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Probabilistic seismic fragility assessment of LNG pipe rack – piping system accounting for soil-structure interaction

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Abstract. In petrochemical and midstream facilities, an extensive amount of hazardous and flammable materials is transformed from one unit to another via piping systems. The seismic response of pipework that is usually supported on racks is considered as critical mainly due to mechanical and geometrical idiosyncrasies of pipes and is strongly related with the behaviour of supporting structure. It is essential for the seismic design criteria of piping systems to account for a number of uncertainties towards achieving an up-to-standard degree of seismic safety. To date, the research has focused on component level without considering coupling effects in the design or assessment of existing piping systems. Considering the high irregularities existing in rack-piping system, the selection of seismic input becomes more crucial for the correct prediction of seismic response. Finally, the soil deformability is probably one of the most important challenges the engineer should deal with when designing or assessing structures that belong to mid- or down-stream facilities. Alluvial deposits usually underlie oil refineries, and thus due consideration should be given to Soil-Nonbuilding Structures Interaction. In the present work, an attempt is made to incorporate the aforementioned parameters into a numerical model through a case study that pertains to a Liquified Natural Gas terminal concrete rack that carries piping of different diameter. Assuming that the seismic Intensity Measure follows the lognormal distribution, dynamic coupling effects and Soil-Structure Interaction are proved to detrimentally affect the seismic response of structural components. The nonstructural ones are less influenced by the soil deformability and the results are comparable among Limit States when the dynamic interaction is accounted for.

Keywords: pipe rack, dynamic coupling, soil-structure interaction, seismic input, fragility curves.

1.INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Mid- and down-stream facilities e.g. Liquified Natural Gas (LNG) terminals and oil refineries play an important part in the process and distribution of gas and oil products at national or international level. The connection between a process plant and the prosperity of nearby and not only communities is highly reciprocal since a Loss of Containment Event (LOC) due to a failure to nonstructural component or nonbuilding structure may cause loss of human lives and catastrophic repercussions to environment and financial state. Nowadays, the resilience of industrial facilities may not be adequate to resist natural hazards e.g. inundation or seismic events due to uncertainties involved in the design or assessment of lifeline facilities. Therefore, Europe and other societies have prioritised the integrity of critical plants in order to remain resilient after severe natural hazards. The seismic hazard that the present study intends to examine is one of the most considerable ones in European countries e.g. Italy, Greece and Portugal considering that 28 LNG terminals and several oil refineries are located along European coastline some of which are planned or being expanded and the number is going to increase in the future particularly in the Mediterranean Sea due to the rise of LNG imports (almost 1% annual increase till 2040, EN., [2018]) as well as future exploitation of natural resources in the region.

The position of midstream or liquefaction plants is a strategic choice that tries mainly to minimize the cost of transportation via pipelines. The earthquake hazard potentiality in the East and Central Mediterranean Sea is rather high and that makes a robust and well-completed seismic risk assessment of these plants a strong necessity. This type of assessment is a key one to be addressed for industrial facilities as shown by Antonioni et al., [2007]. A literature review that was conducted at the time showed that 14 seismic events caused damage to 182 equipment items and the 70% of that damage led to Loss of Containment (LOC) events. Oil/gas industrial plants constitute complex systems of structural and nonstructural components (Figure 1) being dynamically interacted during earthquake events. Hundrends or thousands of meters of industrial piping outifits nonbuilding structutes within an oil refinery in order to transfer hazardous and/or toxic materials from one unit to another. To minimise the computational cost, many times the analysis focuses on components that have been found to be the most critical during earthquake events of the past. In particular, it has been found that nonstructural components supported on racks e.g. pipelines are the most vulnerable against seismic hazard due to the incompatibility of displacement of adjacent pipe supports within the rack [Paolacci et al., 2012]. This statement is another evidence that due consideration should be given to the dynamic interaction (coupling) between the rack and piping system and the decision whether it will be neglected or not should rely on boundary conditions except for the weight ratio that the American codes provisions e.g. ASCE/SEI 7-16., [2017] mainly propose [Di Sarno and Karagiannakis., 2019].

A piping system comprises several straight pipes and fittings e.g. elbows, T-joints, flanges, reducers and others. As shown in Figure 1, pipelines present high geometrical irregularities and could be quite vulnerable due to errors in the design or assessment process. To comprehend better the difference between nonstructural components on building and nonbuilding structures included in industrial facilities, it should be imagined that a failure in both environments may put human lives at risk, however, the financial damages as well as potential environmental repercussions are significantly higher in the second case. Thus, design parameters prescribed by the code provisions do not apply for the type of facilities in-hand. As an example, it can be noted that the behaviour or ductility factor that is currently proposed only for common building structures in European codes without taking justifiably the higher risk of industrial facilities into account. Furthermore, dynamic analysis methodologies are considered more appropriate for industrial structures due to the high irregularities in plan and mainly along the height e.g. stiffness, strength, geometry and/or weight of adjacent storeys. Usually, one or more nonbuilding structures e.g. pressure vessels and components with considerable mass vis-à-vis the supporting structure cause higher modes even torsional that cannot be captured by nonlinear static methods.

