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SEISMIC FRAGILITY ASSESSMENT OF LNG PIPE RACK ACCOUNTING FOR SOIL-STRUCTURE INTERACTION

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Abstract

The increasing momentum for Natural Gas exploitation in Europe and worldwide constitutes Liquified Gas terminals indispensable links of energy supply network. Infrastructures of this kind should be resilient against the earthquake hazard and thus designed accounting for as much as possible sources of uncertainty such as modelling issues, analysis methods, seismic input selection, soil effects and others. To date, research efforts have not assessed the response of pipe racks sufficiently, let alone the interaction between the rack and piping system and analysis methods of the past have proved to be neither adequate nor efficient towards evaluating the earthquake hazard potentiality. Further, soil-structure-interaction has not been incorporated into a fragility analysis framework; albeit it is considered as a critical parameter since midstream and downstream facilities are usually rested on alluvial deposits.

In the present work, a supporting RC rack is analysed through a 3D finite element model in the nonlinear regime both as coupled and decoupled vis-à-vis a piping system. The fragility functions are evaluated for structural components and limit states in the global and meso-scale, through the Incremental Dynamic Analysis (IDA) considering far-field conditions. In the end, the SSI is encountered accounting for linear and nonlinear soil, and soil effects are demonstrated by the fragility curves. It is inferred that the fragility of the rack soars considerably by the piping system boundary conditions in the coupled case and the SSI has detrimental impact, and thus should be accounted, particularly, for industrial structures that are located at coastal sites.

Keywords: RC pipe rack, pipelines, dynamic interaction, fragility assessment, far-field, soil-structure-interaction.

1 INTRODUCTION

The strategic role of Liquified Natural Gas (LNG) terminals all over the world and particularly in Europe due to the high energy demands, which have been increasing recently, makes these infrastructures focal points of energy world network. Climate changes due to global warming was, is and will be high on the agenda of societies, and thus the consumption of more environmental friendly fuels e.g. LNG will be prioritized in the future. There are currently 28 LNG terminals (24 ground and 4 floating) along European coastline some of which are planned or being expanded and the number is going to increase considerably in the Mediterranean Sea by the first half of the next decade since the LNG imports are expected to soar by almost 20% by 2040 compared to 2016 levels [1]. Furthermore, ongoing European projects, e.g. EastMed or future exploitation of natural resources in the East Mediterranean regions, will probably support exports to global markets and Europe via LNG or pipelines increasing even more the need for constructing LNG terminals in the region. The terminals play an essential part since their purpose is to store, process and distribute natural gas mainly by freight and pipelines at regional or supra-regional level. There are two types of terminals, namely liquefaction and regasification (or off-loading facility); in the first type, the gas is liquified by compression and cooling to low temperature, whereas in the second case, the LNG is converted to its gaseous form for further distribution to the market (Figure 1a). The natural gas is liquified because it takes up approximately 1/600 less volume compared to gaseous form. In this view, LNG terminals are made of a port, storage tanks and the main process area (Figure 1b) that includes pipe racks, knock-out drum (or vapor-liquid separator) and other process equipment.

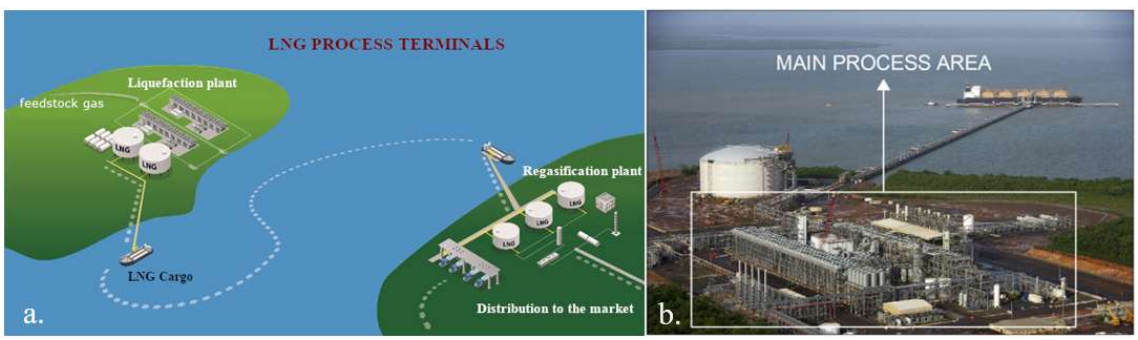

Figure 1: a. LNG process pathway and b. an LNG liquefaction plant ([2])

Many of LNG terminals could impose high risk to human and countries financial state considering that they might be vulnerable against natural hazards such as inundations, fire, earthquake and others. The earthquake hazard potentiality, which the present study intends to examine, is high in Central and East Mediterranean basin in which countries e.g. Italy, Greece and Turkey have experienced severe seismic events during the last decades. The seismic hazard is a key one to be addressed in the analysis of major accident hazards within a well-completed risk assessment of industrial facilities as clearly proved in [3]. A literature review that was conducted at the time showed that 14 seismic events caused damages to 182 equipment items and the 70% of that damage led to Loss of Containment (LOC) events. The number of non-building structures e.g. pipe rack and nonstructural components e.g. pipelines that an LNG or oil refinery encompasses are numerous. To adjudicate what the most vulnerable structures in a process plant are in order to account for their fragility during a quantitative risk assessment process (this kind of risk assessment is out of the scope of the present work), we should examine the seismic behaviour during past seismic events. In particular for oil refinery pipe racks, it has been found that are not the most vulnerable structures themselves against seismic hazard,

however, they could be due to the differential displacement of pipe supports that are not compatible with the pipelines ([4]). Supporting structures or pipe racks carry complex systems of nonbuilding structures and nonstructural components that transfer hazardous substances from one unit to another and thus their seismic integrity is critical.

To-date, the seismic research has focused on the decoupled case by investigating the seismic response of critical components such as elbows, t-joints or bolted flange joints without analyzing the rack and piping system in unison (coupled case) ([5], [6]). This engineering practice of decoupling the response of structural and nonstructural components has been adopted not only for research purposes but also in industrial sector in virtue of simplifying the design process due to the lack of code provisions or due to possible limitation of structural analysis software in the market that causes engineers to overlook important design aspects and overestimate or underestimate the seismic response. These limitations along with the ones that refer to modelling of pipes, the soil deformability or the selection of seismic input increase considerably the number of uncertainties (epistemic and aleatory since they are based both on the lack of knowledge e.g. modelling as well as inherent/unavoidable randomness e.g. soil properties or seismic excitation characteristics) that comes in stark contrast with the magnitude of process plants cruciality. Additionally, the Limit States (LSs) of common building structures are not applicable e.g. for pipe racks considering the exceptional high human, environmental or financial risk that a failure of structural or nonstructural component may induce [7]. That being said, it is essential to assess in a detailed fashion the dynamic interaction of pipe racks vis-à-vis pipelines or other supported components considering also that testing campaigns on this research topic cannot be found in the literature.

