IM that takes into account the damage capacity of medium-period structures; the $E - ASA_{R,x}$ equipment relative average spectral acceleration, an innovative IM that is able to take into account shifts in the frequency range of the structure due to damage experienced by the installed equipment. A complete list and description of implemented IMs can be found in Table 6.2.

Besides, Figure 6.A6 in Appendix A. illustrates the correlation among the IM parameters, which varies $[-1, +1]$. It clearly emerges:

- a high correlations for $E - ASA_{R,x}$ IMs, as also confirmed by recent studies, see [32], this IM tends to be advantageous especially when acceleration-sensitive NSCs are involved;
- the lowest negative correlations can be found between F_m , $\dot{F}_m$ and the other IMs;
- total durations showed negative correlations, consistently with the

Table 6.2: A comprehensive list of ground motion IM parameters [18]- [31].

No.	Description of IM	Symbol	Mathematical model
1	Peak ground acceleration	PGA	$\max(\ddot{u}(t))$
2	Peak ground velocity	PGV	$\max(\dot{u}(t))$
3	Peak ground displacement	PGD	$\max(u(t))$
4,...,8	Spectral displacement	Sd	$Sd(T_x)$
9,...,13	Spectral velocity	Sv	$Sv(T_x)$
14,...,18	Spectral acceleration	Sa	$Sa(T_x)$
	* where $T^* \in [0.50; 0.35; 0.25; 0.15; 0.10]$		
19	Arias intensity	I _A	$\pi/2g \cdot \int_0^{t_f} a^2(t)dt$
20	Total cumulative energy	E _{cum}	$\int \ddot{u}^2(t)dt$
21	Riddell–Garcia Intensity Acceleration	I _{RG,a}	$(PGA)^{+1} \cdot (T_d)^{+1/3}$
22	Riddell–Garcia Intensity Velocity	I _{RG,v}	$(PGV)^{+2/3} \cdot (T_d)^{+1/3}$
23	Significant time duration	T _d	$t_{95} - t_{05}$
24	Root mean square of acceleration	RMS($\ddot{u}(t)$)	$\sqrt{(\frac{1}{N} \sum_{n=1}^N x_N^2)}$
25	Characteristic intensity	IC	$\text{RMS}(\ddot{u}(t))^{1.5} \cdot T_d^{0.5}$
26	Cumulative absolute velocity	CAV	$\int_0^{t_f} a(t) dt$
27	Cosenza–Manfredi Intensity	I _{CM}	$2g/\pi \cdot (PGA)^{-1} (PGV)^{-1} (AI)^{+1}$
28	Average spectral acceleration	ASA ₄₀	$2.5/f_1 \int_{0.6 \cdot f_1}^{f_1} Sa(f, \varepsilon)df$
29	Acceleration spectral intensity	ASI	$\int_{0.1}^{0.5} Sa(T, \varepsilon)dT$
30	Effective peak acceleration	EPA	$1/2.5 \int_{0.1}^{2.5} Sa(T, \varepsilon)dT$
31	Velocity to acceleration ratio	I _{v/a}	PGV/PGA
32	Fajfar Intensity	I _F	$(PGV)^{+1} \cdot (T_d)^{+1/4}$
33	Mean frequency	F _m	$\sum_i U_i^2(f_i) / \sum_i U_i^2$
34	Rate of change mean frequency	$\dot{F}_m$	$dF_m(T)/dt$
35	Fourier amplitude spectrum area	FAS _{area}	$\frac{1}{4df} \int_{f_1-2df}^{f_1+2df} U(f)df$
36,...,41	Equipment relative average spectral acceleration	E – ASA _{Rxx} **	$\frac{1}{f_1 \cdot (X_f - 1)} \cdot \int_{f_1}^{X_f \cdot f_1} Sa(f, \varepsilon)df$
	** R indicates the chosen percentage of drop of the fundamental frequency ($X_f = 1 - (R/100)$); $R \in [40; 67; 80; 100; 150; 200]$		

physics of the linear dynamic analysis, where the ground motion duration does not directly play an important role. Instead, its effect is dominant in nonlinear analysis.

Finally, histograms and inferred *pdf* for the meta-feature are drawn and reported exhaustively in Appendix A..

To reduce the input order dimensionality, principal component analysis PCA is employed. PCA is a classical data analysis method that provides a sequence of the best linear approximations to a given high-dimensional dataset. It is one of the most popular techniques for dimensionality reduction. The subspace modelled by PCA captures the maximum variability in the data and can be viewed as modelling the covariance structure of the data. An overview in the context of dynamical mechanical systems is provided in [33], where the utility of the method for dynamic characterization and order reduction of linear and non-linear systems is demonstrated. Basically, PCA consists of an orthogonal transformation such that linearly uncorrelated variables- principal components PCs- with decreasing variances are obtained. Generally, the PCs are ordered in a way that the first PC explains most of the variance in the data, and each subsequent one accounts for the largest proportion of variability that has not been accounted for by its predecessors. Along this line, Figure 6.4 reports a scree plot: a graphic interpretation of the data variability captured by each PCs. As a careful reader can notice, the first three PCs are sufficient to describe more than the 80% of the data.

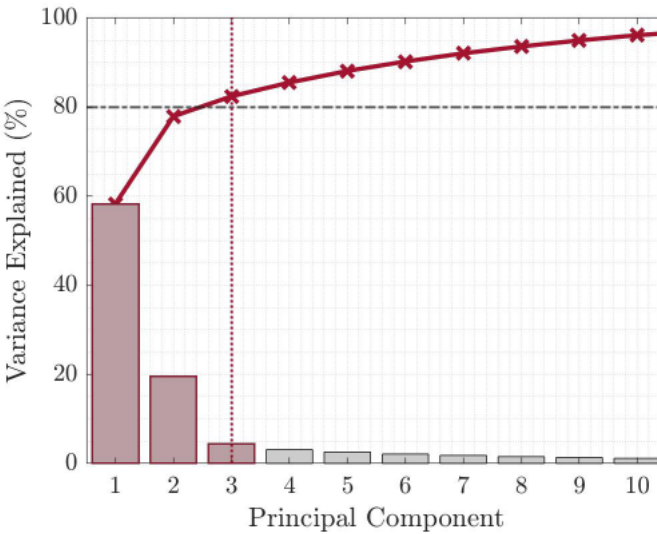


Figure 6.4: Scree plot for the GMM generated gms meta-features.

For the sake of clarity and to highlight which and how each variable contributes to the three relevant PCs, the orthonormal PCs vectors with the significant coefficients are reported herein:

$$\begin{aligned} PC_1 &= +0.202 I_{CM} + 0.198 E_{cum} + 0.190 E - ASA_{R_{200}} + \dots \\ PC_2 &= -0.279 I_{v/a} + 0.245 I_{CM} - 0.207 Sa_{0.50} + \dots \\ PC_3 &= 0.646 T_d + 0.430 CAV + 0.349 I_{CM} + \dots \end{aligned}$$

with meaning of terms given in Table 6.2.

6.2.3 Step $\mathcal{A}_2$ - PCE metamodel

Polynomial Chaos Expansions (PCE) is a powerful metamodeling tool that has important applications in many engineering and applied mathematics fields, see [34]. It provides a functional approximation of a computational model $\mathcal{M}(\underline{\mathcal{X}})$ through its spectral representation on a suitably built basis $\psi_\alpha(\underline{\mathcal{X}})$ of polynomial functions.

In real-world applications, due to limited computing power, the bases are truncated and the governing equation becomes 6.2:

$$\mathcal{Y} = \mathcal{M}(\underline{\mathcal{X}}) \approx \mathcal{M}^{PCE}(\underline{\mathcal{X}}) = \sum_{\alpha \in \mathcal{A}} c_\alpha \psi_\alpha(\underline{\mathcal{X}}), \quad (6.2)$$

where $\mathcal{A}$ is the set of selected multi-indices of multivariate polynomials. The corresponding expansion coefficients c_α can then be calculated efficiently via least-square analysis based on an existing sample of the input random vector $\underline{\mathcal{X}}$, i.e. on the ED of cardinality N , and the corresponding model responses $\mathcal{Y} = \{y^{(1)}, \dots, y^{(N)}\}$ as:

$$c_\alpha = \operatorname{argmin} \frac{1}{N} \sum_{i=1}^N \left[y^{(i)} - \sum_{\alpha \in \mathcal{A}} c_\alpha \psi_\alpha(\underline{\mathcal{X}}^{(i)}) \right]^2 \quad (6.3)$$

Adopting a least-square regression strategy to calculate the coefficients in Eqn. 6.3 allows using the bootstrap resampling method to obtain information on the variability in the estimated coefficients caused by the finite size of the ED. This can be achieved by first generating a set of bootstrap-resampled EDs $\underline{\mathcal{X}}^{(b)}$, then by calculating a corresponding set of coefficients $c_\alpha^{(b)}$. Therefore, the response of each b -PCE can be evaluated yielding in fact a family of b surrogate models that can be interpreted as *trajectories*. As recently explored in [25], this process of bootstrap-based trajectories resampling can be directly employed to assess the confidence bounds of the tackled UQ problem.

6.2.4 Step C - Fragility analysis

As the last step, fragility curves are evaluated to assess the vulnerability of systems to earthquakes. In particular, two classes of fragility are adopted: parametric *vs* non-parametric. The former consists of the classical approach: it relies on the assumption of the log-normal shape of the fragility through the maximum likelihood (MLE) method, see [35]-[36]. The latter, instead, consists of the empirical derivation, see [37], by deploying the huge ensemble of bootstrapped-PCE surrogates' results. This allows to verify and avoid the bias induced by the *a priori* log-normal distribution assumption. For completeness, in the classic approach, fragility curves assumes the following general form:

$$P(DS \geq d | IM = im) = \Phi\left(\frac{\ln(im_n) - \hat{\mu}}{\hat{\sigma}}\right) \quad (6.4)$$

where N is the total number of data, $\Phi(\cdot)$ is the CDF of the normal distribution and $\theta = \{\hat{\mu}, \hat{\sigma}\}$ are the parameters of the log-normal. Those latter might be estimated via MLE, clustering data into data that overpass defined threshold $n \in [1; \hat{N}]$ with $\hat{N} \leq N$, and data that do not overpass $n \in [\hat{N} + 1, N]$. Therefore, the likelihood function for the entire set of data is given by

$$\mathcal{L}(\theta) = \underline{\mathcal{L}}(\theta) \overline{\mathcal{L}}(\theta) = \prod_{n=1}^N \phi\left(\frac{\ln(im_n) - \mu}{\sigma}\right) \left[1 - \Phi\left(\frac{\ln(im_n) - \mu}{\sigma}\right)\right]^{(N - \overline{N})} \quad (6.5)$$

where $\phi(\cdot)$ is the standard normal PDF. Observe that in general the likelihood function can be written also for a generic PDF $f(x; \theta)$ (where θ is the set of parameters); and, thus, the generalized estimate of the parameter is found by optimization as

$$\hat{\theta} = \arg \min_{\theta} [-\ln \mathcal{L}(\theta)] \quad (6.6)$$

The same MLE method is then applied to empirical functions for inference among a broad class of probabilistic models and to verify if the log-normal assumption holds, see [38].

6.3 The case studies

This section presents two case studies where the framework for seismic fragility assessment discussed above is validated and applied. The first case, conceived as a benchmark, deals with the pursuit of state-dependent fragilities in terms of inter-storey drift (ID) ratio for a simplified multi-degree-of-freedom (MDoF) shear-type system with a hysteretic Bouc-Wen model of a multi-storey industrial frame, i.e. the SPIF

#2 braced frame (BF); more details are given below and in [16]-[19]. The second instead deals with the full model complexity of the SPIF #2 industrial frame equipped with process components, with a particular focus on fragilities for a critical NSC for the system, i.e. the vertical Tank #1 and its maximum acceleration at footing level.

6.3.1 The MDoF shear-type system with Bouc-Wen hysteresis

To test the validity of the proposed method, an equivalent mechanical no-cost to-evaluate system with Bouc-Wen hysteresis implemented is examined. The MDoF system represents a 2D simplification of the much more complex 3D problem of the SPIF #2 industrial frame, mentioned in [16]-[19]. The SPIF #2 project focused on investigating the seismic behaviour of an archetype multi-storey BF structure equipped with complex secondary industrial components by means of shaking table tests. To derive state-dependent fragility functions with ID-ratio as an engineering demand parameter (EDP), the simplified model aims to capture the response of the real system in terms of displacement and shear reactions time histories at floor levels, with as minimal computational effort and time as possible. This allows to perform a consistent number of sequential NLTHAs whereby state-dependent fragility functions can be derived. Thus, according to Table C1-3 *Structural Performance Levels and Damage* of FEMA 356 [39], the damage limit state thresholds for fragility assessment considered are 0.5% and 2.0%, for an Immediate Occupancy (IO) and a Collapse Prevention (CP) limit states, respectively.