Figure 1. Typical configuration of nonstructural components and nonbuilding structures within a process plant (http://www.pentechglobal.com/en)

The seismic response of nonbuilding and nonstructural components may alter due to the Soil-Structure Interaction (SSI) effects. The soil deformability may have beneficial or detrimental impact depending on the soil properties and structural mechanical characteristics as has been shown for common building structures and nuclear power plants [Hoseyni et al., 2014; Karapetrou et al., 2015]. Although the soil is rather loose in midstream and downstream facilities due to the placement near the coastline, mainly liquid storage tanks have received attention upon this matter probably due to the long span and heaviness. However, the process plant pipe racks could be massive and rigid as shown in Figure 1 due to the numerous supported structures and components as well as rigid connection, and thus, the SSI effects may have negative impact on the most flexible nonstructural components. The foundation movements may lead in period elongation (over than 20% in stiff and heavy structures), which may further result in the decrease or increase of force and displacement demand. Nonstructural components e.g. pipelines are flexibly supported on pipe racks and thus the higher displacement demand could sensationally affect the design requirements. The earthquake characterization seems also to be predominant and supersede other sources e.g. modelling [Kwon & Elnashai., 2006]. Ground motion variability is reflected on structural response due to near- and far-source conditions that are discretised based upon the Epicentral Distance (ED), fling step and fault directivity. A complete examination of the seismic input parameters is out of the scope of the present study, thus, more information can be found in Pacor et al., [2018] and Stefanidou et al., [2017].

1.2 OBJECTIVE

A sound way of demonstrating each of the aforementioned parameter effects including uncertainties (demand) on the response of structural and nonstructural components (capacity) is the seismic fragility analysis. As discussed in the following section, there are different ways to pursue the fragility evaluation of a system; nonlinear static as well as dynamic analyses are employed depending on the structural system or the assessment target as well as other constraints. Fragility Functions (FFs) are an essential tool that risk managers and decision makers exploit for systemic seismic vulnerability and risk analysis within an oil/gas refinery or at community level. A succinct description of fragility analysis methodologies is conducted by highlighting the benefits and shortcomings (Section 2). Furthermore, the description of modelling of an LNG RC rack coupled and decoupled with pipelines considering also SSI is given in Section 3. Finally, the fragility assessment of the pipe rack is addressed where the fragility curves are estimated both for structural (meso-scale) and nonstructural components (local-scale). With the premise that fragility estimation is an indispensable part of Quantitative Risk Assessment (QRA) of down- and mid-stream facilities and recognising the high number of uncertainties included in the different steps of design and assessment, in a few words, this paper intents to form a starting point for further research and recommendations upon: the response variability of structural and nonstructural components by considering different seismic input (near- and far-field), soil deformability (linear and nonlinear), damage index (shear, chord rotation, pipe strain),

coupling as well as decoupling effects. The fragility curves that will be produced might not reflect the exact response of the system due to a number of limitations that are discussed in the following.

2.FRAGILITY ANALYSIS OF PROCESS PLANTS

The probabilistic seismic assessment of building structures has become quite popular particularly during the last two decades in earthquake engineering due to the increasing demand of correlating the seismic level intensity with the probability of damage (performance-based approach) towards minimizing the uncertainties e.g. seismic input or IM selection and achieving a more justifiable picture of existing or under design structures' response. There are three main parameters to be considered when selecting a fragility assessment methodology for structures, viz the structural type, the assessment target e.g. evaluation of serviceability or ultimate Limit State (LS) and time-constraints related to model scale and computational capacity. Regarding the former parameter, industrial structures similar to buildings e.g. pipe racks present horizontal (differential drift among perimetrical points) and mainly vertical irregularities e.g. inequality of stiffness, strength, geometry and/or weight between adjacent storeys. Pendulum modes that come from the support of suspended vessels and other equipment as well as higher local modes due to beam-to-column relative weakness constitute nonlinear static analysis methodologies inappropriate [Di Sarno & Karagiannakis., 2019]; the aforementioned facts lend weight to the use of probabilistic dynamic analysis methods that is reviewed hereafter.

2.1 METHODOLOGIES

Analytical fragility derivation methods rely on numerical models and thus the investigation of aleatory uncertainties e.g. soil properties or seismic excitation characteristics is more robust compared to empirical methods e.g. field observation or judgment. One common approach is the so-called Incremental Dynamic Analysis (IDA), where the structural model is subjected to a suite of ground motion records, each one scaled at various levels of Intensity Measures (IMs), resulting in response curves (or IDA curves) that parameterize the intensity level with the Damage Measure (DM) [Vamvatsikos & Cornell., 2002]. The outcomes of the IDA study are the understanding of structural response with the increase of IM e.g. peak deformation pattern and stiffness or strength degradation, the understanding of global system stability from one record to another and calculation of useful estimates of dynamic capacity e.g. mean annual frequency of collapse. The IDA curve response "path" strongly depends on the structural model and accelerogram. The selection of Damage Measure (DM) in the abscissa depends on the structural model; for instance, the Interstorey Drift Ratio (IDR) in the vicinity where nonbuilding structures not similar to building are supported on, the base shear resistance (V) or the peak floor acceleration for the assessment of nonstructural components could be a choice for oil/gas nonbuilding structures. Many researchers have expressed the concern that records of low magnitude cannot be representative of stronger ones, and this is one reason along with the high computational effort for scaling each record to large IM values till collapse that this method may not be adopted for structural assessment. To this effect, the truncated IDA is used that proposes the record scaling to be performed up to a maximum IM level (since higher levels are of less interest), above of which no further analysis is conducted. The latter method requires different fitting methods than the straightforward IDA, as discussed in the following.