Another important aspect that may increase the seismic vulnerability particularly of mid- and down-stream facilities is the soil deformability. The soil beneath LNG terminals and oil refineries that are located at the seaside is rather weak, and thus strengthening measures are undertaken e.g. pile foundation so as to decrease possible settlement by dead loads or enhance the lateral capacity against earthquake loading. As it has been investigated up to a point for common buildings, the soil deformability affects mainly squat and heavy structures by increasing the lateral deformation and lessening the force demand ([8]). Steel and concrete pipe racks come in many sizes and layouts and it is definitely impossible for someone to deduce whether the soil should be considered or not in the final design. This is the reason that seismic code provisions encourage the incorporation of Soil-Structure Interaction (SSI) in the design, however, European ones in particular fail to form a practical way so as the SSI to be considered in the modelling of common buildings let alone for pipe racks that differentiate in many ways e.g. irregularity along the height and in plan. Furthermore, the decision could be even more complex by considering that pipelines are supported flexibly on racks and may be affected by the soil-induced higher displacements. To the best of our knowledge, there is no a probabilistic approach that undertake the seismic vulnerability of mid- or down-stream pipe racks – pipelines systems accounting for SSI, even though the demand for acquiring a better insight of the seismic vulnerability of existing or under design process plants is rather essential.

The present paper is organized as follows: first, the peculiarities of analysis methodologies of nonbuilding structures and nonstructural components as can be found in the current literature and codes of practice are presented in Section 2. Furthermore, it follows a brief review of soil-structure interaction effects and two models for linear and nonlinear soil that are adopted for the case-study (Section 3). In the last section, an LNG RC pipe rack is considered as a representative case study and the vulnerability at global- and meso-scale accounting for the coupled case is evaluated. The objective of the work herein is twofold: i) evaluation of the importance of dynamic interaction between an LNG RC rack and steel pipelines and ii) investigation of soil-deformability effects on structural components by means of Fragility Functions (FFs).

2 ANALYSIS METHODOLOGIES FOR PIPE RACKS

Petrochemical plant pipe racks are usually made by steel to avoid corrosion due to harsh environmental conditions that process plants are exposed to, however, concrete modular frames could be found as more preferable option due to the lower cost of the material, the high uncertainty in the welding process and time constraints relating to the long installation period of steel frames. They are complex systems since numerous nonbuilding structures and nonstructural components are supported on them (Figure 2). The detailed analysis methods of supported components specifically are excluded in this section; however, some references will be made to the point that are related to the connection with the supporting structure.

2.1 Code provisions

The main European (EN) contribution for seismic-resistant design of structures [9] do not make reference to oil refinery pipe racks yet to irregular building structures that, of course, differentiate in many ways e.g. different types of loading due to the supported nonbuilding and nonstructural components or importance class definition compared to nonbuilding structures. Even though EN regulations deal with other type of structures included in process plants such as tanks, silos, towers and pipelines ([10], [11]), the important design aspects of pipe racks along with the pipelines that are outfitted by are not mentioned. On the other hand, the American (AM) code [12] or the petrochemical plant structures guideline [13] encompass a few regulations for the design and analysis of pipe racks. Oil refinery pipe racks are called nonbuilding structures similar to buildings in the codes since they are designed and constructed in a manner similar to buildings, respond to strong ground motion in a fashion similar to buildings and constitute moment, braced or dual systems. Building-like structures share many design parameters and expected behaviour with regard to common building structures but there are also considerable differences.

Figure 2: An oil refinery complex with pipe racks and supported components (source: www.rainbow941.fm)

According to the AM code [12], several parameters should be considered for determining the analysis methodology of nonbuilding structures similar to buildings, viz the vertical and horizontal irregularities, the configuration of nonbuilding structures e.g. heat exchanger or tower vessels mass, the relative rigidity of beams that should not be confused with the rigid or flexible way of supporting e.g. a piping system on a nonbuilding structure, the Seismic Design Category (the SDS in the AM code is defined as a function of importance class, seismicity and site class) and the fundamental period, T. In contrast with the EN codes that are not dealt with

pipe racks, AM ones specify several analysis methodologies and give high latitude to the engineer in selecting the most suitable. Petrochemical pipe racks are usually subjected to vertical irregularities in comparison with horizontal ones (the last are defined in the code based upon the differential drift between perimetrical points when there is diaphragmatic behaviour but pipe racks most commonly have no diaphragms). The vertical irregularity could be due to the inequality of stiffness, strength, geometry and/or weight of adjacent storeys. For instance, it is rather common when a unique floor supports significant mass, whereas other adjacent carry inconsiderable nonstructural components, the equivalent lateral force analysis method may be applied since the response is dominated by the first mode; however, it is also common dissimilarities in stiffnesses and strength to exist due to the distinct vibration of concentrated masses on upper floors as well as geometry e.g. side overhang cantilevers that are necessary to support nonstructural components that run out of the main frame. The fact that pipe racks or other building-like structures present similar skeleton to buildings does not mean necessarily that behave like them. The resulting pendulum modes that come from the support of suspended vessels and other equipment may have strong impact on pipe rack response depending on the clearance and make the use of dynamic analysis inevitable. Another reason that necessitates the use of response spectrum or time history analysis is the higher local modes that come from the weakness of beams compared to columns or braces and may contribute substantially to the total response. This is a common behaviour of nonbuilding structures with absence of floor slab.

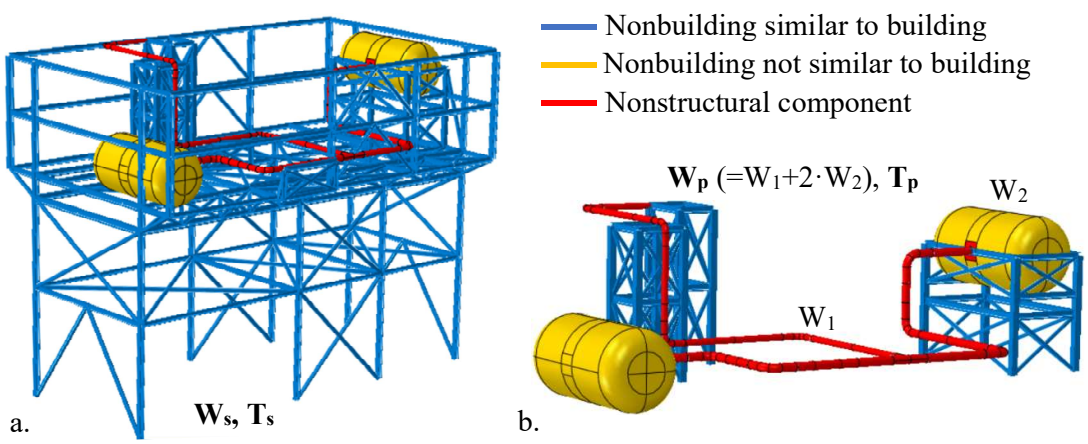

Figure 3: a. A steel nonbuilding structure, b. the piping system considering the towers that is support