6.3.2 The steel BF structure endowed with industrial process NSCs

The second case study stems from the aforementioned SPIF #2 extensive experimental campaign on the steel BF frame industrial substructure equipped with typical process components. Figure 6.7(a) reports a photo of the tested coupled multi-component system on the shaking table at EUCENTRE facility in Pavia, Italy. The mock-up consists of a full-scale 3-storey steel BF structure, with flexible floors. On the steel structure, several typical NSCs of the industrial process were installed, like tanks, cabinets, bolted flange joints, T-joints and so on. During the campaign, the complex dynamic interaction between the main steel structure and one of the NSC, i.e. the vertical tanks mounted on the first level, was revealed to be of particular interest. A detailed zoom of the investigated tank can be appreciated in Figure 6.7(a), along with the discussion of the adopted FE models in the next Section. The experimental campaign

revealed some criticalities in the performance NSCs: dynamic interactions with the main steel structure were significant. This holds true with particular regard to the vertical tanks installed on the first floor, that, as further FE analyses clarified, acted as tuned mass dampers. On these premises, combining the information gained during the extensive experimental campaign and in reference to the industry standard, see [19] and [40], respectively, two limit states were identified for the NSC EDP, i.e. the maximum acceleration recorded at tank's base-level: the first, linked to the operation and functionality of the process plant, namely the Design Basis Earthquake (DBE); the second related to the safe shutdown, thus called Safe Shutdown Earthquake (SSE). Thresholds are consequently associated with each limit state, in terms of the maximum acceleration recorded at the base of the tank of 9 m/s^2 for DBE and 18 m/s^2 for SSE, respectively.

6.4 Computational numerical and FE models

In this Section, the numerical and FE models of the MDoF shear-type system and of the fully equipped with process NSCs SPIF #2 mock-up are presented. The definition of the numerical models represents step $\mathcal{A}$ of the outlined framework for seismic fragility analyses in Section 6.2.1.

6.4.1 Numerical MDoF shear-type system model

The considered hysteretic MDoF is defined by the differential Equation 6.7:

$$\mathbf{M} \ddot{\vec{x}} + \mathbf{C} \dot{\vec{x}} + \mathbf{K} [\alpha \vec{x} + (1 - \alpha) \vec{z}] = -\mathbf{M} \ddot{u}_g \quad (6.7)$$

where the stochastic base input $\ddot{u}_g$ is derived from the GMM defined in the previous Subsection 6.2.2. The mass, $\mathbf{M}$, and stiffness, $\mathbf{K}$, are selected to match main frequencies of the experimental SPIF test campaign, thoroughly described in Section 6.4; while the damping, $\mathbf{C}$, is set to match a viscous damping ratio of 4.5%. The term α controls the degree of hysteresis and $\vec{z}$ follows the Bouc-Wen hysteresis law, according to [41] formulation:

$$\dot{\vec{z}} = \frac{A \dot{\vec{x}} - \{\beta |\dot{\vec{x}}| \vec{z}|^{n-1} + \gamma |\vec{z}|^n \dot{\vec{x}}\} \cdot \nu}{\eta} \quad (6.8)$$

All system parameters are given in Table 6.3, whilst Figure 6.5 depicts the hysteresis of the top DoF of the system under seismic excitation, i.e. at the third level of the equivalent system.

Concerning the excitation to assign to the system, a sequential chain of seismic events derived from the GMM of 6.2.2 is created. To be

Table 6.3: Bouc-Wen parameters for the simplified 3DoF system.

Parameter		Value
α	[-]	0.01
γ_N	[-]	0.500
δA	[-]	2.00E-12
n	[-]	1.5
β_N	[-]	0.1667
$\delta \nu$	[-]	1.00E-08
K_0	[N/m]	[1.72;1.77;0.96]E+07
M	[kg]	[9.5;14.5;12.4]E+03
$\delta \eta$	[-]	1.00E-08

precise, as a careful reader can notice, no assumption on main-shock and after-shock sequence correlation was drawn since it was beyond the scope of the benchmark test. In this regard, the research by Shokrabadi et al. [13] provides compelling considerations.

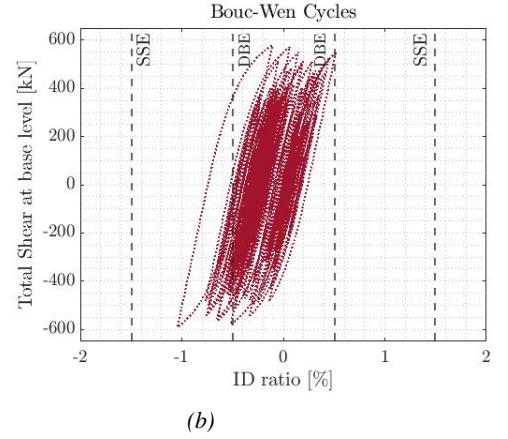
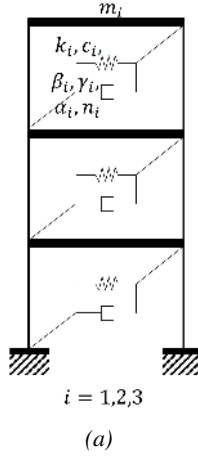


Figure 6.5: (a) Schematics of the simplified 3DoF system; (b) Bouc-Wen hysteresis for 1st DoF of the simplified system.

6.4.2 FE physics-informed model for SPIF #2

The considered FE model for applying the state-dependent fragility procedure represents the results of the recent SPIF #2 experimental research campaign. Figure 6.7(a) depicts the industrial substructure model tested on the shaking table, along with the FE model developed to assess the performance and investigate the complex dynamic interactions of the mock-up. In particular, Figure 6.7(b) reports a detail of the *ad-hoc* designed stick-model for the tanks' test. This simplified model was

developed with the goal to (i) mimic the modal properties and (ii) to catch the different effects of the participating seismic mass on the supporting girders system, related to the intensity level of the seismic excitation. This was accomplished by deploying complex and time-consuming high-fidelity local FE models, which were then used to assemble and calibrate the global physics-informed FE model in Figure 6.7(b). More details about the modelling and calibration can be found in [16]. Instead, a complete description of the carried out experiments and a discussion of the main results can be appreciated in [19] and [42].

The shake table tests showed that among the various components, precisely the vertical tanks were found to be critical and of interest: therefore, fragility analysis is focused on these elements, leveraging bootstrap PCE and campaign data insights. To this aim, applying the method outlined in Section 6.2.2, a total of 100 gms sequences, consisting of a chain of 5 earthquakes each, were created as input for the SPIF #2 FE model. The computational cost pro single sNLTHA is of 30 min on an Intel(R) Core(TM) i9-10900K CPU @3.70GHz, 10 Core(s) - 128 GB RAM. The number of chained gms and the total number of sNLTHAs are the compromises of a trade-off between the total time required by the analyses and a satisfactory population of each transition cluster. Anew, simulations are clustered according to the transition state, as summarized in Table 6.4:

Table 6.4: Transition states for SPIF #2: classification based on the initial-final reached damage level limit state. Size population of each cluster and its fragility definition.

Cluster ID	Initial L.S.	Ending L.S.	Realizations #	Fragility Functions Definition
0 – 0	0	0	135	$\mathbb{P}_{00} = 1 - \sum_{i=1}^2 \mathbb{P}_{0i}$
0 – 1	0	1	65	$\mathbb{P}[DS \geq DS_1 DS_0, IM = im] - \mathbb{P}[DS \geq DS_2 DS_0, IM = im]$
0 – 2	0	2	34	$\mathbb{P}[DS \geq DS_2 DS_0, IM = im]$
1 – 1	1	1	80	$\mathbb{P}_{11} = 1 - \mathbb{P}_{12}$
1 – 2	1	2	110	$\mathbb{P}[DS \geq DS_2 DS_1, IM = im]$
2 – 2	2	2	76	1

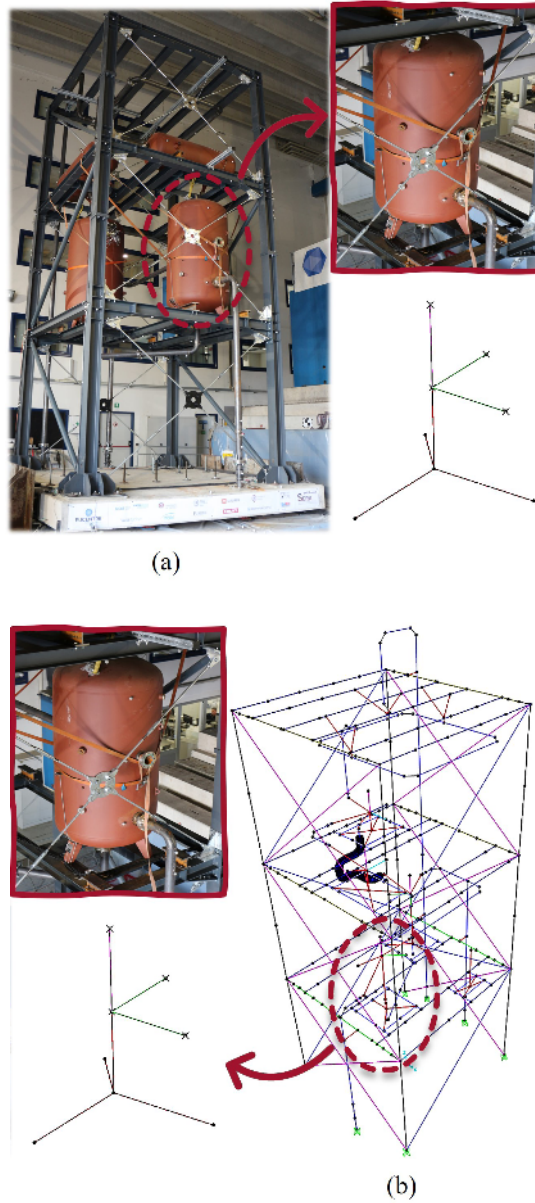


Figure 6.7: The SPIF #2 mock-up: (a) photo of the braced frame configuration on the shake table of EUCENTRE Facilities and detail of the vertical tank installed at the first level; (b) SAP2000®FE model and focus on the vertical tank ad-hoc implemented stick-model.

6.5 Seismic fragility analyses

In this Section, the seismic state-dependent fragility framework is applied to the aforementioned case studies. A validation of the method is presented in Subsection 6.5.1 by comparing fragility functions evaluated *via* Monte Carlo analyses with the ones derived by bootstrapped-PCE. Then, in Subsection 6.5.2 the framework is applied to derive state-dependent curves for the critical vertical tank of the SPIF #2 industrial BF structure. Moreover, insights on the classical *log*-shape distribution assumption for fragility functions are discussed for this latter case.

6.5.1 Benchmark: the MDoF system with Bouc-Wen model

Monte Carlo Analyses: Fragility Functions

To derive benchmark state-dependent fragility curves, a Monte Carlo approach was followed. To this scope, $1e3$ sequences of 3 seismic events for 50 different random seeds were generated. In this respect, in order to take into account the system's damage accumulation through time, sequential NLTHAs were deemed necessary. Thanks to a minimal computation cost of sequential NLTHA¹, a total of 150000 seismic simulations were performed. For each simulation, the initial and final damage configuration is identified, thus clustering it into transitions bins, according to the nomenclature defined in Table 6.5

Table 6.5: Transition states for the Bouc-Wen MDOF: classification based on the initial-final reached damage level limit state. Size population of each cluster and its fragility definition.

Cluster ID	Initial L.S.	Ending L.S.	Realizations #	Fragility Functions Definition
0 – 0	0	0	87250	$\mathbb{P}_{00} = 1 - \sum_{i=1}^2 \mathbb{P}_{0i}$
0 – 1	0	1	29200	$\mathbb{P}\left[DS \geq DS_1 DS_0, IM = im\right]$
0 – 2	0	2	500	$\mathbb{P}\left[DS \geq DS_2 DS_0, IM = im\right]$
1 – 1	1	1	32650	$\mathbb{P}_{11} = 1 - \mathbb{P}_{12}$
1 – 2	1	2	147	$\mathbb{P}\left[DS \geq DS_2 DS_1, IM = im\right]$
2 – 2	2	2	255	1

¹~3s on an Intel(R) Core(TM) i9-10900K CPU @3.70GHz, 10 Core(s) - 128 GB RAM

where the label '0' identifies the undamaged condition, whilst '1' and '2' refer to exceeding the thresholds of IO and CP damage limit state, respectively. Therefore, assuming the classical log-normal model, as reviewed in [35], fragility functions were evaluated separately for each transition's state and for each IM listed in Table 6.2.

Thus, Figure 6.8 represents the state-dependent fragility functions: each simulation is associated with a grey line; the curves associated with the 50% and 90% confidence bounds are plotted in dashed dark red, whilst the ones for the 99% are indicated with a solid line. Lighter to darker shaded areas in turn, identify the corresponding regions of the quantiles indicated above. The depicted fragility functions introduce $I_{RG,a}$ as IM, i.e. the Riddel-Garcia Intensity for acceleration. As the Author describes in [43], $I_{RG,a}$ is a compound IM that minimizes the dispersion of hysteretic energy-dissipation acceleration spectra of inelastic systems, according to: $I_{RG,a} = (PGA)^{+1} \cdot (T_d)^{+1/3}$, where T_d is the significant duration of the strong ground motion, $T_d = t_{0.95I_A} - t_{0.05I_A}$, i.e. the 90% of Arias Intensity I_A .