On the other hand, the Multiple-Stripes Analysis (MSA) method is performed at specific IMs each of which has a unique set of ground motions [Jalayer & Cornell, 2009]. It has been found that the use of the same record at different stripes may not affect the results substantially [Baker, 2015]. Both MSA and IDA can be characterised as wide-range assessment methods, since they can be conducted for a large range of IMs. The main competitive edge of MSA versus IDA is the efficiency since it is performed at discrete and more practical IMs and sometimes accuracy due to the compatibility of records with the conditional spectrum at different IMs. Furthermore, another popular method in recent years that attempts to substitute the IDA

towards minimizing even more the computational time by performing analysis at different IMs for un-scaled records is the so-called Cloud Analysis (CA). The CA is not only used to describe the ground motion variability (a.k.a. uncertainty in ground motion representation) but also to propagate other types of uncertainties such as modelling or component capacity. More information about this method can be found in [Fatemeh Jalayer et al., 2015]. Except for these probabilistic assessment methods, there are modified or new ones, however, a thorough examination of methods is out of the scope of the present work.

2.1.1 Fitting approaches

It has been found that the IM distribution with respect to structural response follows the Lognormal Distribution [Eads et al., 2013]. A random variable X e.g. IM is lognormally distributed if Y=lnX has lognormal distribution. The lognormal Cumulative Distribution Function (CDF) that is used to describe the relation between the IM and probability of collapse (CDF because we would like to find the probability of collapse cumulatively up to an IM) holds [Baker, 2015]:

$$P(C|IM = x) = \Phi\left(\frac{\ln\left(\frac{x}{\theta}\right)}{\beta}\right) \tag{1}$$

where P(C|IM=x) is the probability of collapse given the IM=x, Φ is the CDF function and θ&β are the median and standard deviation (or dispersion of IM), respectively. The first and second moment of the distribution can be calculated easily from common formulas, however, it should emphasised that both refer to the natural logarithm of the IM (Y=lnX) and not the common values. In case that the truncated IDA is adopted to minimize the computational effort, another fitting approach is used, viz the maximum likelihood method, given that record motions cause no collapse. If L_C and L_{NC} are the probabilities that a ground motion with IM_i causes ($IM_i<IM_{max}$) and do not cause collapse, respectively, then it holds that the probability of observing the entire data set is the product of individual probabilities:

$$L = \prod_i^m L_{C_i} \cdot L_{NC}^{n-m} \tag{2}$$

where n-m and m are the number of ground motions that cause and dot not cause collapse, respectively. Actually, the L_C or L_{NC} is the Probability Density Function (PDF), which is equal to the derivative of CDF Φ (usually sympbolised with φ or f). In this case, to derive the lognormal CDF, the engineering analyst should try repetitively to maximise the likelihood L for different combinations of moments θ and β.

In contrast with the previous methods, the MSA does not correlate the IM with the onset of collapse for a given ground motion IM_i; instead, it relies on the fraction of records that cause collapse. This statement can be correlated with Bernoulli trials and binomial distribution where a structure collapses (success) or do not collapse (failure) accounting that individual seismic records are mutually exclusive statements. Finally, the linear regression method in the logarithmic scale is used for the data fitting in the CA method. The interested reader may find more information in Jalayer et al., [2015].

2.2 PREVIOUS STUDIES

This section is devoted to the IDA method that has been used up to date mainly for concrete buildings and nonstructural components in order to review the results in conjunction with the fragility analysis examination of RC pipe rack that is used as a case-study in the sequel. To the best of our knowledge, there is no research effort that undertake the fragility analysis of nonbuilding similar to building structures or pipe racks accounting for SSI and coupling with nonstructural components, yet the research has focused on liquid storage tanks, nuclear power plant containment structures or decoupled piping systems and components. Pipe racks could be heavy and stiff due to the nonbuilding structures and nonstructural components supported on them, and this may result in unpredictable response due to the seismic input variability and soil-structure interaction effects. The number of intrinsic uncertainties e.g. in modelling,

seismic input or soil effects, which increases further due to the inadequate code provisions that cause engineers to overlook important design aspects for nonbuilding structures, makes the seismic fragility investigation of pipe racks a *sine qua non* for the integrity of mid- and down-stream facilities.

A fragility assessment of a sample of existing regular RC buildings according to EN 1998-3., [2004] by considering both flexural and shear failure modes as well as PGA as IM (the same code and parameters are used for the assessment of the following case-study) was conducted in Tsonis and Fardis., [2014]. Among others, the research showed that the shear failure mode is the predominant for the collapse LS of columns (the same deduction is made for the RC rack) and the modelling e.g. space of stirrups in columns could affect considerably the fragility. The seismic assessment of two sets of RC regular buildings of different height and ductility was examined in Al Mamun & Saatcioglu., [2017] via IDA. Buildings with higher ductility showed greater fragility at the design compared to the effective period spectral acceleration. The selection of period to be considered for the spectrum may deserve more attention, particularly for critical nonbuilding structures. The fragility curves were constructed according to spectral acceleration at the design and effective period of buildings.