To assess further what the AM code proposes regarding the interaction between supported nonbuilding structures and/or nonstructural components and supporting structure, a nonbuilding steel rack is demonstrated in Figure 3a consisting of a unique pipeline that runs across the third floor and connects two horizontal pressure vessels. When the weight of nonbuilding structure similar to building W_s (Figure 3a) and nonbuilding structures not similar to buildings as well as nonstructural components W_p (Figure 3b) is less than the 25% of the W_t (W_s+W_p), the decoupled case (no interaction) between them should be considered according to the code requirements. This is a very rough rule that relies on the low influence the supported components will have on the system response and intents some nonlinearity to be appeared on the nonbuilding structure to avoid resonance response and lessen the interaction. Also, the supporting structure and nonstructural components design parameters e.g. behaviour factor or component amplification factor a_p (this factor multiplies the design force and takes value between 1 and 2.5 for rigid and flexible supported components) are defined separately. Should this not be the

case, the dynamic interaction is accounted for; however, if nonbuilding structures and/or nonstructural components (the AM code states only the nonbuilding structure but implies the nonstructural components e.g. pipes) are rigidly attached to the supporting structure (T_p<0.06s, the value is estimated by considering the flexibility of beams that the components is attached to e.g. the towers in Figure 3b), they should be analysed as rigid elements considering only the behaviour factor of the rack, otherwise both the supporting and nonbuilding structure should be modelled together in a combined model (Figure 3a) adopting the lesser behaviour factor between them. Finally, the two predominant dynamic analyses proposed by the code is the response spectrum and dynamic linear analysis probably due to safety reasons since nonlinearity increases the number of uncertainties in modelling and interpretation of results, however, the code implicitly recommends the use of nonlinear time-history to be used with caution.

2.2 Previous studies

According to [14], the most commonly causes of industrial accidents are the human factor (commission or omission errors), the organization/management errors e.g. design deficiencies or lack of maintenance and equipment/mechanical failure e.g. material failure or malfunctioning of equipment. The investigation of 284 case studies that referred solely to chemical process plants accidents by [15] was in unison with the previous statement, but it yielded additionally that the majority of accidents (25%) was related to piping system. That conclusion indicates that the piping network is one of the most prone parts within a process plant and due consideration should be given to technical human errors e.g. defective design by positioning unjustifiably the pipes on the rack or incorrect pipe bents configuration for increasing the flexibility of the system due to thermal pressure.

To-date, it still remains to be examined the dynamic interaction between a supporting structure and a piping system towards highlighting the most critical design challenges and preventing accidents in the future that put human lives and environment at risk. To the best of our knowledge, the research is rather limited on this topic; the majority of research has focused on the analysis of piping system or critical components individually ([5] &[16] among others), whereas the analysis of the combined models (supporting structure and piping) is rather obscure. An interesting research upon the effects of dynamic interaction on pipe-way and piping system response by considering different weight ratio, diameters and end-condition of pipes as well as thickness of U-ring elements (they are used to capture the pipes along the perimeter restraining only the movement in vertical and transverse direction) was conducted in [17]. The governing result of the case-study referred to the significant role the end-conditions and the stiffness of U-bolt rings played in the seismic response; the last two parameters may sensationally affect the whole system more than the weight ratio. The outcomes of the research showed that in case of multi-secondary structures e.g. pipelines that are multiply supported on a supporting structure, the relative stiffness of connection and the end-conditions except for the weight ratio should be examined; however, these parameters are missing from the codes which deal with singe secondary systems that are rigidly or flexibly supported on a nonbuilding structure.

3 SOIL-STRUCTURE INTERACTION

This section is devoted in the review of SSI models and effects on buildings and similar to building structures. More attention will be given to former structural type due to the higher research interest depicted in the literature. The limited research on soil deformability influence on the latter structural type demands a better insight to be acquired that can assist during the design process both of structural and nonstructural components. The fact that oil refineries are

located close to seaside where the soil is rather weak makes the examination of SSI phenomenon a sine qua non for the seismic integrity.

3.1 State-of-the-art

Except for the well-known effects of SSI on system response, namely period elongation and damping increase, the SSI has been found to affect mostly heavy and stiff structures. The longer period as well as the added damping in the system reduce the seismic demand forces (lower response spectrum) and thus SSI could be beneficial from this point of view. On the other hand, the higher displacement demand due to the soil deformation could be strictly necessary within the displacement-based design particularly for pipe racks that are outfitted by sensitive to high-deformation pipelines. According to [8], the ratio of $h/(V_s \cdot T)$ (h is the effective height, V_s is the shear velocity and T is the natural period) could be a reliable indicator of the degree of period elongation and damping increase.

Depending on the structural type under investigation, different parameters are of interest to evaluate the effects of SSI. For instance, foundation settlement or sliding and change in dynamic properties could be of interest for common building structures. However, when it comes to nonbuilding structures such as bridges, pipe racks or nuclear power plant reactors, the seismic demand on piles as well as the peak floor acceleration for checking the response of nonstructural components are of importance. Usually, pile foundations are adopted for pipe racks due to the liquefiable alluvial deposits existing at coastal sites and the high area that oil refineries cover making the replacement of soil practically or financially infeasible.

Except for the identification of changes in the response of structures, which is still a challenging task particularly when soil nonlinearity is accounted for, the researchers have been trying during the last decade to evaluate the effects of SSI in a probabilistic manner by estimating fragility functions that constitute an essential tool for assessing the response of existing or under design structures for further loss estimation and risk management. Even though this attempt has been partially completed for common building structures and bridges ([18]–[20]), the research lacks clearly of a probabilistic methodology that investigates the damage of pipe racks in a probabilistic manner accounting for dynamic and soil-structure interaction. The influence of SSI has been examined mostly for storage tanks and nuclear containment structures ([21], [22]), although process plant pipe racks could be stiff due to the vertical and horizontal bracing that intend to keep low the ductility as well as heavy since other nonbuilding structures and components are usually supported on them [7].

3.2 SSI models

This section is the forerunner for the case-study that follows, thus SSI models will be examined in connection with the ones that have been adopted afterwards. The literature abounds with models that are used to describe the soil-foundation-structure interaction. These models can be categorized into three main categories, namely ‘domain type models’ that refer to local scale since the soil is examined by constitutive laws, macroelements (intermediate-scale) where the soil-foundation-structure interface is described by a link element and finally soil springs which is the simpler type of model that is mostly used by practitioners and is based on impedance functions by considering mostly only for the fundamental frequency of the superstructure (global scale). The last type of model could be computationally efficient accounting for soil nonlinearity response and is suggested by code-of-practice provisions as a practically acceptable method [8]. Having said that, this latter type of model will be described hereafter and used for the fragility analysis in the following case study.