The choice of this IM derives from the application of the efficiency criterion characteristic within the vulnerability assessment; see in reference [31] and [35]. Hence, the optimal IM is the one whereby the variability of the EDP is reduced. Therefore, for each of the 41 IM identified in Table 6.2, the following efficiency index β_{eff} was evaluated:

$$\beta_{\text{eff}}(im) = \min \mathbb{E} (IQR|_{im,x1} + IQR|_{im,x2} + IQR|_{im,x3}) \quad (6.9)$$

where IQR stands for the 90th inter-quantile range evaluated in correspondence of the 25th – 50th – 75th IM's quantile axes. Table 6.6 collects the top four efficiency indices calculated for each transition state, i.e. from '0' to '1', '0-2', '1-2' and onto the same state '0-0', '1-1'. As a careful reader may notice, the progression to more severe damage states results in a different solution of the set of optimal IMs. For example, in the undamaged state '0-0', the optimal IM turns out to be $I_{RG,a}$, while in the damaged state '0-2', PGD becomes the new optimum. This is explained by the evolution of the physics of the problem: the mechanical and dynamic properties of the system change as damage accumulates. Thus, for a system initially characterised by a rigid response accurately described by quantities such as PGA or related to acceleration, the damage accumulation and the consequent plasticisation of the system induce a more flexible response, related to IM like PGD, $S_d(T_5)$ etc.

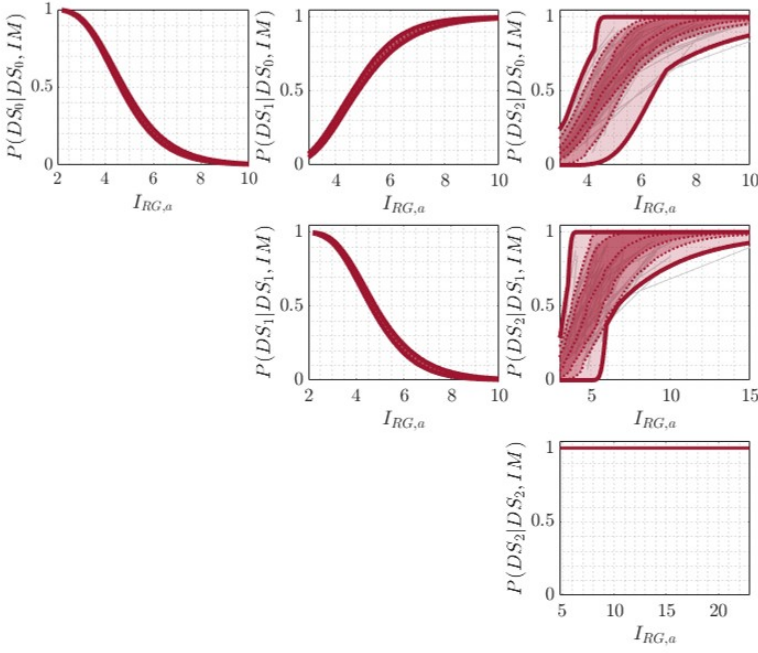


Figure 6.8: Monte Carlo derived state-dependent fragility functions: grey lines represent single simulations; dashed-dark red lines depict the 50% and 90% confidence bounds, whilst continued-dark red ones the 99%. Lighter to darker shaded areas highlighted the corresponding regions of the aforementioned quantiles.

PCE metamodel: fragility functions

According to the procedure outlined in Section 6.2.1, to accurately populate the ED for meta-modelling different initial damage limit state conditions, 25 random simulations for each previous MC transition states were sampled. The size of the ED was determined by iteration on reduced samples dimensions of $10 - 15 - 25 - 50$, to simulate limited and realistic dataset density: the most favourable trade-off between ED size and coefficient of variation CoV of surrogates' response was sought. Then, both for undamaged, i.e. '0', and damaged initial condition, i.e. '1', PCE surrogate models were cast. As a criteria to test accuracy and predictive quality of the meta-models, an estimate of the relative generalization error through the modified leave-one-out error ε_{LOO}^* is performed, since it only requires the points used for training the meta-model. The ε_{LOO}^* is evaluated as

$$\varepsilon_{LOO}^* = \varepsilon_{LOO} \cdot T(P, N) \quad (6.10)$$

Table 6.6: Monte Carlo state-dependent fragility functions: the top four β_{eff} efficiency indices for each transition states.

Frag. '0-0' Optima IM	β_{eff} index	Frag. '0-1' Optima IM	β_{eff} index	Frag. '0-2' Optima IM	β_{eff} index
I _{RG,a}	4.81E-05	I _{RG,a}	1.25E-04	PGD	9.57E-03
PGA	8.11E-05	PGA	1.28E-04	E _{cum}	2.03E-02
Sv(T ₂)	8.35E-05	Sv(T ₂)	1.38E-04	IC	2.06E-02
Sa(T ₂)	8.35E-05	Sa(T ₂)	1.38E-04	PGV	2.28E-02

Frag. '1-1' Optima IM	β_{eff} index	Frag. '1-2' Optima IM	β_{eff} index
I _{RG,a}	2.24E-05	Sd(T ₅)	3.18E-02
PGA	6.68E-05	Sv(T ₅)	3.19E-02
Sv(T ₅)	7.06E-05	ASI	3.41E-02
Sd(T ₅)	7.06E-05	E – ASAR ₄₀	4.76E-02

$$\varepsilon_{LOO} = \frac{\sum_{i=1}^n \left(\mathcal{M}(\mathbf{x}^{(i)}) - \mathcal{M}^{PC \setminus i}(\mathbf{x}^{(i)}) \right)^2}{\sum_{i=1}^n \left(\mathcal{M}(\mathbf{x}^{(i)}) - \hat{\mu}_Y \right)^2} \quad (6.11)$$

where ε^* is the modified error, ε is the original leave-one-out error and $T(P, N)$ is a correction factor of the form

$$T(P, N) = \frac{N}{N - P} \left(1 + \frac{\text{tr}(\mathbf{C}_{emp}^{-1})}{N} \right) \quad (6.12)$$

with P the number of regressor, N the ED size and $\mathbf{C}_{emp} = \frac{1}{N} \mathbf{A}^T \mathbf{A}$ the so-called experimental matrix that contains the values of all the basis polynomials in the ED points, i.e. $\mathbf{A}_{ij} = \psi_j(x^{(i)})$, as clearly described in [34]. Moreover, to handle the dimensionality of the input, PCA is applied to the 1E4 gms generated as described in Section 6.2.2. Figures 6.9-6.10 report the performances of the surrogates for the pristine and non-pristine initial damage conditions. The better the PCE model surrogates the true model, the closer the points are to a 45° line. Thus, a careful reader can appreciate that the calibrated PCE model performs well, only slightly diverging from the 45° line in the low-left extreme for the non-pristine surrogate model.

In addition, to provide a local error estimator, bootstrap technique is applied to PCE. In more detail, resampling with substitution is used on the ED $\mathcal{X}$, thus generating a set $B = 500$ of bootstrap replications with

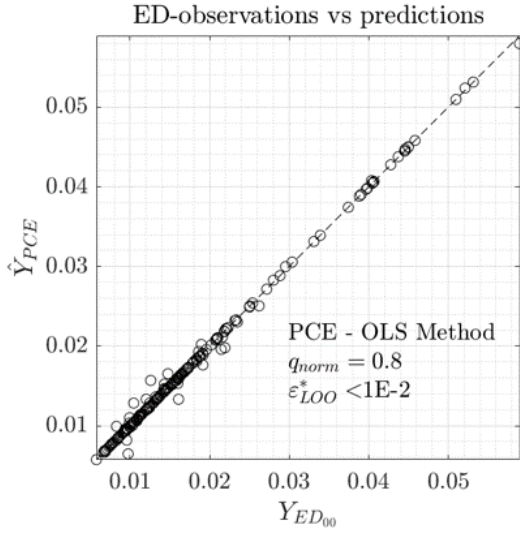


Figure 6.9: Comparison of the output given by the reference model ($Y_{ED_0} = \mathcal{M}(\mathbf{X})$) and the PCE model ($\hat{Y}_{PCE_0} = \mathcal{M}^{PC}(\mathbf{X})$).

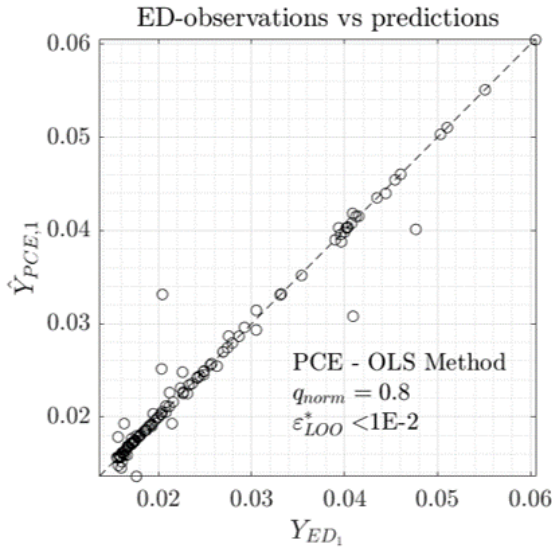


Figure 6.10: Comparison of the output given by the reference model ($Y_{ED_1} = \mathcal{M}(\mathbf{X})$) and the PCE model ($\hat{Y}_{PCE_1} = \mathcal{M}^{PC}(\mathbf{X})$).

the same number of sample points as the original $\mathcal{X}$. Each of the B replications is used to evaluate corresponding PCE, yielding to B different sets of PCE's coefficients and predictions. Those B sets of responses are used to calculate the local variance and quantiles of the PCE predictor,

exploiting all the information of the reduced finite ED size. As a result, the estimation of PCE coefficients converged at polynomial degree 4 and 6 for surrogate the '0' and '1' initial condition, with $\varepsilon_{LOO} < 1\text{E-}2$, respectively. Therefore, Figure 6.11 represent the bootstrapped-PCE derived state-dependent fragility curves: this round, in grey the bootstrapped replications are plotted, while, as before, in dashed-dark-red line the confidence bounds associated with the 50%, 90% and 99% quantiles (the latter continuous-dark-red line) are shown. The more pronounced shading denotes the smaller interval, i.e. the 50%, as earlier mentioned. Anew, the β_{eff} efficiency index is calculated separately for each transition states: Table 6.7 collects results. Once again, as a careful reader can notice, different sets of optimal IMs are obtained as the transition state changes, in good agreement with what observed with MC simulations. In addition, the variability increases moving towards both damaged states and states with initial condition other than '0', as expected.

Table 6.7: Bootstrapped-PCE state-dependent fragility functions: the top four β_{eff} efficiency indices for each transition states.

Frag. '0-0' Optima IM	β_{eff} index	Frag. '0-1' Optima IM	β_{eff} index	Frag. '0-2' Optima IM	β_{eff} index
$I_{\text{RG,a}}$	4.60E-04	$I_{\text{RG,a}}$	1.22E-04	PGD	5.02E-03
PGA	9.13E-04	PGD	1.50E-04	E_{cum}	6.11E-03
$\text{Sd}(T_4)$	9.44E-04	$\text{Sv}(T_2)$	1.88E-04	$\text{Sd}(T_1)$	8.40E-03
$\text{Sv}(T_4)$	9.44E-04	PGA	2.85E-04	PGV	9.13E-03

Frag. '1-1' Optima IM	β_{eff} index	Frag. '1-2' Optima IM	β_{eff} index
$I_{\text{RG,a}}$	2.43E-05	$\text{Sd}(T_5)$	3.64E-02
PGA	4.24E-05	$\text{Sv}(T_5)$	3.77E-02
$\text{Sv}(T_5)$	6.70E-05	$\text{Sd}(T_4)$	4.05E-02
$E - \text{ASA}_{\text{R}40}$	8.64E-05	$E - \text{ASA}_{\text{R}40}$	4.61E-02

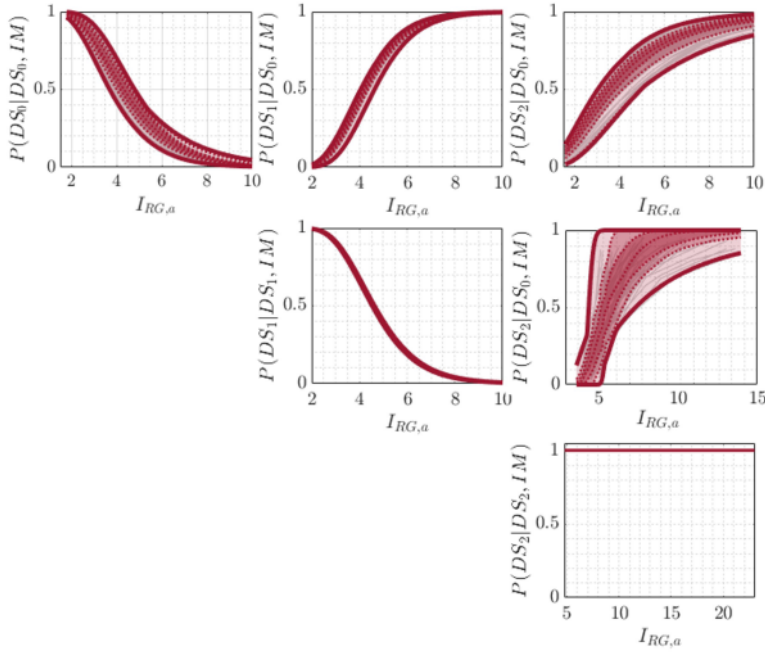


Figure 6.11: Bootstrapped-PCE derived state-dependent fragility functions: grey lines represent single simulations; dashed-dark red lines depict the 50% and 90% confidence bounds, whilst continued-dark red ones the 99%. Lighter to darker shaded areas highlighted the corresponding regions of the aforementioned quantiles.