Except for the structural fragility derivation, assessment methodologies intend to evaluate as much as possible the number of uncertainties. A sufficient number of uncertainty sources related to record selection (natural and artificial records were selected accounting for spectral acceleration-to-velocity ratio), statistical interpretation of results and material modelling were considered within a fragility analysis framework of RC building in Kwon & Elnashai., [2006]. The most relevant result was the significant higher effect of seismic source conditions compared to modelling parameters. This conclusion comes in unison with the SSI effects on the RC rack when soil-inelasticity (modelling) is accounted for as discussed in the following case-study. The investigation of seismic response of a set of RC frames that represent the modelling/epistemic uncertainty by adopting the IDA method for a suite of ground motions (aleatory uncertainty) is conducted in Dolsek., [2009]. It was found that the initial stiffness, concrete strength and ultimate rotation of columns are the most considerable random variables of epistemic uncertainty, however, the structural response (or dispersion) is not substantially affected by the epistemic uncertainty, particularly, far from collapse. The IDA was significantly time consuming and surely cannot be used for the assessment of large or complex models.

The assessment of nonstructural components, which mostly present flexible behaviour and high geometrical irregularities e.g. piping, relies on different damage parameters, therefore alternative probabilistic estimators and methodologies should be adopted. A reasonable question that arises when investigating the seismic response of nonstructural components is the reliable correlation of IM with the seismic response. Also, the assessment of nonstructural components strongly relies on the Engineering Demand Parameter (EDP) selection; pipe failure is described by three main modes, namely tensile fracture, local buckling and fatigue cracking (more information can be found in Vathi et al., [2017]). In both cases, the selection is not straightforward; for example, piping systems may rigidly or flexibly outfit a rack and present different configuration according to supporting structure layout. Thus, the selection of common IMs e.g. Sa or PGA and unique EDP is not recommended due to the high geometrical nonlinearities and pipe fittings that can present critical response. For instance, the IDA method is applied to nonstructural components for near- and far-field conditions (rigid blocks) that are included in healthcare facilities by considering different IMs in Di Sarno et al., [2017]. The research clearly showed that the PGA is the most efficient IM for short rigid blocks, whereas PGV was the most appropriate for taller ones and the seismic input sets did not affect substantially the damage dispersion. The necessity of ensuring acceptable seismic performance of nonstructural components existing in mid- and down-stream facilities is the same or even higher considering the repercussions a failure may cause to human, environment and financial state at regional or supra-regional level. The fragility estimation of a nuclear power plant piping system was addressed in Salimi Firoozabad et al., [2015] and it was found that the fragility estimation yields better results when considering as IM the relative displacement between the pipe support and the ground.

3.MODEL DESCRIPTION

A RC pipe rack included in an existing LNG terminal is adressed. Except for the process pipe rack area, the terminal constitutes several utility zones (Figure 2a) e.g. knock-out drum, liquefaction, flare, ethylene storage tank that are interconnected with pipelines. We will focus on the pipe rack (Figure 2b) that serves so as to support the pipelines that come from the tank, which distribute LNG to the surrounding units. The pipes are supported on the rack for safety and operational purposes e.g. maintentance. Even though the rack was placed in low-seismicity region in the initial design, in the framework of this study, the RC rack is placed in one of the highest seismicity regions in Italy, Priolo Gargalo in the southeastern part of Sicily. To this effect, the performance of the most critical components can be highlighted. The RC rack consists of two sub-racks (Figure 2b); a 6x9x8.3 m short rack that supports the pipelines that come immediately from the LNG storage tank and a 102x6x7.3 long rack that transfer LNG to nearby units. All the intermediate spans formed by the beams are 3 m long. Also, an intermediate floor exists at 5.3 m.

The structural members are made by concrete of grade C40/50 and reinforcement S500. The concrete material is described by the Mander model, which accounts for transverse reinforcement, whereas the Menegotto-Pinto one is used for the ribbed reinforcement with isotropic hardening. Distributed inelasticity was attributed to columns (600x600mm) and beams (350x350mm) described by inelastic force-based frame elements that rely on the nonlinear fibre section method (uniaxial stress-strain relationship). Structural members with 6 Gauss-Lobatto integration sections is considered an acceptable choice to describe the spread of soil inelasticity. It is common for pipe racks, additional uniform load to be considered for the beams where pipes are supported on for safety and future installation of pipes. Thus, all the beams on the upper floor were subjected to 4kN/m, which refer to two times the maximum concentrated load that a pipe applies on beam.

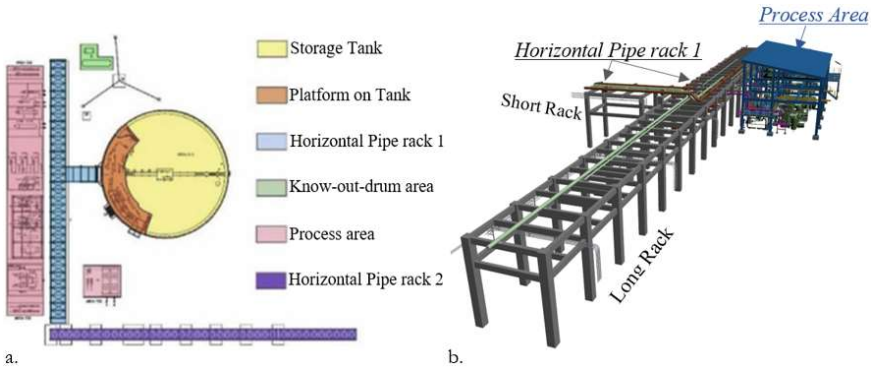

Figure 2a. LNG terminal layout., b. the pipe rack under consideration and a process area