Lumped springs that represent the soil compliance are used mainly for shallow foundations making the assumption that the superstructure is underneath by a homogeneous, elastic and semi-infinite medium [23]. The assumption of linear behaviour in the vicinity of foundation could be an acceptable approximation when the foundation is rigid and the seismic excitation is not enough to develop soil inelasticity. The energy dissipation of soil due to radiation and hysteretic damping is represented by dampers. Although the stiffness and damping of soil is frequency-dependent, usually, they are modelled as frequency independent at the fundamental frequency of the structure since the analysis is conducted in time domain.

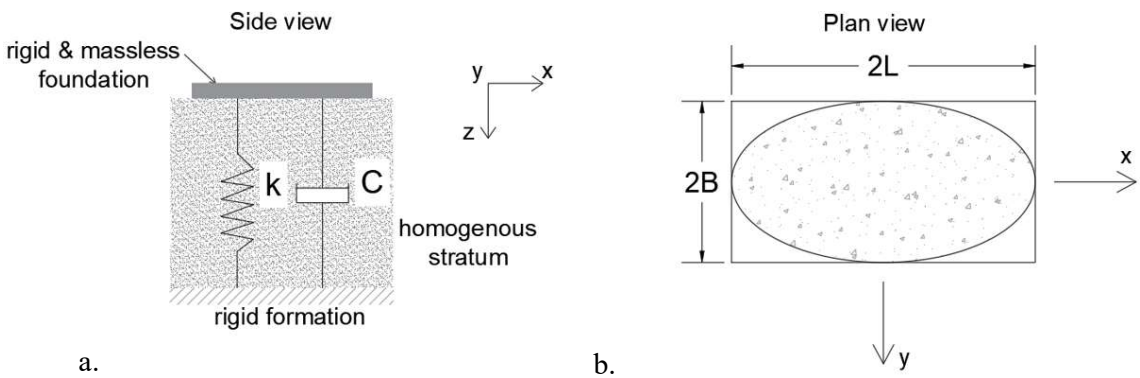

Figure 4: a. Idealisation of soil and foundation as a spring, dashpot and mass system, b. general shaped foundation (L>B) for the calculation of impedance functions

	K_i	C
u_z	$\frac{2GL}{1-\nu}(0.73+1.54\chi^{0.75})$	$\rho V_{La} B^2 \bar{c}_z$
u_y	$\frac{2GL}{2-\nu}(2+2.5\chi^{0.85})$	$\rho V_s A_b \bar{c}_z$
u_x	$K_y - \frac{0.2}{0.75-\nu} GL\left(1-\frac{B}{L}\right)$	$\rho V_s A_b$
r_z	$\frac{G}{1-\nu} J_t^{0.75}\left(4+11\left(1-\frac{B}{L}\right)^{10}\right)$	$\rho V_s J_t$
r_y	$\frac{G}{1-\nu} I_{by}^{0.75} 3\left(\frac{L}{B}\right)^{0.15}$	$\rho V_{La} I_{by} \bar{c}_{ry}$
r_x	$\frac{G}{1-\nu} I_t^{0.75}\left(\frac{L}{B}\right)^{0.25}\left(2.4+\frac{0.5B}{L}\right)$	$\rho V_{La} I_{bx} \bar{c}_{rx}$

I & J_t area and polar moment of inertia of the soil-foundation surface around the pertinent axis, $\bar{c}$ a damping factor with $\bar{c}=\bar{c}(L/B, a_o)$, $\chi= A_b/(4L^2)$, G the soil shear modulus, A_b foundation area

Table 1: Static stiffness K and radiation damping coefficient C for arbitrary shaped foundations ([24])

The dynamic impedance function of the system shown in Figure 4a is given by:

$$K(\omega) = \bar{K} + i \cdot \omega \cdot C \tag{1}$$

where both $\bar{K}$ and C are functions of frequency ω. In particular, $\bar{K}$ is called dynamic stiffness and represents the soil stiffness and inertia being frequency independent up to a good approximation for soil properties. The dynamic stiffness is given by:

$$\bar{K} = K_i \cdot k(\omega) \tag{2}$$

The static soil stiffness K_i in the three modes of vibration for a general shaped foundation (Figure 4b) rested on homogenous half-space is given in Table 1. The dynamic stiffness coefficient (k) is a function of L/B, the soil Poisson ratio ν and the dimensionless parameter a_o ($=\omega \cdot B/V_s$, ω is the fundamental angular frequency of the superstructure and V_s is the average soil shear wave velocity commonly estimated at the upper 30 m of a soil deposit). The coefficient k can be found by plots existing in the literature as a function of the aforementioned parameters. Furthermore, the total damping is represented by the product $\omega \cdot C$ where C is the radiation damping (hysteretic damping is zero for linear soil) coefficient that for a general shaper foundation are given also Table 1, where ρ is the soil density, V_{La} is the Lysmer's analog wave velocity ($=3.4V_s/(\pi \cdot (1-\nu))$) and c_i is a factor that is given by plots being a function of the dimensionless ratio α_o and the ratio L/B. The interested reader may find more information about the methodology for the soil springs configuration and the forms of rotational modes of vibration in [24].

The soil behaves in the nonlinear regime under strong-ground motion and in the vicinity of foundation interface (near-field) even at low strain levels. The hysteretic behaviour is described by the inclination (that refers to soil stiffness) and the size of the loop (the larger the loop the higher the energy dissipation, Figure 5a). Usually, the secant stiffness ($G_{sec}=\tau_c/\gamma_c$) is used to describe the average stiffness of soil along an entire loop instead of the tangent G_{tan} and the damping ξ for the dissipation of energy due to the material nonlinearity. The G_{sec} and ξ constitute the equivalent linear soil properties; the increase of shear strain amplitude decreases the secant modulus and increases the energy dissipation due to the hysteresis phenomenon. The change in shear modulus and damping are usually depicted through the G-γ-D curve ([25]) (Figure 5b).