Validation of the method's outcomes

A first qualitative comparison, with Monte Carlo-derived curves as benchmarks, demonstrates good agreement with the results obtained. This is evident, particularly in the '0-0', '0-1' and '1-1' state transitions; while more scatter is noted in the '0-2' and '1-2' fragilities, due to the fewer simulations populating these states. From a quantitative point of view, this is also confirmed by cross-comparison of the efficiency indices evaluated for each state. Table 6.8 gathers this delta difference $\Delta(\beta_{\text{eff}})$ expressed as $(|\beta_{\text{eff}}^{\text{PCE}} - \beta_{\text{eff}}^{\text{MC}}|) / \beta_{\text{eff}}^{\text{MC}}$ for each of the top indices evaluated separately for every transition states and metrics, i.e. MC and bootstrapped-PCE. The closer the delta is to 0, the better the matching of PCE results with MC. Moreover, an asterisk and italics font denoted the β_{eff} s for which there was a difference in the ranking of optimal IMs between the curves obtained with MC and PCE. As already appreciated from the graphs, $\Delta(\beta_{\text{eff}})$ also reports greater dispersion in the '0-2' and '1-2' transition states.

Table 6.8: $\Delta(\beta_{\text{eff}})$ efficiency indices calculated as $\frac{(|\beta_{\text{eff}}^{\text{PCE}} - \beta_{\text{eff}}^{\text{MC}}|)}{\beta_{\text{eff}}^{\text{MC}}}$ for the same top IMs for each transition states.

Frag. '0-0' Optima IM	$\Delta(\beta_{\text{eff}})$ index	Frag. '0-1' Optima IM	$\Delta(\beta_{\text{eff}})$ index	Frag. '0-2' Optima IM	$\Delta(\beta_{\text{eff}})$ index
$I_{\text{RG,a}}$	0.04	$I_{\text{RG,a}}$	0.02	PGD	0.48
PGA	0.13	PGA	0.17	E_{cum}	0.70
* $S_v(T_2)$	0.32	$S_v(T_2)$	0.36	* IC	0.81
* $S_a(T_2)$	0.22	* $S_a(T_2)$	0.75	PGV	0.60
$S_d(T_4)$	0.53	PGD	0.27	$S_d(T_1)$	0.69
$S_v(T_4)$	0.45				

Frag. '1-1' Optima IM	$\Delta(\beta_{\text{eff}})$ index	Frag. '1-2' Optima IM	$\Delta(\beta_{\text{eff}})$ index
$I_{\text{RG,a}}$	0.09	$S_d(T_5)$	0.15
PGA	0.37	$S_v(T_5)$	0.18
$S_v(T_5)$	0.05	* ASI	0.56
* $S_d(T_5)$	0.66	$E - \text{ASA}_{R_{40}}$	0.03
$E - \text{ASA}_{R_{40}}$	0.53	$S_d(T_4)$	0.46

6.5.2 Industrial NSC state-dependent fragility functions: the SPIF #2 vertical tank

The state-dependent seismic fragility framework outlined in Section 6.2.1 is applied herein to the SPIF #2 vertical tank case study. Furthermore, the goodness of fit of the lognormal distribution hypothesis for the fragility functions is tested by checking it against the distribution inferred from the corresponding empirical curves.

Bootstrapped-PCE metamodel

The data acquired from the NLTHA simulations populate the ED of the bootstrapped-PCE models for both the initial condition '0', i.e. maximum acceleration values below the DBE threshold, and for the initial condition '1', i.e. exceeding of DBE threshold. For completeness, index '2' denotes the exceedance of the ultimate threshold, i.e. the SSE limit state. Figure 6.12(a) shows the control plot of the performance of the surrogate model vs the true samples against which the ED is populated. This shows how the points of the '0'-surrogate model with polynomial degree equal to 5 and $\varepsilon_{\text{LOO}}^* = 7.12\text{E-}02$ are quite close to the 45° line, which represents the ideally perfect match between true and surrogate data. Figure 6.12(b) conversely reports the histogram of the surrogate output variable over the histogram distribution of the original ED. Anew,

a good agreement between the two distributions is appreciated.

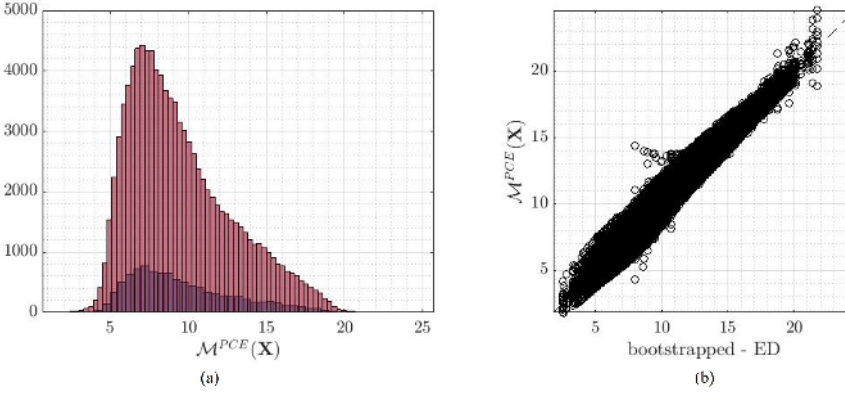


Figure 6.12: (a) Histogram distribution of the surrogate initial condition '0' outputs (red) vs original data of the ED; (b) control plot of the performance of the surrogate-'0' model vs the true ED samples.

Similar observations hold for the '1' surrogate model, defined by a 4^{th} order polynomial, with an ε_{LOO}^* equal to 4.73E-02.

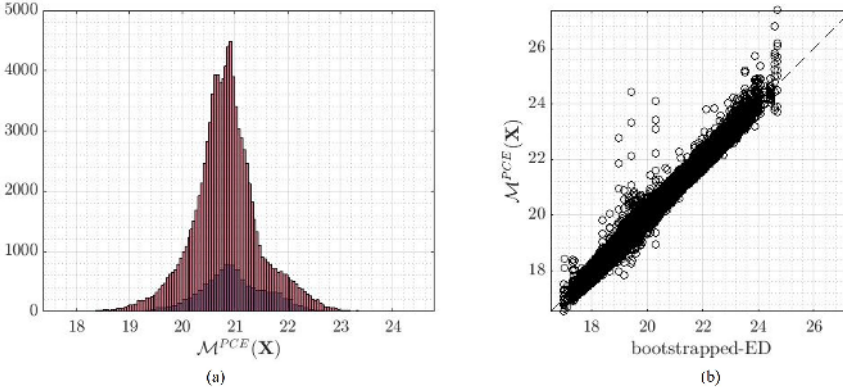


Figure 6.13: (a) Histogram distribution of the surrogate damage condition '1' outputs (red) vs original data of the ED; (b) control plot of the performance of the surrogate-'1' model vs the true ED samples.

To determine which IM is better, the efficiency index β_{eff} is evaluated for each transition bin. Table 6.9 provides the five IMs with the lowest beta for each bin. A careful reader can observe as CAV, PGD, PGV, E_{cum} , $I_{\text{RG},a}$ are the IMs that reoccur, in a different order, among the states of transition: measures, the former, CAV – PGD – PGV, associated with signal velocity and displacement; the latter, E_{cum} - $I_{\text{RG},a}$,

with signal acceleration and energy content. Among the optimal IMs, however, two IMs, which are consistently repeated in all transition states, stand out: $E - ASA_{R_{67}}$ and $E - ASA_{R_{67}}$, namely, the equipment relative average spectral acceleration with a percentage of drop of the fundamental frequency $R = \{67, 80\}$, see [18]. This IM captures the presence of significant spectral acceleration ordinates over the structure's dominant-frequencies interval. Therefore, they are able to account for damage effects and structure-component coupling in terms of frequency range shifts. These observations are in agreement with the experimental evidence in [19], where from the initial processing of the collected data an optimal correlation had been observed between the maximum floor spectral acceleration $Sa_{floor}(T_{NSC})$, evaluated at the identified period $T \simeq 0.16s$ of the vertical tanks, and the $E - ASA_{R_{100}}$.

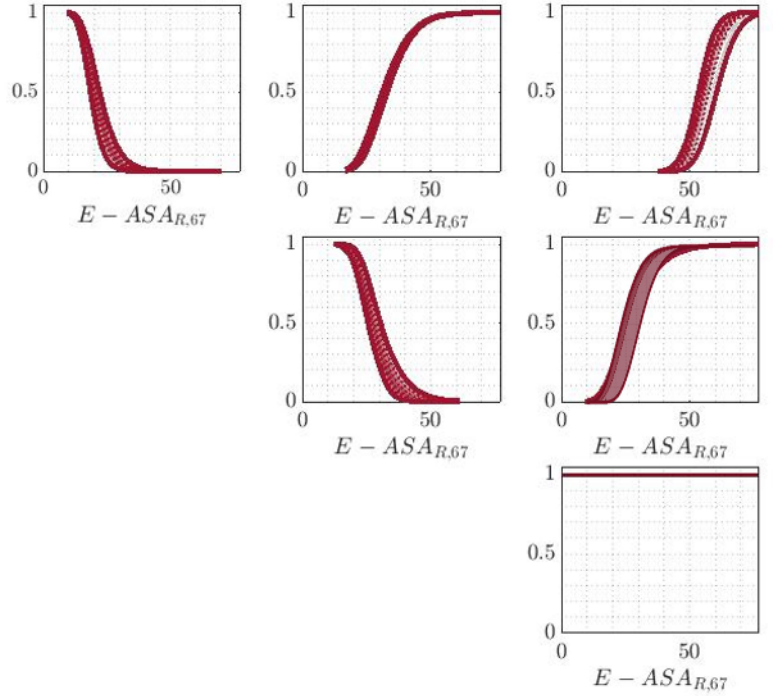


Figure 6.14: Bootstrapped-PCE state-dependent fragility curves of the SPIF #2 vertical tank: dark-red thick lines stand for the 50%, 90% and 99% confidence bound, along with lighter to darker shaded areas.

Thus, fragilities with $E - ASA_{R_{67}}$ are represented in Figure 6.14. Anew, dark-red lines highlight the 50%, 90% and 99% confidence bounds,

together with the associated shaded areas. For the same initial damage state, (graphically for the same row), moving toward higher damage states (to the right) increases the variability associated with the curve. Also, for row '0', moving from state '0' to state '1' or '2' requires greater IM intensities: a result that finds agreement with the curves derived. Instead, for row '1', the already damaged structure requires lower IM intensities to change state: again, this finds agreement graphically.

Table 6.9: Bootstrapped-PCE state-dependent fragility functions for the SPIF #2 NSC: the top five β_{eff} efficiency indices of IMs for each transition states.

Frag. '0-0' Optima IM	β_{eff} index	Frag. '0-1' Optima IM	β_{eff} index	Frag. '0-2' Optima IM	β_{eff} index
CAV	1.55E-04	PGV	4.14E-05	I _{RG,a}	1.71E-04
PGD	1.86E-04	E _{cum}	5.41E-05	E _{cum}	3.32E-04
E _{cum}	3.84E-04	PGD	4.46E-04	E – ASA _{R80}	8.81E-04
I _{RG,a}	6.43E-04	E – ASA _{R67}	2.03E-03	E – ASA _{R67}	9.32E-04
E – ASA _{R67}	8.03E-04	E – ASA _{R80}	2.11E-03	PGV	1.02E-03

Frag. '1-1' Optima IM	β_{eff} index	Frag. '1-2' Optima IM	β_{eff} index
E – ASA _{R40}	4.61E-03	I _{RG,a}	3.38E-05
E – ASA _{R67}	4.98E-03	PGV	1.20E-05
CAV	5.74E-03	PGD	1.71E-04
PGD	5.96E-03	E – ASA _{R67}	3.46E-04
E – ASA _{R80}	9.03E-03	E – ASA _{R40}	5.49E-04

Comparison between the Lognormal and the Empirical distribution

Finally, a check on the classic assumption of log-normal distribution for fragility functions is carried out on the mean curve with E – ASA_{R67} as IM of each transition state above examined.