Numerous pipelines are supported on the pipe rack, however, to facilitate the modelling, only seven pipelines that run from the short to the long rack have been selected for the fragility assessment. The pipelines are all welded and tranfer ethylene. To take the weight of the material into account, the density of steel has increased correspondingly. The A312/TP304L material has been used for the pipes (more info can be found in Bursi et al., [2018]). It should be emphasised that in view of large model under investigation and the availabilities of the analysis software [SeismoSoft, 2018], stick pipe models are used in both coupled and decoupled case. In particular, the pipe bents have been calibrated according to Bursi et al., [2015]. Also, the low operating pipe pressure (P_{max}=1.63 MPa) will not have significant impact on the pipe response and in order to stay on the safe side it was neglected [Bursi et al., 2015; 2018]. More information about the geometrical and mechanical properties of the RC rack and pipelines can be found in Bursi et al., [2018] and Di Sarno & Karagiannakis., [2019].

The pipelines layout in the decoupled case is demonstrated in Figure 3 along with the different Boundary Conditions (BCs) that were considered. The modelling of the entire length of pipelines is not practically feasible due to the complexity and high computational cost, thus any part of pipelines that runs out of the main frame of the rack is not considered. Usually, pinned connections are adopted, which is a conservative assumption, since pipes that present relative flexibility -mostly bend downwards or upwards after the main frame- are considered more rigidly restrained. Regarding the internal supports of the pipes on the rack, it is generally acceptable the longitudinal and all the rotational degrees of freedom to be unrestrained in order to attribute flexibility to the system (roller support). During the initial design process of the rack that placed the structure in a low-seismicity region, except for the previous type of BCs, fixed supports were considered as well, probably, for operational purposes. To examine the performance of the pipes and rack for earthquake events of lower probability of occurrence, that consideration was not modified in our case study (Figure 3).

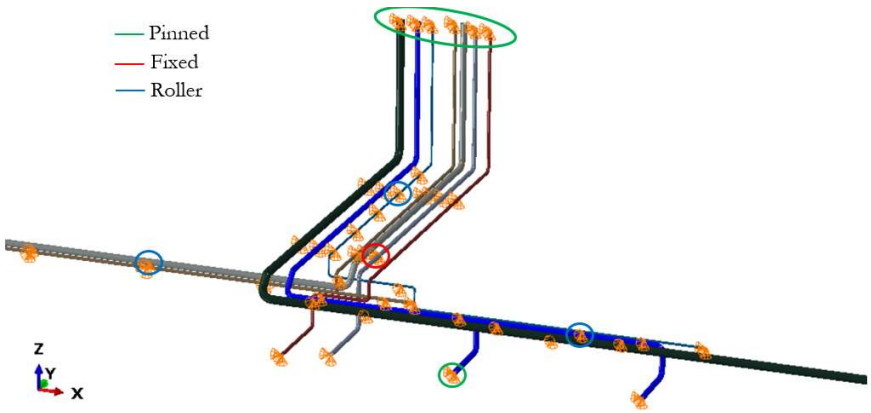

Figure 3. The pipelines and boundary condition types considered

The consideration of link elements under the footing and strip beams of the pipe rack is considered acceptable by code-of-practice provisions and computational efficient [Elnashai & Di Sarno, 2015]. Lumped springs that represent the soil compliance are used mainly for shallow foundations making the assumption that the superstructure is underneath by a homogenous, elastic and semi-infinite medium. When the foundation is rigid enough and the seismic excitation is not severe, the assumption of linear behaviour of soil in the vicinity of foundation could be acceptable. To acquire a better insight of soil-nonlinearity effects, both linear and nonlinear springs are calibrated. For the former case, the mechanical properties are estimated based upon the formulae derived by Mylonakis et al., [2006], whereas for the latter case an ad-hoc calibration of Ramberg Osgood (RO) hysteretic curve is conducted in MATLAB., [2018] by using the Root Mean Square Error (RMSE). The type of soil that is used for the elastic foundation design [Fardis., 2009] and the calibration of original G-γ-d curves is categorized as soil type C in EN 1998-1., [2004] (V_s=210 m/s, G_{max}=105 GPa and Poisson ratio ν=0.33). This soil type constitutes alluvial clayey sand to sandy clay that can be found at seaside. The calibration of original G-γ-d curve was attained through the formulae proposed by Ishibashi & Zhang., [1993] for plastic clay. More information about the calibration of original G-γ-d curve and the calibration can be found in Di Sarno & Karagiannakis., [2019]. The final model that considers the soil-foundation, the pipe rack and pipelines is shown in Figure 4.