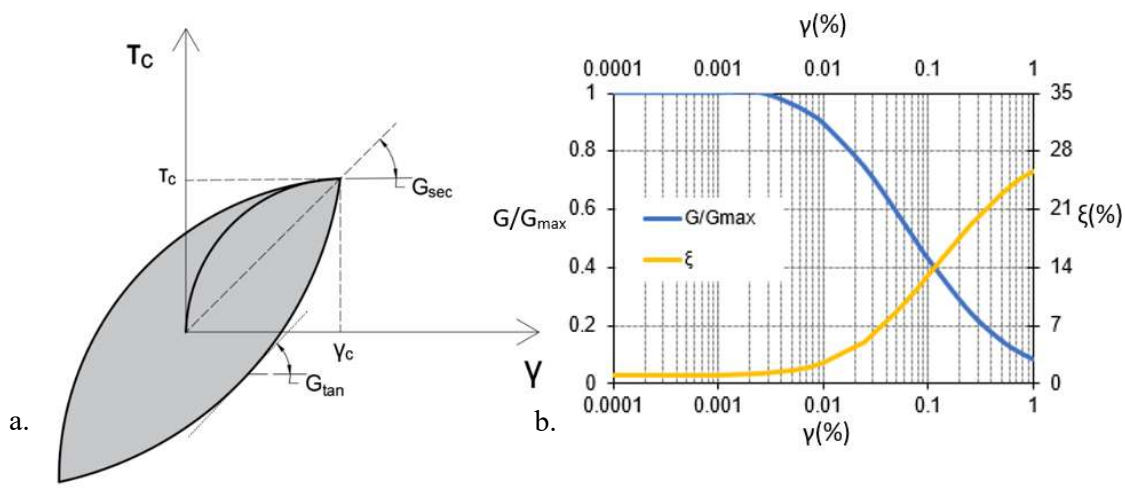

Figure 5: a. hysteresis loop of soil element subjected to symmetric cyclic loafing ([25]), and b. typical G-γ-D curves of a soil deposit

The simplest way to estimate the soil nonlinearity is the equivalent linear method that accounts for the modification of secant and damping during an iterative analysis procedure by considering a percentage (roughly 65%) of the peak strain from the previous step. More information about this method can be found in [26]. The equivalent nonlinear method is a straightforward and simple to be implemented procedure, however, it still remains an approximation of the actual soil behaviour. Thus, plentiful nonlinear constitutive models of different rigorousness and complexity exists in the literature. Soil models with hysteretic behaviour can be rather difficult to be calibrated since they require a lot of parameters. In this regard, a simplified model will be used in the sequel to account for the nonlinear behaviour of soil underneath an LNG RC

rack. That model relies on the Ramberg-Osgood (RO) curve [27], which is described by the following equation:

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{\tau_y}\left(1 + a\left|\frac{\tau}{\tau_y}\right|^{r-1}\right) \tag{3}$$

where γ_y and τ_y are the yield strain and stress ($G_{max}=t_y/\gamma_y$) and the parameters α and r are positive constants ($r\geq1$ and $a\geq0$). When the strain is very small ($\gamma\rightarrow0$ and $\tau\rightarrow0$ given that $r>1$), the eq. 7 can be rewritten according to Masing's rule as follows:

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{G_{max}\cdot\gamma_y}\left(1 + a\cdot\left|\frac{\tau}{G_{max}\cdot\gamma_y}\right|^{r-1}\right) \tag{4}$$

Also, the hysteretic damping according to RO model is defined as:

$$\frac{G}{G_{max}} = 1 - \frac{D\cdot\pi\cdot(r+1)}{2(r-1)} \tag{5}$$

The parameters a, r and γ_y are to be determined by using a code that tries repetitively to estimate the best fit given a soil type with G_{max}. This code has been developed in MATLAB and the results are presented in the following section. Once the parameters are known, D-γ curve can be determined as well.

To represent the soil properties of the following case study by using the G-γ-D curves shown in Figure 5b and calibrate the pertinent curves of RO model, afterwards, the empirical formulae by [28] have used that refer to alluvium deposit of sandy clay to clayey sand with plasticity I_p. The quotation of those formulae is omitted herein for brevity.

4 FRAGILITY ANALYSIS OF A RC PIPE RACK

The seismic fragility of a RC pipe rack is examined herein considering a 3D model in dynamic nonlinear analysis of framed structures software Seismostruct ([27]). Towards achieving robustness in the assessment methodology, a particular number of steps were followed for that purpose: i) material and frame element modelling, ii) modal analysis for identifying the fundamental modes in horizontal and vertical direction, iii) pushover analysis of the rack for determining the weakest direction, iv) selection of 7 spectrum compatible seismic records considering the predefined fundamental period, v) definition of the acceptance criteria for the assessment of structural components and, vi) time-history analyses and results compilation and representation via probabilistic functions. All the following steps are explained in more details in the sequel.

4.1 Model Description

The pipe rack that is examined comes from an existing LNG terminal plant (Figure 6a) and is shown in Figure 6b. Although the LNG terminal is originally placed and designed in a low-seismicity region, the pipe rack is replaced to Priolo Gargalo, a high seismicity area in southeast Sicily, in order to acquire a better insight of the seismic performance. The pipe rack under assessment consists of 2 sub-racks; 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 (Figure 6c). All the intermediate spans by the beams are 3 m long. The structural components are made by concrete class C40/50, whereas the 7 pipelines that run along the rack by steel grade ASTM A312/TP304L with yield and ultimate strength 370 MPa and 461 MPa, respectively. More information about the mechanical properties of the rack and pipelines can be

found in [29] and [30]. The concrete material is described by the Mander model, which accounts for transverse reinforcement, and the Menegotto-Pitto one is used for the ribbed reinforcement. The inelasticity of beams and columns is described by force-based frame elements that rely on nonlinear fibre section method [27]. A five-element non-uniform subdivision is adopted for the members.