For completeness, the procedure applied is explained on transition state '1-2' and is meant to be extended in all the others. From the results of the state-dependent fragility curves derived aforementioned, the mean curve and ED dataset of the transition bin of interest are extracted and the empirical data curve is drawn. From the latter, among a broad class of probabilistic models of marginal distributions, the model that best fits the given dataset is looked out, according to the Akaike information criterion (AIC). Then, the parameters of the distribution family are fitted through the MLE method. Figure 6.15 illustrates the results: on the left, the histogram of the ED dataset is plotted against the inferred best PDF selected; whilst, on the right, the CDFs of the inferred and of the log-normal distribution are plotted against the empirical one.

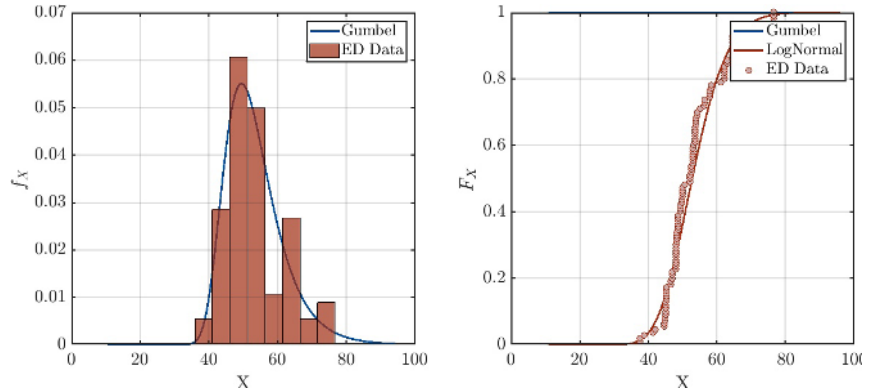


Figure 6.15: Transition state '1-2': on the left, the histogram of the ED dataset and the inferred Gumbel PDF; on the right, log-normal, Gumbel and empirical CDFs.

An interested reader can find the same representation for all the other transition states in Appendix B.; while herein in Table 6.10 are summarized the results of the inference procedure, along with the fitted parameters of the distributions.

Table 6.10: Summary of the log-normal distribution parameters and of the inferred distributions and their fitted parameters for each fragility function of the SPIF#2 NSC.

	LogNormal Distribution			Dist.	Inferred Distribution	
	Dist.	Par #1	Par #2		Par #1	Par #2
Frag. '0-0'	LogNormal	$\mu = 2.136$	$\sigma = 0,701$	Gamma	$1/\lambda = 1/0.23$	$k = 2.426$
Frag. '0-1'	LogNormal	$\mu = 3.880$	$\sigma = 0.192$	Weibull	$p_1 = 52.82$	$p_2 = 6.82$
Frag. '0-2'	LogNormal	$\mu = 3.884$	$\sigma = 0.142$	Gamma	$1/\lambda = 1/1.049$	$k = 51.48$
Frag. '1-1'	LogNormal	$\mu = 3.907$	$\sigma = 0.156$	Gumbel	$p_1 = 46.66$	$p_2 = 6.327$
Frag. '1-2'	LogNormal	$\mu = 3.966$	$\sigma = 0.153$	Gumbel	$p_1 = 49.58$	$p_2 = 6.691$

It is clear that the *a priori* assumption of log-normal distribution is not the strongest; although in general qualitatively quite good agreement might be found with the empirical. Instead, the distributions that better describe the data are the Gamma and Gumbel: the former particularly for cases with an initial undamaged condition; the latter for transition states starting with initial damage conditions. Figure 6.16 collects the comparison between log-normal, inferred and empirical CDFs for the entire transition states.

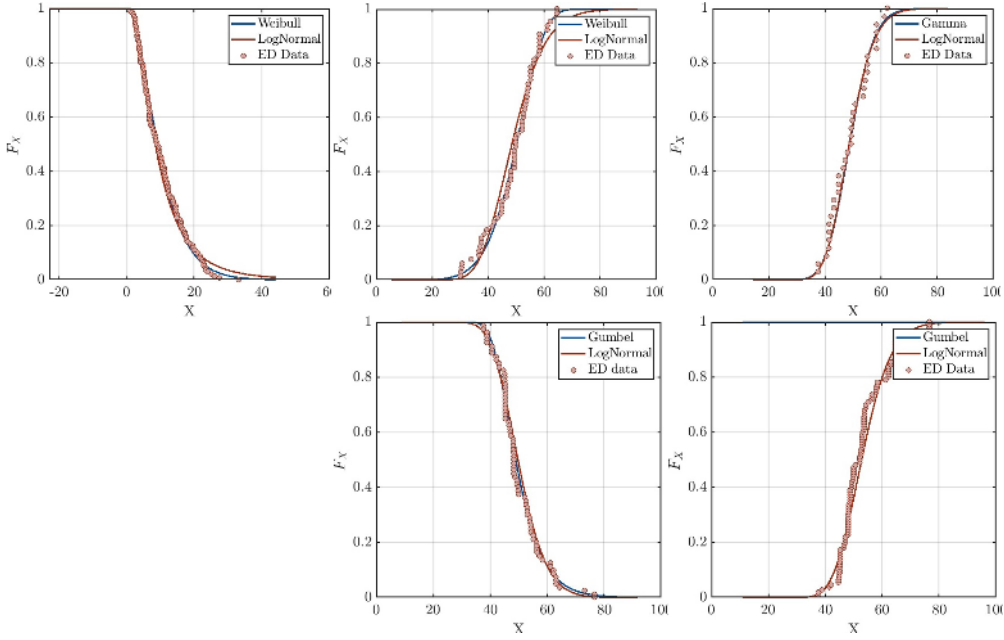


Figure 6.16: Log-normal, inferred and empirical CDFs of the entire set of transition states fragility functions for the NSC of SPIF#2, with E – ASA_{R67} as IM.

6.6 Conclusions and future developments

An innovative methodology for the assessment of state-dependent fragility functions has been proposed. It is based on the combination of advanced surrogate modelling techniques, like bootstrapped-PCE, with data from FE models or real world to yield state-dependent fragilities. These fragility curves are functions of: (i) the selected IM and (ii): the initial damage condition limit state. The proposed method has been applied and validated *in primis* on a simple yet realistic MDoF system endowed with Bouc-Wen hysteresis; then, on a vertical tank of a more realistic industrial substructure archetype, comprehensively tested by means of a shaking table. In particular, in the first case, the inexpensive-to-run model was engaged to compute state-dependent curves through 50000 nonlinear time history analyses by means of the Monte Carlo approach. The outcomes then resembled the ones obtained through the bootstrapped-PCE method. Based on the variance of the estimated fragility curves, an efficiency index β_{eff} was then defined to identify the optimal IM. It was clear, even numerically, how the optimal IM changes with the damage level experienced by the system and with the population size of the original dataset. In the second example, the application to a complex and realistic industrial case study allows for the derivation of state-dependent

fragility curves for a component that proved to be critical during the experimental campaign: the vertical tank mounted on the first floor of the 3-storey braced frame industrial steel structure. Again, through the estimation of the efficiency index, it was possible to identify, among the optimal IMs, the relevant occurrence of the $E - ASA_{R_{67}}$. This IM is of particular interest since it considers the damage experienced by the system through a frequency range shift. Finally, empirical fragility curves were derived by leveraging the large number of outputs generated from the bootstrap and PCE combination. The best probability distribution was then inferred by employing the maximum likelihood estimation method and the Akaike information criterion. Therefore, the classical lognormal distribution assumption for the definition of fragility curves was verified. Despite the good fitting of the lognormal parameters, for the case of an initially undamaged system, a Gamma distribution is preferable; whereas a Gumbel distribution fits better a structure already damaged. Nonetheless, further analysis of other system configurations are needed to enhance and generalise those findings. Finally, based on proper assumptions, the state-depend fragility curves derived from critical industrial substructures could populate Markovian transition matrices very useful for the quantification of restoring functions.

A. Histograms and distributional models for input meta-features.

Table 6.A.1: Histograms and distributional models for IMs.

Index	Name	Type	Parameters	Moments
1	PGA	Gumbel	2.027e+00, 6.236e-01	2.387e+00, 7.998e-01
2	PGV	Gumbel	1.408e-01, 5.683e-02	1.736e-01, 7.289e-02
3	PGD	Gamma	3.853e+01, 3.489e+00	9.055e-02, 4.848e-02
4	Sd(T_1)	Gumbel	1.447e-02, 6.679e-03	1.833e-02, 8.566e-03
5	Sd(T_2)	Gumbel	1.031e-02, 4.263e-03	1.277e-02, 5.468e-03
6	Sd(T_3)	Gumbel	7.792e-03, 2.635e-03	9.313e-03, 3.379e-03
7	Sd(T_4)	LogNormal	-5.551e+00, 3.517e-01	4.133e-03, 1.500e-03
8	Sv(T_5)	LogNormal	-6.831e+00, 3.239e-01	1.137e-03, 3.782e-04
9	Sv(T_1)	Gumbel	1.814e-01, 8.370e-02	2.297e-01, 1.074e-01
10	Sv(T_2)	Gumbel	1.835e-01, 7.591e-02	2.274e-01, 9.736e-02
11	Sv(T_3)	Gumbel	1.958e-01, 6.622e-02	2.341e-01, 8.493e-02
12	Sv(T_4)	LogNormal	-1.815e+00, 3.517e-01	1.732e-01, 6.285e-02
13	Sv(T_5)	LogNormal	-2.690e+00, 3.239e-01	7.156e-02, 2.380e-02
14	Sa(T_1)	Gumbel	2.273e+00, 1.049e+00	2.878e+00, 1.345e+00
15	Sa(T_2)	Gumbel	3.268e+00, 1.352e+00	4.048e+00, 1.733e+00
16	Sa(T_3)	Gumbel	4.922e+00, 1.664e+00	5.883e+00, 2.134e+00
17	Sa(T_4)	LogNormal	1.920e+00, 3.517e-01	7.260e+00, 2.634e+00
18	Sa(T_5)	LogNormal	1.452e+00, 3.239e-01	4.502e+00, 1.497e+00
19	I _A	LogNormal	-6.729e-01, 5.712e-01	6.006e-01, 3.730e-01
20	E _{cum}	LogNormal	1.342e+00, 5.453e-01	4.439e+00, 2.612e+00
21	I _{RG,a}	Gumbel	4.151e+00, 1.222e+00	4.856e+00, 1.567e+00
22	I _{RG,v}	LogNormal	-5.098e-01, 2.745e-01	6.237e-01, 1.745e-01
23	T _d	Gaussian	8.696e+00, 1.637e+00	8.696e+00, 1.637e+00
24	RMS($\ddot{u}(t)$)	LogNormal	-7.590e-01, 2.845e-01	4.875e-01, 1.415e-01
25	IC	LogNormal	-6.627e-02, 4.070e-01	1.017e+00, 4.315e-01
26	CAV	Gamma	9.262e-03, 9.638e+00	1.041e+03, 3.352e+02
27	I _{CM}	LogNormal	2.178e+00, 4.817e-01	9.915e+00, 5.067e+00
28	ASA ₄₀	Gamma	1.122e-01, 8.580e+00	7.647e+01, 2.611e+01
29	ASI	LogNormal	6.025e+00, 2.910e-01	4.315e+02, 1.283e+02
30	EPA	Gamma	4.737e-02, 1.134e+01	2.393e+02, 7.108e+01
31	I _{v/a}	Gamma	1.343e+02, 9.941e+00	7.400e-02, 2.347e-02
32	I _F	Gumbel	2.407e-01, 9.524e-02	2.957e-01, 1.221e-01
33	F _m	Gaussian	4.649e+00, 4.801e-01	4.649e+00, 4.801e-01
34	$\dot{F}_m$	GumbelMin	-2.667e-04, 2.347e-04	-4.022e-04, 3.011e-04
35	FAS _{area}	Exponential	3362	2.974e-01, 2.974e-01
36	E – ASA _{R40}	LogNormal	2.371e+00, 3.504e-01	1.138e+01, 4.115e+00
37	E – ASA _{R67}	LogNormal	3.283e+00, 3.387e-01	2.823e+01, 9.842e+00
38	E – ASA _{R80}	LogNormal	3.561e+00, 3.325e-01	3.719e+01, 1.271e+01
39	E – ASA _{R100}	LogNormal	3.917e+00, 3.229e-01	5.296e+01, 1.756e+01
40	E – ASA _{R150}	LogNormal	4.495e+00, 3.110e-01	9.398e+01, 2.995e+01
41	E – ASA _{R200}	LogNormal	4.879e+00, 3.056e-01	1.378e+02, 4.313e+01