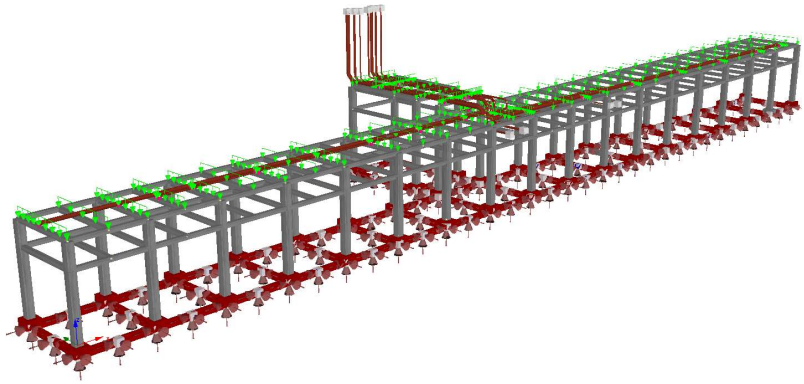

Figure 4. the mathematical model of soil-foundation, pipe rack and pipelines

4.ASSESSMENT OF RC RACK

4.1 METHODOLOGY

The robust assessment of RC pipe rack entails six main successive steps that are illustrated in Figure 5 and used as guidance for the consecutive paragraphs. After the material and frame element modelling that were presented in the previous section, a pushover analysis is conducted for the identification of the weakest direction based upon the fundamental frequencies observed from the eigenvalue analysis. The selection of records considering the fundamental period should comply with the recommendations of EN 1998-1., [2004]. In this case study, seven spectrum compatible records have been found both for far- and near-field conditions for the SLLS by using the REXEL software [Iervolino et al., 2010]. The discretization of seismic input relies only on the epicentral distance being greater or lower than 15 kms (more information about this selection can be found in Heydari & Mousavi., [2015]). The records attain compatibility in the horizontal direction at least, though, the three components are used for the seismic excitation.

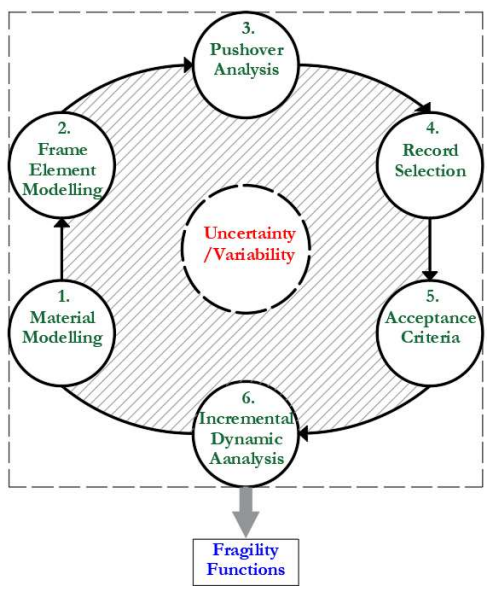

Figure 5. Seismic assessment flow diagram considered for the pipe rack

The fifth step in the assessment process includes the damage characterisation. Performance levels or LSs are defined for the rack as well as pipelines. The consideration of three LSs is a common assumption in

PBEE that the present study adopts. Concerning the structural members, two failures modes are considered, viz shear and flexure, according to Fardis., [2014]; the first defines the failure of individual members or macroelements and this is why the failure mode type may be categorised within the "meso-scale". On the other hand, failure due to exceedance of chord rotation or storey-drift ratio pertains to "global-scale" since the structural members involved are more than one. Finally, failure in tension and local buckling is considered for the pipelines according to Vathi et al., [2017]. It should be emphasised that pipe failure is an immediate type of failure on a critical component that poses a sudden threat within an industrial facility, and thus, it can be categorised in the "local scale". The distinguishment of failures in three scales might be used for future risk assessment, since it provides with useful information on the severity of earthquake and prioritization of prevention actions. The three LSs, namely Serviceability Limit State (SLS), Safe-Life Limit State (SLLS) and Collapse Limit State (CLS) for each of the failure mode of pipes are shown in Table 1 for structural and nonstructural components, respectively.

In particular, the SLS pertains to tensile strain ε_p=0.5%, whereas the conservative value of ε_{Tu}=2% is considered as ultimate tensile strain. Furthermore, the compressive strain resistance ε_{Cu} in the axial pipe direction primarily depends on the diameter to thickness ratio (D/t) and is given by:

$$\varepsilon_{Cu} = 0.5\left(\frac{t}{D}\right) - 0.0025 + 3000\left(\frac{\sigma_h}{E}\right)^2 \tag{3}$$

where σ_h is the hoop stress due to internal pressure, which is zero in our case, and E is the elastic modulus.

Table 1. Acceptance criteria of steel pipelines [Vathi et al., 2017]

Mechanism	EDP	Performance Level	Limit States (LSs)
Tensile fracture	tensile strain, ε_T	$\varepsilon_T < \varepsilon_T \leq \varepsilon_P$ $\varepsilon_P < \varepsilon_T \leq \varepsilon_{Tu}$ $\varepsilon_T \geq \varepsilon_{Tu}$	SLS SLLS CLS
Local buckling	compressive strain, ε_C	$\varepsilon_Y < \varepsilon_C \leq \varepsilon_{Cu}$ $\varepsilon_{Cu} < \varepsilon_C \leq 5\varepsilon_{Cu}$ $\varepsilon_C \geq 5\varepsilon_{Cu}$	SLS SLLS CLS

The final step of the assessment framework includes the decision-making for the fragility analysis selection. Due to the limited number of earthquakes found given the constraints on epicentral distance and spectrum compatibility, the IDA method is employed within a wide range of IM ($0 \leq a_g \leq 1$ for PGA). The number of ground motions may be insufficient to fully depict the record-to-record variability, however, it constitutes the minimum number of records for seismic assessment proposed in ASCE/SEI 7-16., (2017). Scaling step equal to 0.05g is used and the scaling factor is applied both for the horizontal (H) and vertical (V) component in order to keep the ratio V/H constant.