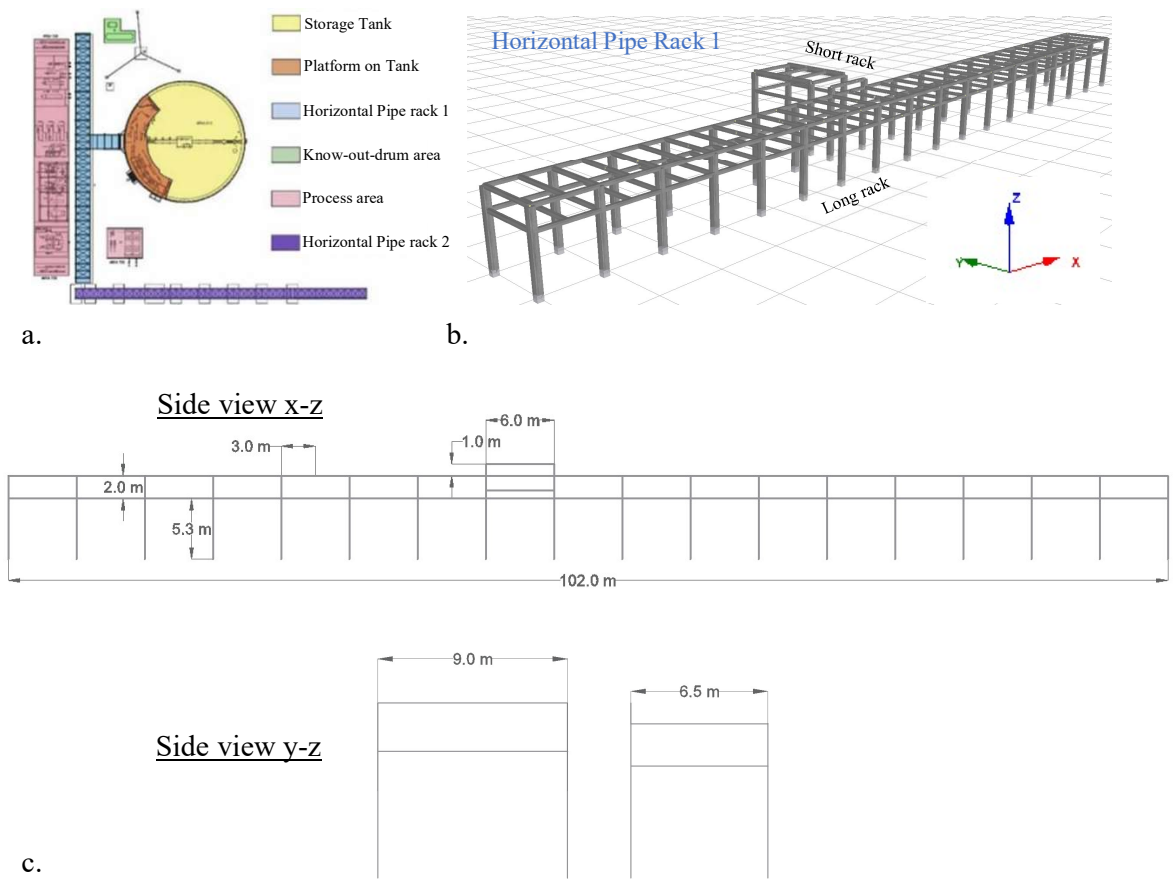

Figure 6: a. LGN terminal layout ([30]), b. RC pipe rack under assessment and c. dimension of the rack

Since the modelling of the entire terminal is not practically feasible due to the excessive computational cost, the assumption of pinned connections at the edges of the pipes that run beyond the pipe rack main frame is made. This decision is an approximation and states on the safe side since pipes that present relative flexibility -mostly bend downwards or upwards after the main frame of the rack- are considered more rigidly restrained. Furthermore, another type of modelling constraint refers to pipe modelling. Although EN and AM codes of pipelines design do not make reference of shell elements yet only to beam ones, the last type of element may be incapable of capturing the exact strain-stress response [7]. In view of the large model under investigation and the availabilities of the analysis software [27] for pipe modelling, stick models both for straight and curved pipe have been used and calibrated according to [32]. It is worth mentioning that the internal operating pipe pressure is low (P_{max}=1.63 MPa) as quoted in [30] and thus neglected in order to remain on the safe side since the pressure increases at low operating levels the bending stiffness of pipe bends ([32]). The pipelines are supported mainly in a flexible way on the rack by keeping free the displacement in the axial direction of the pipe as well as all the degrees of freedom.

Finally, to account for the SSI effects, an elastic footing and strip beams foundation system is designed according to [33] for the new site. A coastal sandy clay to clayey sand soil profile with V_s=210 m/s and G_{max}=105 GPa (STC according to [9]) has been adopted and the linear soil springs for the 6 modes of vibration are estimated according to [24] and placed both under the footings and strip beams as shown in Figure 7.

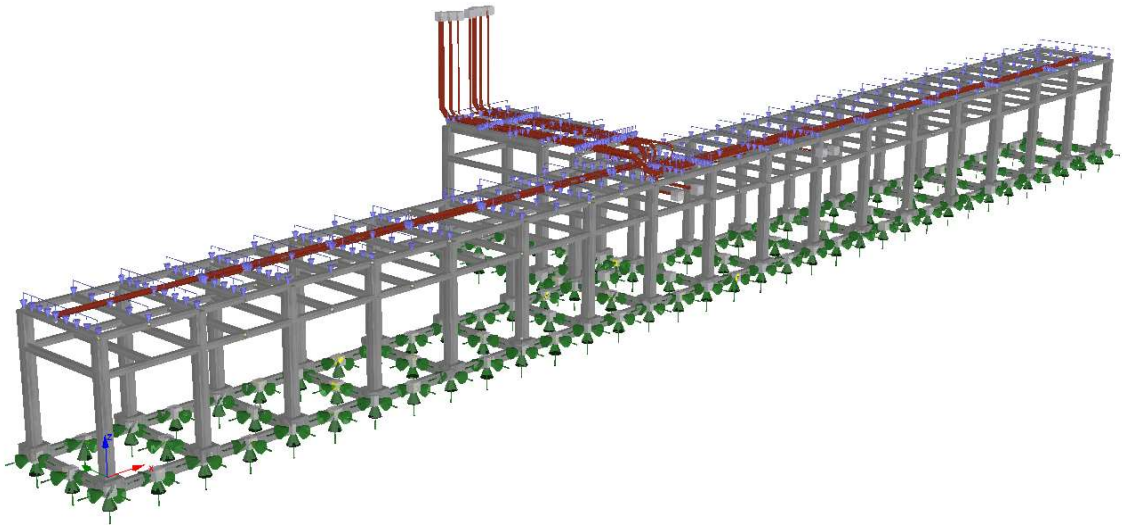

Figure 7: The soil-foundation, pipe rack and pipelines in the same model on Seismostruct ([27])

The original G-γ-D curves of the soil type, which rely on the formulae of [28] are shown in Figure 5. As mentioned in the section 3, the soil nonlinearity is described by calibrating the RO model included in the toolset of [27] for the STC. For that purpose, the unknown parameters of the model are estimated by building up a code in MATLAB ([34]) and the yielded G-γ-D curves are shown in Figure 8a&b for two fitting methods, namely the Root Mean Square Error (RMSE) and the coefficient of determination (R^2). The former was found to calibrate better the G-γ curve and thus the parameters of that fitting are adopted.