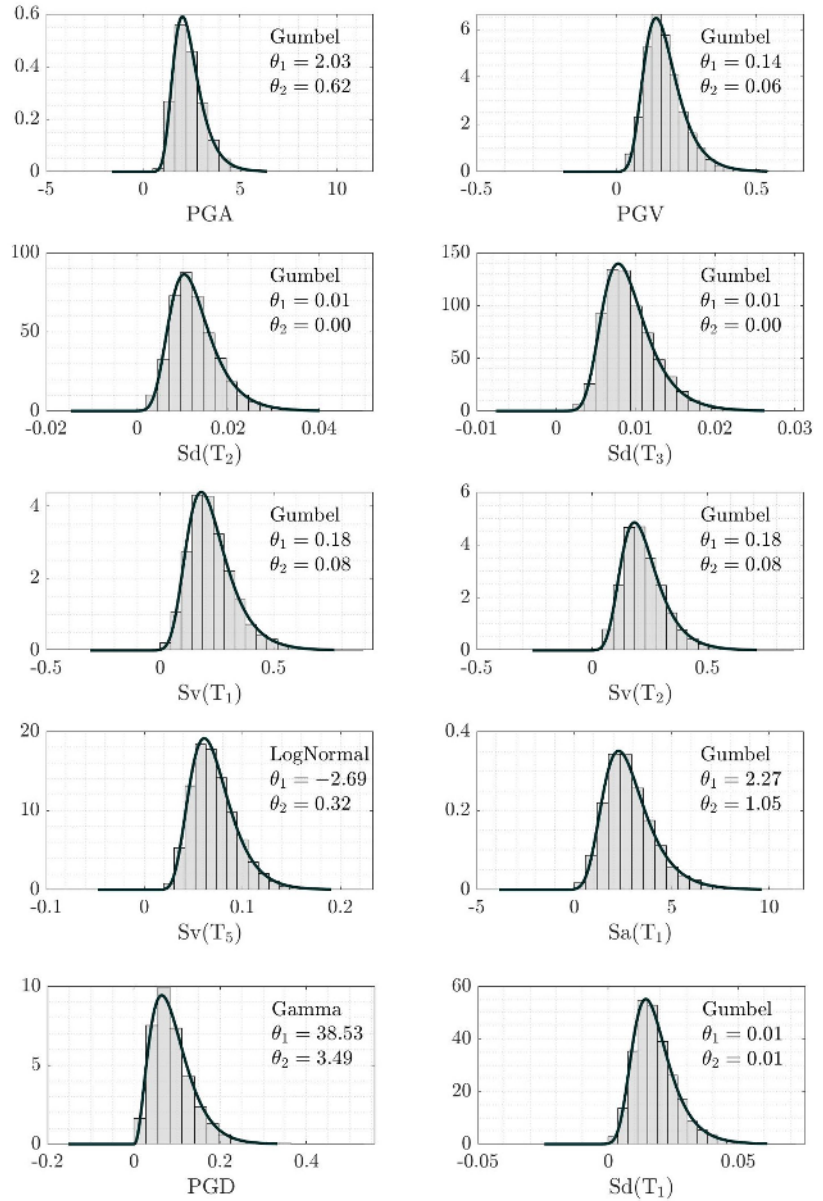


Figure 6.A1: Histograms and distributional models for IMs from 1 to 10.

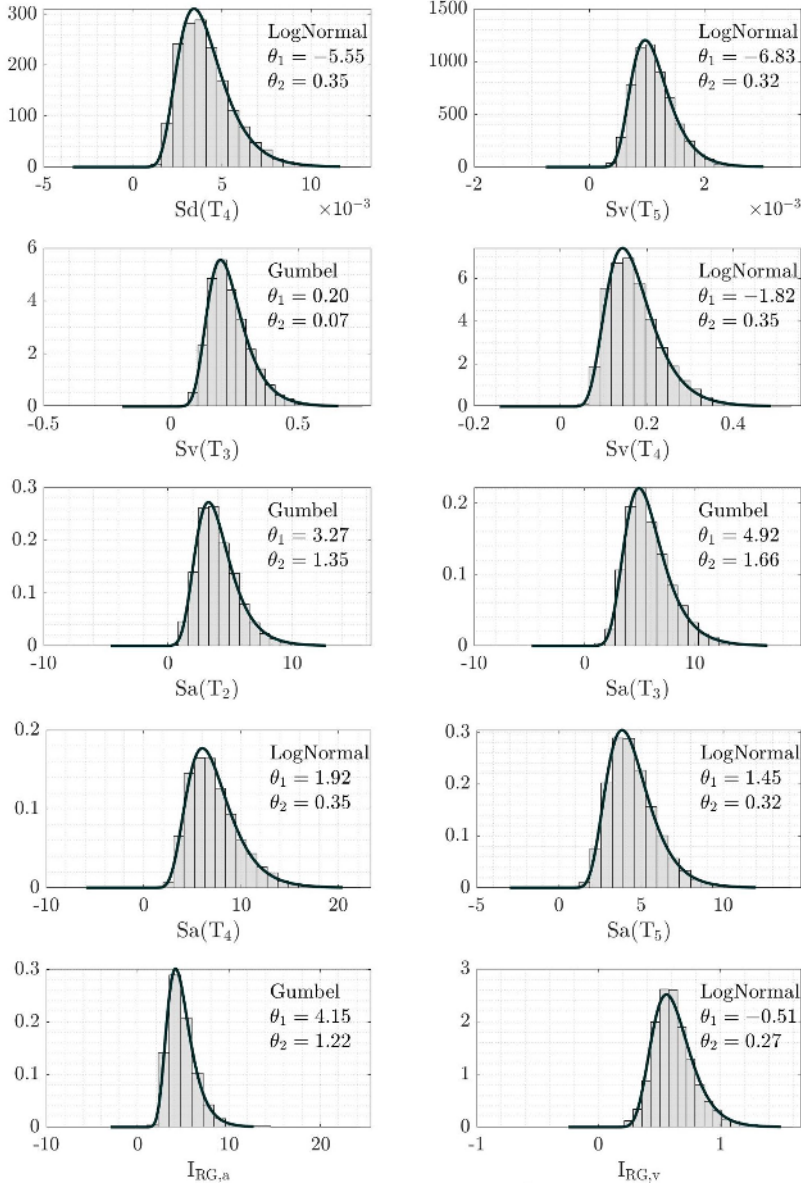


Figure 6.A2: Histograms and distributional models for IMs from 11 to 20.

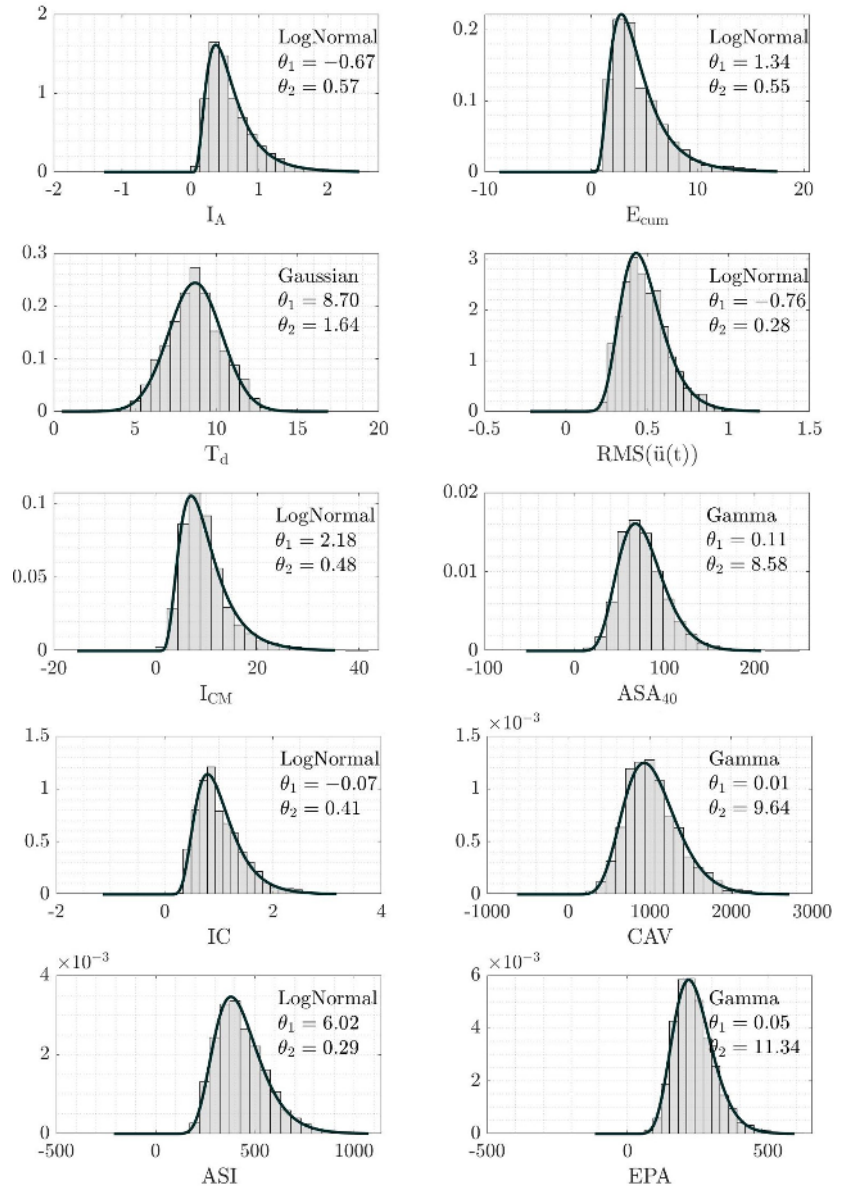


Figure 6.A3: Histograms and distributional models for IMs from 21 to 30.

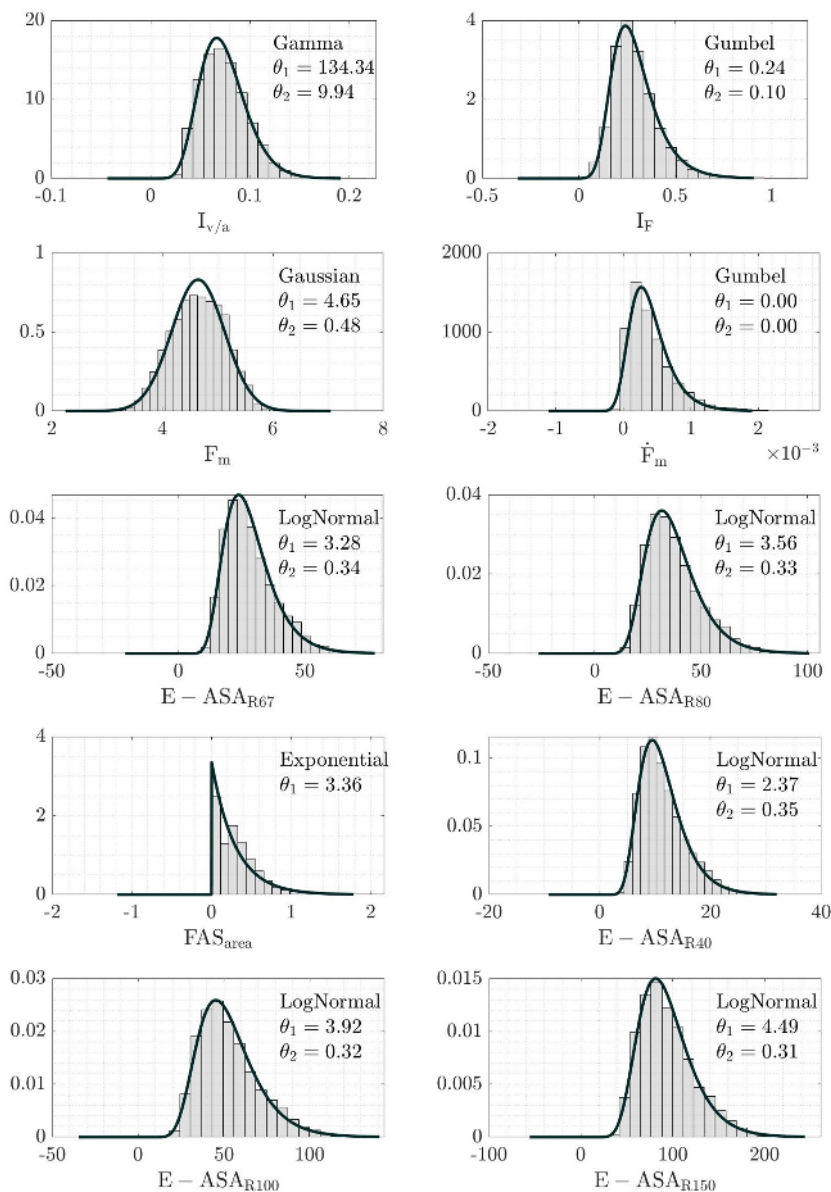


Figure 6.A4: Histograms and distributional models for IMs from 31 to 40.