4.2 STRUCTURAL COMPONENTS

The pipe rack is assessed first in the decoupled and coupled case for far-field conditions as well as linear and nonlinear soil in order to evaluate the effects of dynamic interaction between the structural and nonstructural components as well as soil deformability. The fragility functions of beams are illustrated in Figure 6 only for the meso-scale (shear) since the failure in flexure was not predominant. Comparing the results between Figure 6a&b, it can easily be observed that the coupled case (consideration of nonstructural components) sensationally deteriorates the fragility of beams shifting the curves on the left. For instance, the increase in fragility for the design PGA (a_g=0.25g, dashed line) without SSI and far-field conditions is estimated nearly 125% for the SLS. When it comes to SLLS, the probability of exceedance soars from zero to 70%, approximately. This behaviour is attributed to the initial design assumptions and type of pipe-to-

beams constraints, as discussed also in the following. With regard to SSI effects, considering again linear soil and decoupled case, the increase is estimated around 75% for SLS at the design PGA. For higher values of IM e.g. $2a_g$, the increase comes up to 60% and 100% for the SLLS and CLS, respectively. The pertinent values for nonlinear soil are much higher e.g. 100% for the SLS. Considering now the results for the near-field conditions (Figure 6c), there are two intriguing comments to be made in conjunction with the far-field; first, the fragility is lower due to the near-field sources e.g. -100% for SLS and a_g and secondly, it may be a function of both the IM and LS as shown for the nonlinear soil. In particular, it is illustrated in Figure 6c that the SSI may affect detrimentally the response at lower levels of IM and first two LSs, however, this tendency changes conversely for higher PGA.

The representation of curves for columns as well as other combinations is omitted here for brevity. It is mentioned only that beams fail earlier than columns for the SLS and columns are less influenced by the pipe's response in the coupled case. The results in terms of dispersion of the beams are included in Table 2. Lower dispersion is observed for far-field conditions. The most intriguing outcome comes from the soil nonlinearity that caused rather low dispersion values in far-field conditions; notwithstanding, this tendency was not observed for near-field source.

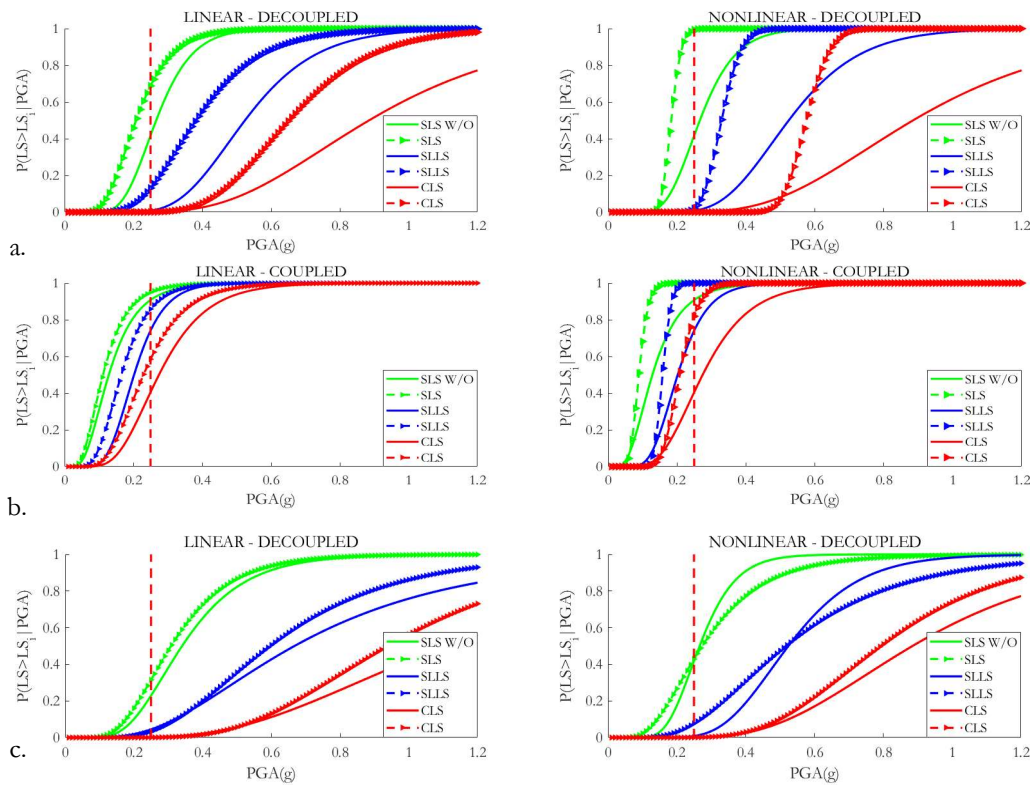

Figure 6. Fragility curves for beams considering a&b. far-field conditions, c. near-field conditions (red dashed line represents the design PGA=0.25g)

Table 2. Dispersion β for beams in decoupled case

		W/O SSI			W/ SSI - Linear			W/ SSI - Nonlinear		
		SLS	SLLS	CLS	SLS	SLLS	CLS	SLS	SLLS	CLS
PGA	Far-field	0.31	0.31	0.41	0.37	0.38	0.32	0.12	0.13	0.09
	Near-field	0.42	0.58	0.50	0.43	0.47	0.40	0.50	0.51	0.36