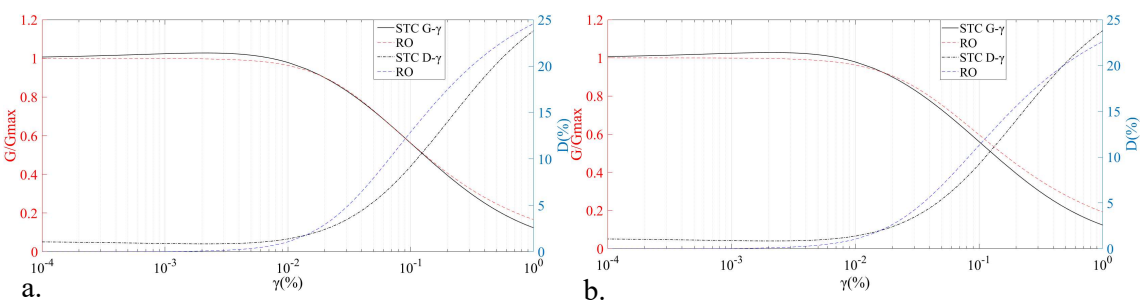

Figure 8: Calibration of RO model for the STC using a. the RMSE and b. R^2 fitting methods

4.2 Seismic assessment

Initially, a modal analysis is conducted for the pipe rack shown in Figure 7 with (W/) and without (W/O) SSI to identify the fundamental modes, which are quoted in Table 2. The maximum increase of the period fluctuated between 17 and 20% for the two modes. To select the seismic records for the fragility assessment of the rack, it is necessary to identify the fundamental period along the weakest direction. This is achieved by means of pushover analysis both for uniform and 1st mode load distribution as well as for the two modes shown in Table 2. The

capacity curves are illustrated in Figure 9 where it is obvious that the principal mode in the Y direction (T_3=0.276 sec) is the weakest one.

	$T_{W/O}$ (sec)	$T_{W/}$ (sec)	ΔT (%)	$M_{W/O}$ (%)	$M_{W/}$ (%)
Mode 3(Y)	0.269	0.315	17	43	34
Mode 5 (X)	0.230	0.276	20	87	55

Table 2: The two fundamental modes of the pipe rack

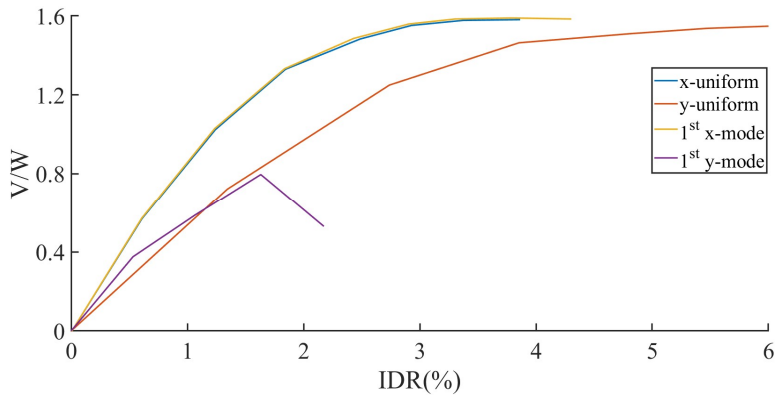

Figure 9: The capacity curves of the first mode and uniform load pattern distribution

Furthermore, 7 compatible records are selected through REXEL software ([35]) in the [0.2 T_3, 2T_3] period range. The spectrum that the compatibility is achieved for refers to SLV limit state, STC and Usage Class III ([36]), although three LSs, namely Serviceability (SLS), Safe Life Limit State (SLLS) and Collapse (CLS) are accounted for the assessment in the following. This topic is under investigation by the Authors since the best option could be records compatibility for each LS. It is also noteworthy that the compatibility of records is difficult to be attained both for the two horizontal and one vertical component, thus, the records shown in Figure 8 have compatibility for the two horizontal ones at least (the vertical component is used in all cases). The epicentral distance of the seismic records is greater than 15 kms (far-field, [37]) and the duration of the records selected for the analysis is bracketed with a_g>0.05 g.

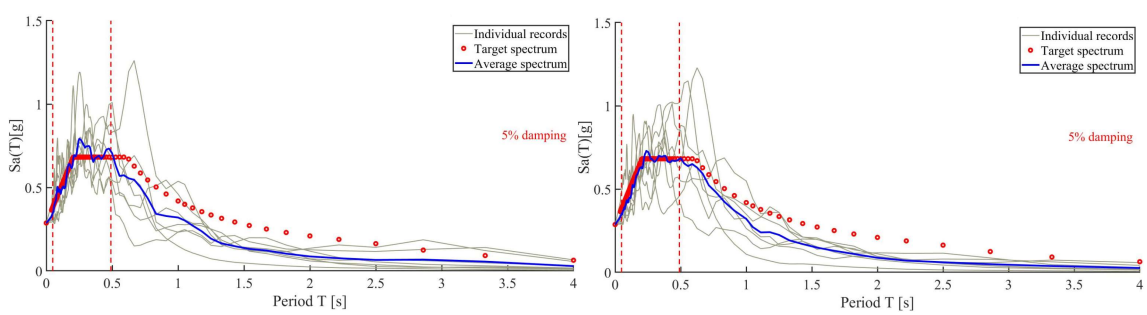

Figure 10: The spectrum-compatible seismic records in far-field conditions for the a. X and b. Y component

4.3 Derivation of fragility functions

Numerous methodologies exist in the literature for the evaluation of fragility functions. A common methodology that proposes the successive scaling of records till structural collapse onset, viz Incremental Dynamic Analysis (IDA, [38]), is adopted in the framework of this study.

The method has the competitive edge against other methodologies such as Multi-Stripes Analysis (MSA) or Cloud Analysis that it can be used even if a limited number of spectrum compatible records can be found for a particular site accounting also for the constraint in the epicentral distance, whereas other methodologies need a considerably greater number ([39]). The constraint of limited number of records is also in connection with the model scale. Before assessing the structural damage, the LSs should be determined. Only the structural members are assessed herein, whereas the nonstructural components will be presented in future publications.

Accounting for the fact that an LNG pipe rack is a critical structure that carries nonstructural components, a multi-scale approach should be considered by discriminating the collapse in local scale e.g. pipelines, meso-scale (structural members) and global scale in order to be utilized in the future within a quantitative risk framework. In particular, the Engineering Demand Parameter (EDP) that is adopted for the meso-scale is the shear force V_E, since it is related to the response of individual macroelement whilst the chord rotation θ_E or IDR can describe the global collapse (joint of more than one element, Table 3). More information about the EDPs of structural components can be found in [40].

Mechanism	Serviceability LS (SLS)	Safe Life LS (SLLS)	Collapse LS (CLS)
Flexure (rad)	$\theta_E \leq \theta_y$	$\theta_E \leq \theta_y$	$\theta_E \leq \theta_{u,\, m\text{-}\sigma}$
Shear (kN)	$V_E \leq V_{Rd.EC2}$	$V_E \leq 0.75 \cdot V_{Rd.EC8}$	$V_E \leq V_{Rd.EC8}$

Table 3: Limit states of concrete members in meso- and global-scale ([40])

The fragility functions of columns and beams are shown in Figures 11&12 only for the meso-scale investigation since the failure under bending was not found to be predominant. The SSI affects detrimentally the response of structural components both in linear and nonlinear case. An interesting result pertains to the dispersion of damage when the two cases are compared. The soil nonlinearity yields much lower dispersion (Table 4) and that might be due to the independence of structural damage vis-à-vis the modelling parameters caused by the energy dissipation of soil. We mention that the probability of exceedance of columns CLS for PGA=0.6g in the decoupled case W/ and W/O SSI soars by 43% and 81% for the linear and nonlinear soil, respectively. The pertinent values for the beams are equal to 55% and 270%.