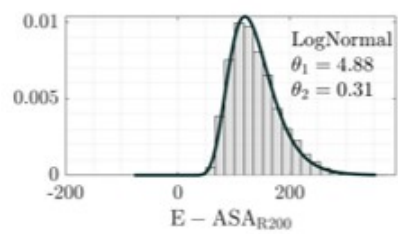


Figure 6.A5: Histograms and distributional models for IM 41.

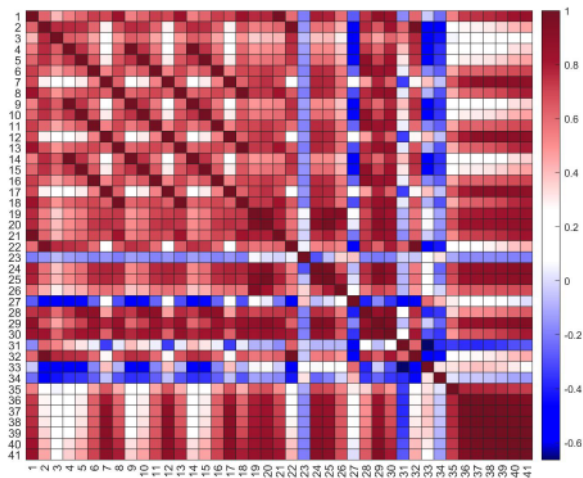


Figure 6.A6: Correlation map between input IM parameters.

B. - Inferred distribution of the state-dependent fragility functions for the NSC of SPIF#2

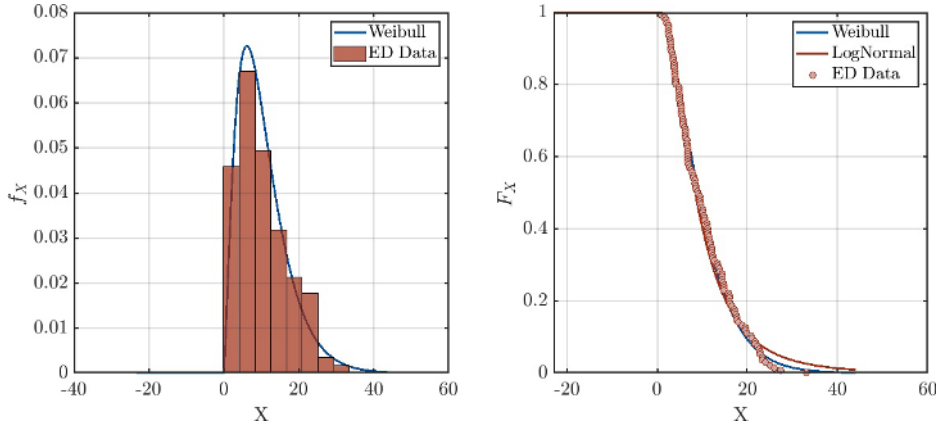


Figure 6.B1: Transition state '0-0': on the left, the histogram of the ED dataset and the inferred Gamma PDF; on the right, log-normal, Gumbel and empirical CDFs.

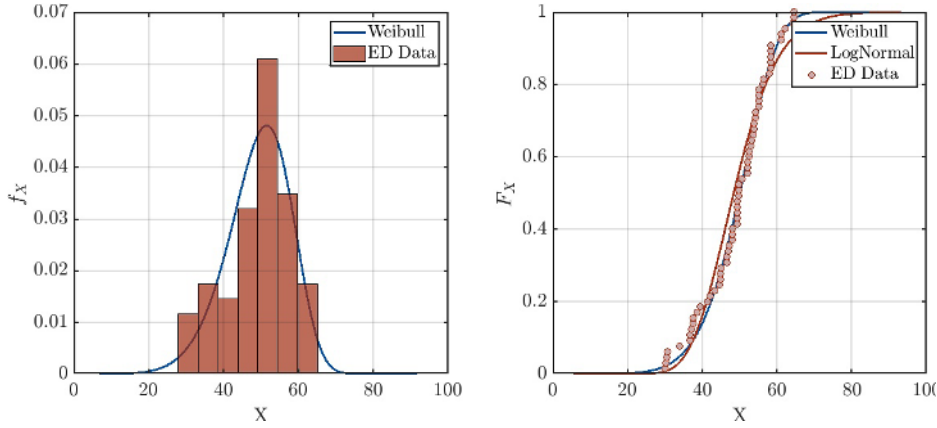


Figure 6.B2: Transition state '0-1': on the left, the histogram of the ED dataset and the inferred Weibull PDF; on the right, log-normal, Gumbel and empirical CDFs.

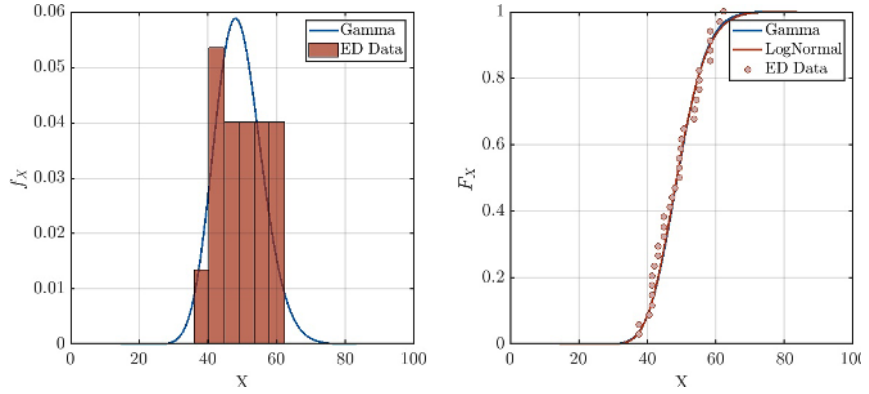


Figure 6.B3: Transition state '0-2': on the left, the histogram of the ED dataset and the inferred Gamma PDF; on the right, log-normal, Gumbel and empirical CDFs.

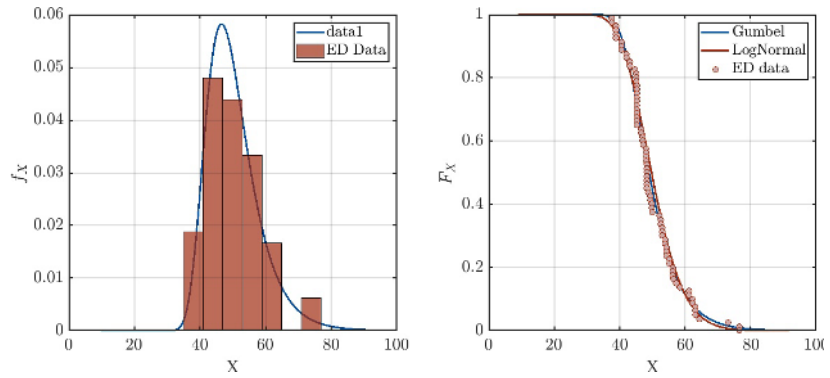


Figure 6.B4: Transition state '1-1': on the left, the histogram of the ED dataset and the inferred Gumbel PDF; on the right, log-normal, Gumbel and empirical CDFs.

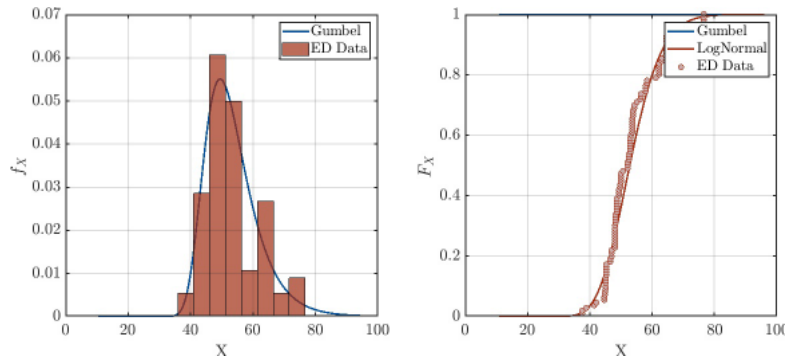


Figure 6.B5: Transition state '1-2': on the left, the histogram of the ED dataset and the inferred Gumbel PDF; on the right, log-normal, Gumbel and empirical CDFs.

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7. Conclusions

7.1 Summary and discussion of research results

Earthquakes are among the most devastating natural phenomena, impacting loss of life, damage to buildings, and disruption of economic activities. The last decade, with Tohoku 2011, Emilia 2012, and New Zealand 2016 earthquakes, emphasised that non-structural components and process equipment have been significant drivers for overall physical damage and business interruption. Particularly in process industries, this may lead, in addition to business disruption, to the possibility of loss of containment and release of hazardous material within the area. The consequences undoubtedly are catastrophic. Along this line, recently, research in this area received interest from the community and gained momentum. Among those research projects, the year-long study carried out by the National Institute of Standards and Technology (NIST) and the European INDUSE2SAFETY experimental campaigns on process industry equipment are worth mentioning. In this respect, the former was keen to collect and document the body of available knowledge related to the seismic performance of NSCs; whilst the latter was devoted to testing the seismic performances of critical components. Nevertheless, from these studies emerged: (i): the need of performing fully comprehensive testing campaigns and investigations on coupling effects between main structures and NSCs; and (ii): a lack of definition of performance objectives for NSCs in a performance-based design methodologies perspective. The European project SERA SPIF - *Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities* - was conceived for this scope. More precisely, the project's main objective was to investigate the seismic behaviour of a representative industrial multi-storey moment-resistant frame (MRF) structure equipped with complex process components by means of shaking table tests from a PBEE perspective. The testing campaign highlighted the dynamic interaction between the primary steel structure and secondary process units that influenced the whole system's performance. The experimental campaign also allowed to testing innovative structural health monitoring systems based on fibre optics. More precisely, a fiber Bragg grating system was installed on critical bolted-flange joints (BFJs) to track and monitor leakage phenomena. Testing results demonstrated a dynamic interaction between the primary MRF steel structure and secondary process units that influenced the whole system's performance. In particular, the primary steel structure remained undamaged for safe shutdown earthquake

levels mainly due to its design and inherent flexibility. However, due to the adoption of common practice solutions for NSC components, some damage appeared in web plates of fin plate connections between main members and secondary beams. In fact, due to the intense rocking of the vertical tanks mounted at the first level, the fin plate connection under the supporting beam systems for the NSCs experienced warping of the crossbeam's web and cracks in the transition zone from beam to web. Nonetheless, the experimental campaign left some open issues: (i) the verification of the efficiency of other recurring structural configurations; (ii) the selection of a seismic input capable of emphasising the coupling between primary structure and secondary elements. Therefore, to respond to these requirements, the SPIF #2 project was conceived. The existing SPIF MRF mock-up was converted into a braced frame (BF) configuration, by installing a dedicated bracing system. The mock-up was tested by means of a shaking table with several earthquake levels within a PBEE perspective. Thus, limit states and performance objectives for a primary BF structure and NSCs were proposed and identified. Besides, to excite the vibration periods of the most relevant and critical NSCs, i.e. the heaviest vertical tanks, a synthetic record derived by a site-based ground motion model was selected. As a result, the complex interaction between the primary BF structure and the NSCs was enhanced; and the experimental engineering demand parameters values were compared with reference thresholds. The SPIF #2 testing campaign was able: (i) to demonstrate the favourable performance of the BF configuration under strong motions; (ii) to experimentally verify the effectiveness of the proposed PBEE limit states and thresholds; (iii) to provide data about the dynamic interaction between the primary steel BF and the NSCs; (iv) at the very least, to interpret the causes of these interactions. In this respect, to better understand the complex dynamics of the system, FE models of the SPIF mock-ups were deemed necessary. The rationale behind developing these models was an optimal balance between computational demand time and reliability in assessing SPIF's experimental campaigns. To do so, global models for the MRF and BF configurations, calibrated on local parametric studies of the relevant components, were implemented in the SAP2000 software. Hence, it was possible to identify the tuned mass damper effect performed by vertical tanks in BF configuration tests and to validate numerically the leakage models developed for BFJs. Besides, particular attention was paid to model calibration, leveraging UQ model updating techniques, like Monte Carlo Markov Chain in Metropolis-Hastings algorithm.

In order to perform seismic fragility assessment of critical industries process equipment, like the vertical tanks of SPIF #2 mock-up, an inno-

vative methodology for developing state-dependent fragility functions is presented. The complexity of the problem and its inherent modelling, coupled with a general scarcity of available data and limited computational capability, indeed, concurred in limiting or developing risk assessment methods based on meaningful reduced and quick-to-run models, by losing awareness of the complexity of the entire phenomena. Therefore, the proposed method combines limited data, both experimental and from refined FE models, with the power of well-known metamodeling techniques, i.e. the polynomial chaos expansion technique (PCE), boosted with the bootstrap method. The method is first validated on a simplified and inexpensive-to-run MDoF system endowed with Bouc-Wen hysteresis. Then, it is applied to the SPIF #2 vertical tanks. Therefore, empirical fragility curves were derived based on the large number of outputs generated from the bootstrap and PCE combination. The best probability distribution was then inferred by employing the maximum likelihood estimation method and the Akaike criterion both for the undamaged and damaged structure; and the classical lognormal distribution assumption for the definition of fragility curves was verified.