4.3 NONSTRUCTURAL COMPONENTS

The consideration of the piping system BCs without modifying the initial modalities resulted in the significant dependence of fragility curves upon the response of most critical components and particularly fixed supports. First, it is considered important to compare the response of structural and nonstructural components in unison and without dynamic interaction (Figure 7a&b). The fragility of pipes is greater than the structural members, however, the dynamic interaction has an inverse effect making the beams the most vulnerable component. This is justified by the excessive force that the pipes impose on beams on fixed points leading in the prior failure in shear. Also, another remarkable result concerns the risk that pipes pose on the system in two cases; the probability of exceedance soars from nearly 5% to 25% for the SLLS and a_g value, however, the fragility is mildly different for the SLS. With respect to SSI effects on pipelines, which is another interesting topic since very few publications deal with this issue as well, the soil deformability has a negative impact, which is not as much high as for concrete and beams members. Also, the detrimental influence is proved to be less in coupled case. This behaviour can be attributed to the soil deformability that act as a safe pad for the flexible components. The likelihood increases from 5 to 13% in the decoupled case and SLLS, whereas in the inverse case it comes up from 25% to 29%. Finally, the dispersion and median of pipelines fragility functions are quoted in Table 3. The dispersion remains almost the same in both cases and the median decreases mildly for linear soil.

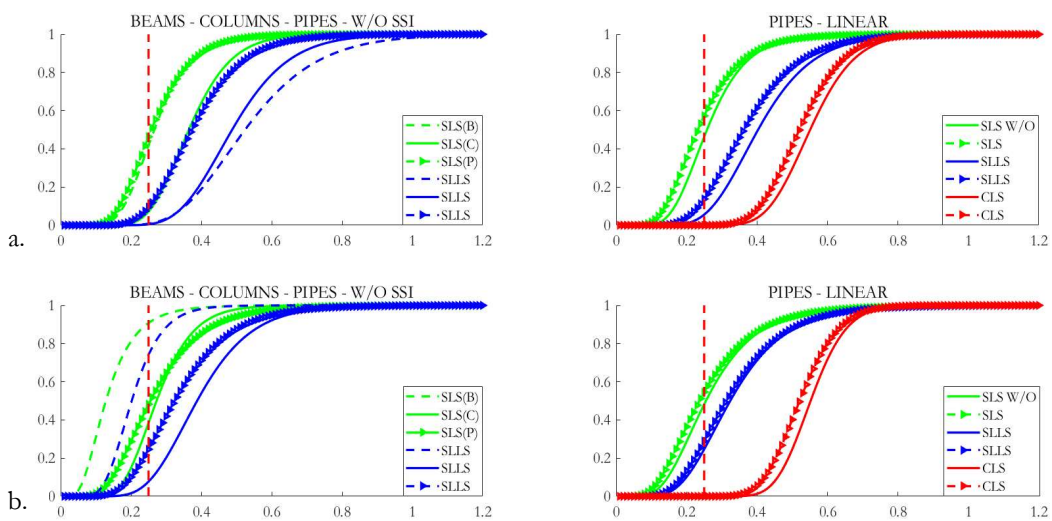

Figure 7. Fragility curves in a. decoupled and b. coupled case considering far-field conditions (red dashed line represents the design PGA=0.25g, B: beam, C: column & P: pipe)

Table 3. Dispersion β and media θ for pipelines in far-field conditions

		W/O SSI			W/ SSI - Linear		
		SLS	SLLS	CLS	SLS	SLLS	CLS
β	Decoupled	0.34	0.30	0.18	0.40	0.34	0.19
	Coupled	0.43	0.38	0.14	0.48	0.40	0.17
θ	Decoupled	0.26	0.40	0.55	0.23	0.36	0.52
	Coupled	0.25	0.32	0.55	0.24	0.31	0.52

5.CONCLUSIONS AND FURTHER WORK

In the present work, an attempt was made to estimate the probability of exceedance of three performance levels for structural and nonstructural components in coupled and decoupled case accounting for SSI and different source conditions. The main outcomes of the present study are summarised as follows:

- The dynamic coupling significantly deteriorates the response of beams due to the primary assumption for the boundary conditions; however, the nonstructural components response is comparable among the LSs.
- The SSI has a negative impact on nonstructural and particularly structural components. It was observed roughly 75% rise in the fragility of beams with linear soil, decoupled case, SLS and a_g seismic intensity, and the increase was even greater for more severe ground motions. Also, the fragility goes up even further for soil inelasticity; in the last case, the dispersion of IM was the lowest among all the other cases. The soil detrimental effect was not the most significant parameter for the nonstructural components.
- The comparison of near- and far-field records yielded higher dispersion for the former case.

To recapitulate, the modelling (e.g. boundary conditions of pipe supports), the seismic input (near- and far-field conditions) as well as the soil deformability were found to detrimentally affect the fragility of the entire system. The pipelines response was not always consistent with the negative effects of dynamic coupling and soil deformability on structural members due to the incompliance of pipe rack – piping system. Further research is required on the record-to-record variability considering more than 7 records and selection of alternative IMs for concrete members and pipelines. The derived fragility curves do not necessarily reflect the precise seismic fragility of the system due to the aforementioned limitations and cannot be used for other pipe racks within a risk assessment procedure due to the idiosyncrasies of the present system.

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