7.2 Future perspectives

Both the MRF and the BF frames tested on the shaking table demonstrated favourable seismic performance and significant NSCs-structure coupling interactions. However, further investigations on how different configurations and arrangements of components within the structures may affect the overall response is of definite interest. This can be accomplished both experimentally and numerically. Hybrid simulations that rely on reduced numerical substructures might be of interest to overcome the expensive cost of a full-scale experimental campaign. In turn, it would be possible to extend and enrich a database of observations and experimental data on the behaviour and interactions of NSCs with the main structure.

To enrich and collect data represent a fundamental key-step for properly populating the experimental design (ED) datasets used in metamodeling. Important sampling techniques might be implemented to overcome this issue and compensate for the lack of sufficient data for some ED dataset. Furthermore, numerical results relevant to both the simplified MDoF system model endowed with Bouc-Wen hysteresis and the realistic and more complex case of the industrial plant substructure endowed with the vertical tank reveal how the optimal intensity measure (IM) changes with the damage level experienced by the system. Thus, 3D state-dependent fragility surfaces instead of fragility functions might

be accurately defined. Finally, the above mentioned perspectives concerning the quantification of state-dependent fragility curves representative of critical process equipment of industrial substructures, naturally lead to draft Markovian transition matrices useful for the seismic risk quantification and assessment. Relevant relationships and considerations on Markovian processes suitable for the analysed structural systems can be found in Appendix A. They pave the way to reframe the seismic vulnerability assessment based on fragility functions in a new and formal fashion.

A. Some considerations on Markovian Processes.

Summary

In this Appendix, we recall a few notes on the Markov chains mentioned in the Section *Future perspectives* 7.2 in relation to state-dependent fragility curves.

A.1 Introduction

In probability theory, a Markov chain, named after the Russian mathematician Andrey Markov, is a stochastic process by which there is no difference between future predictions realized based only on its present state or based on knowing the entire process's history. Actually, there is one difference: the amount of required information needed. In a Markov process, future predictions are based on the actual state only, hence the independence from the whole history of the process. This is also known as the principle of *memorylessness*.

Essentially, the next state of the process only depends on the previous state and not on the entire sequence of states that carried the system to the latest step. This simple assumption makes the calculation of conditional probability incredibly easy and enables this algorithm to be applied in a number of scenarios.

By Markov Chains equation it is possible to explain most complicated real time processes, like speech recognition, text identification, path recognition etc. This process can be also applied to the business or financial environment, in order to make predictions about the next market evolution, along with application in vulnerability and risk assessment for civil infrastructures.

A.2 Theoretical Background

For sake of clarity, the notation and theoretical background formulation is adapted from *Reliability and Availability Engineering: Modeling, Analysis, and Applications*, [1], and *Theory and Problems of Statistics*, [2]. Let $Z(t)$ be a stochastic process defined over a discrete state space Σ of cardinality n . $Z(t)$ is a continuous-time Markov chain (CTMC) if, given any ordered sequence of time instants $(0 < t_1 < t_2 < \dots < t_m)$, the probability of being in state s_{j_m} at time t_m depends only on the state $s_{j_{m-1}}$ occupied by the CTMC at the previous instant of time t_{m-1} and not on the complete sequence of state occupancies of the system at times

t_i , ($0 < i < m - 1$). Formally:

$$\begin{aligned} P\{Z(t_m) = s_{j_m} | Z(t_m - 1) = s_{j_{m-1}}, \dots, Z(t_1) = s_{j_1}\} &= \\ &= P\{Z(t_m) = s_{j_m} | Z(t_m - 1) = s_{j_{m-1}}\} \end{aligned} \quad (\text{A.1})$$

Let us introduce the following notation:

$$p_{ij}(u, t) = P\{Z(t) = j | Z(u) = i\}(u \leq t) \quad (\text{A.2})$$

where $p_{ij}(u, t)$ represents the conditional probability that the Markov chain $Z(t)$ is in state j at time t , given that it was in state i at time u ; it is called the *transition probability* from state i to j . For the transition probabilities $p_{ij}(u, t)$, the following initial conditions hold:

$$p_{ii}(t, t) = 1 \quad p_{ij}(t, t) = 0 (i \neq j) \quad (\text{A.3})$$

Further, let $\pi(t)$ be the (unconditional) probability that the process $Z(t)$ is in state i at time t . $\pi(t)$ is the *state occupancy probability*, or simply the *state probability*, and is defined as:

$$\pi(t) = PZ(t) = i \quad (\text{A.4})$$

From the previous definitions, the following normalization conditions must hold:

$$\sum_{j=1}^n p_{ij}(u, t) = 1, \quad \sum_{i=1}^n \pi(t) = 1 \quad (\text{A.5})$$

By introducing matrix notation, let the matrix $\mathbf{P}(u, u+t) \stackrel{\text{def}}{=} (p_{ij}(u, u+t))_{i,j \in \Omega}$, whose entries are the transition probabilities $p_{i,j}(u, u+t)$, be defined as the *transition probability matrix* of the CTMC and $\Omega = [x_0, \dots, x_i, \dots, x_I]$ denoting the possible states of the Markov chain.

If the process is homogeneous in time the transition probability depends only on the time lag, therefore we can set $u = 0$ and write $\mathbf{P}(t)$. For a homogeneous CTMC the following Chapman-Kolmogorov (CK) equations hold

$$\pi(t) = \pi(0) \cdot \mathbf{P}(t), \quad (\text{A.6})$$

$$\mathbf{P}(t + \Delta t) = \mathbf{P}(t) \mathbf{P}(\Delta t), \quad (\text{A.7})$$

where $\pi(0) = \pi_0$ is the initial state probability vector and $\mathbf{P}(0) = \mathbf{I}$ the identity matrix. Moreover, by introducing the infinitesimal generator of the CTMC

$$\mathbf{Q} = \lim_{\Delta t \rightarrow 0} \frac{\mathbf{P}(\Delta t) - \mathbf{I}}{\Delta t}, \quad (\text{A.8})$$

one can derive the first Kolmogorov forward differential equations

$$\dot{P}(t) = P(t)Q, \text{ with } P(0) = I. \quad (\text{A.9})$$

Each off-diagonal element contains the transition rate from the current state to a future state. Furthermore, by construction, each diagonal term contains the negative of the sum of the off-diagonal terms.

Differentiating Eq. (A.6) and using Eq. (A.9) one can obtain the second Kolmogorov forward differential equation,

$$\dot{\pi}(t) = \pi(t)Q, \text{ with } \pi(0) = \pi_0. \quad (\text{A.10})$$

Eq.(A.10) is a classical ordinary differential equation initial value problem. The coefficients of the system of equations are the the rates of transition. The solution is the state occupancy probability at any given point in time. The general solution of such problem is given by

$$\pi(t) = \pi(0) \exp(Qt). \quad (\text{A.11})$$

A practical implementation can, however, use the following pattern. Given that t_M is the final mission time of the transient analysis, we need to compute $\pi(t)$ in the interval $(0, t_M]$. We take a small time interval $h = t_M/n$ and we exploit the property

$$\pi(t+h) = \pi(t)P(h) = \pi(t)e^{Qh} \quad (\text{A.12})$$

Then, we adopt the following iterative procedure:

$$\begin{aligned} \pi(h) &= \pi(0)e^{Qh}, \\ \pi(2h) &= \pi(0)e^{Q2h} = \pi(h)e^{Qh}, \\ &\vdots \\ \pi(ih) &= \pi((i-1)h)e^{Qh}, \\ &\vdots \end{aligned}$$

To avoid matrix exponentiation in the implementation, the following iterative scheme can be used:

$$\frac{1}{i!} (Qt)^i = \frac{(Qt)}{i} \frac{1}{(i-1)!} (Qt)^{i-1}$$

The series expansion can be truncated when the absolute difference between all the homologous entries in two successive terms in the series

are less than a given ε . Once e^{Qt} has been computed with the required accuracy, the successive time iterations require only matrix–vector multiplication.

Moreover, since the transition from state i to some state $j \in \Omega$ in the interval $(t, t + \Delta t]$ is a certain event:

$$1 = \sum_j p_{ij}(\Delta t) = 1 + q_{ii}\Delta t + \sum_{j:j \neq i} q_{ij}(\Delta t) \quad (\text{A.13})$$

Thus,

$$q_{ii} = - \sum_{j:j \neq i} q_{ij} \quad (\text{A.14})$$

and $Q = [q_{ij}]$ where for $q_{ij} \geq 0, i \neq j$, while for $q_{ii} \leq 0, q_{ii} = - \sum_{j:j \neq i} q_{ij}$.

A.3 State-dependent fragility functions

On these premises, we derive a physical interpretation of transition rates/matrices in terms of state-dependent fragility functions.

Assuming $\Omega = [x_0, \dots, x_i, \dots, x_C]$ denotes the possible states for a structural system, i.e. the damage limit states defined by standards as near collapse, life safety, immediate occupancy and so on, it follows that $x(t)$ represent the damage or functionality state of the system at a given time t .

Figure A1 depicts schematically states and possible transitions among each other. In particular, '0' denotes the undamaged initial condition, while '1' and '2' indicate the progressive level of damage accumulation until 'C' which stands for the ultimate state or collapse.

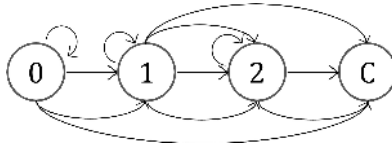


Figure A1: Markov chain for structural damage transitions: from 0, undamaged, to C, collapsed.

The above Markov chain describes the scenarios illustrated in Figure A2. Given a Poissonian occurrence model for seismic events, the structure might experience different state transitions according to the initial state condition and the intensity measure of the excitation.

Therefore, the transition matrix defined in Equation A.7 corresponds to the state-dependent fragility functions evaluated in Chapter 6. Each

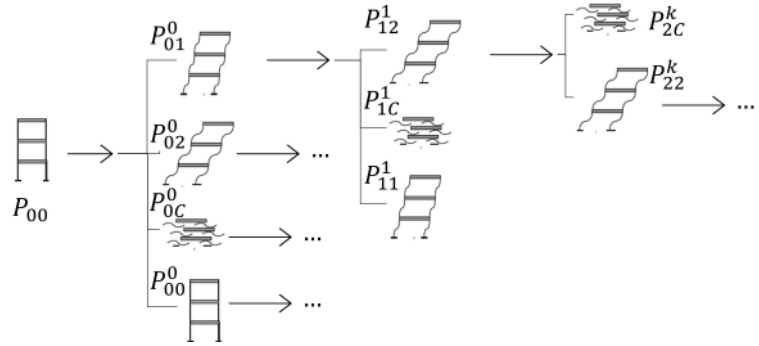


Figure A2: Damage accumulation due to sequence of seismic events, as a Markovian process, described in [3].

terms $p_{ij}(s, s+t)$ is the conditional probability of passing from the state x_i at time s to the state x_j at time $s+t$, given the initial state x_i .

The last relationship paves the way to reframe the seismic vulnerability assessment based on fragility functions in a new and formal manner.

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Nowadays, earthquakes are among the most destructive of natural catastrophes perils. Based on observations of damage following recent disastrous seismic events and numerical studies, it emerges that critical non-structural components (NSCs), ubiquitous to most industrial facilities, are particularly and even disproportionately vulnerable to earthquakes. Nonetheless, seismic provisions for industrial facilities and their process equipment are still based on classical load-and-resistance factor design; a performance-based earthquake engineering (PBEE) approach should, instead, be preferred. However, recent research within a PBEE perspective remarked: (i) a lack of definition of performance objectives for NSCs; (ii) a gap in fully comprehensive testing campaigns on coupled NSC-structure systems. In this respect, this PhD thesis introduces a computational framework for an efficient and accurate seismic fragility analysis, based on a combination of data from experimental test campaigns and refined FE models with cutting-edge surrogate-based uncertainty quantification advancements. Specifically, the framework is applied to a real-world application consisting of seismic shake table tests of a representative industrial multi-storey frame structure equipped with complex process components, within the European SPIF project. The experimental research aspires to improve the understanding of these complex systems and to improve the knowledge of FE modelling techniques, by reducing the high-demanding computational burden and assessing, as well, when the importance of coupling effects comes into play. To assess the seismic risk of NSCs, this thesis also introduces state-dependent fragility curves. To this end, the computational burden is tackled by polynomial chaos expansion (PCE) surrogate models. The dimensionality of the input is reduced by performing the principal component analysis. Then, by bootstrapping on the experimental design, separate PCE coefficients are determined, yielding to full response sample at each point. Finally, empirical state-dependent fragility curves are derived.

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