When it comes to decoupled and coupled case, it is shown that the vulnerability of the pipe rack goes up further, however, the tendency is much higher for the beams since they become considerably fragile. The comparison of beams and columns response W/O SSI deduces that the beams fail earlier for SLS, however, the columns get more fragile for the two consecutive LSs. However, this is not the case for the coupled case for which the beams fail in advance of columns for all LSs. The fragility of beams for the CLS W/O SSI and PGA=0.6g increases by 455% from the decoupled to coupled case. The pertinent raise for the columns is only 40%. This response is justified due to the immediate dynamic interaction of beams with the pipelines. Also, noteworthy is that the pipe rack was initially designed for a low-seismicity area and that make the support of pipes totally inappropriate for the seismicity of the new placement. Also, this proves our statement in the introduction that the type of BCs is considerable in the design of the racks and should be taken into account by the codes in a clear manner considering that only the weight and the rigidity of supported equipment is presently prescribed to be accounted for the analysis.

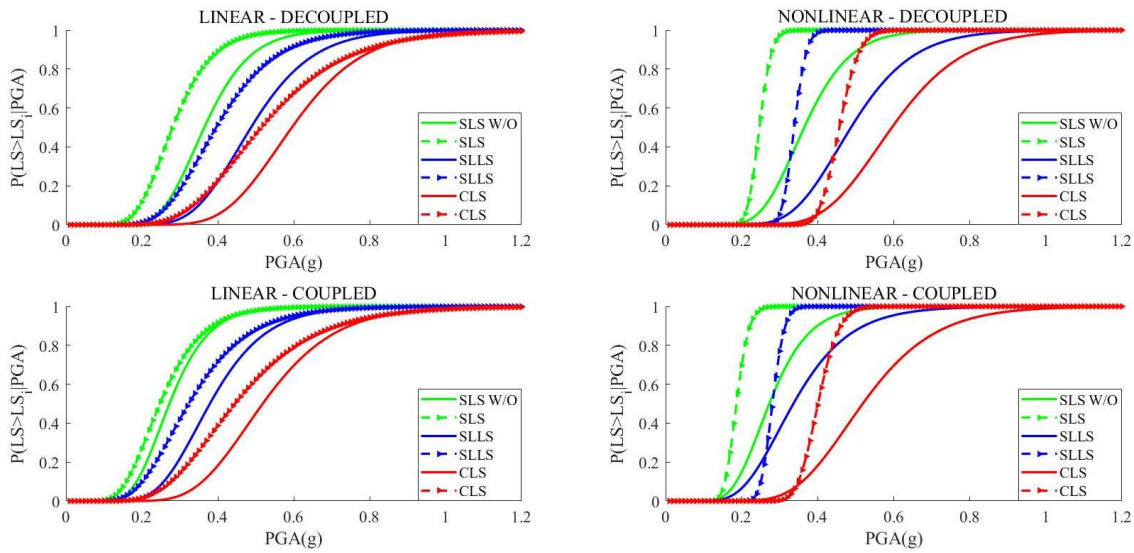

Figure 11: Fragility function for columns

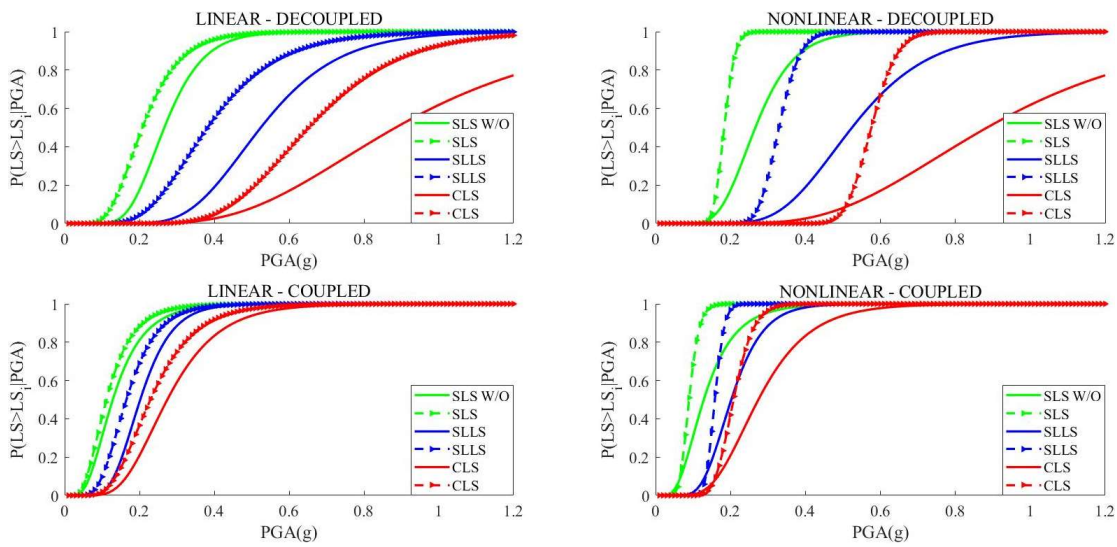

Figure 12: Fragility function for beams

Decoupled	W/O SSI			W/ SSI - Linear			W/ SSI - Nonlinear		
	SLS	SLLS	CLS	SLS	SLLS	CLS	SLS	SLLS	CLS
Columns	0.26	0.26	0.24	0.29	0.30	0.34	0.10	0.07	0.09
Beams	0.31	0.31	0.41	0.37	0.38	0.32	0.12	0.13	0.09

Table 4: Dispersion of damages for columns in decoupled case

5 CONCLUSIONS

In the present analytical paper, a two-fold purpose was addressed; first the investigation of dynamic interaction between a number of pipelines with the supporting structure, and secondly,

the effects of soil-structure interaction on structural components response. The initial review of codes and technical literature showed that there is a gap of information of code provisions upon the parameters to be considered for the analysis methodology selection. The lack of code prescriptions was confirmed by the case-study, since it was shown by means of fragility curves the tremendous impact the pipeline might have on structural components and beams specifically. The rise on beams fragility was greater than 400% for the ultimate limit state without considering the effects of soil-structure interaction. When the last interaction is considered as well, the fragility of pipe rack increases further over than 50% and 250% for linear and nonlinear soil, respectively. It was also demonstrated that the soil nonlinearity or the higher energy dissipation might constitute the response of the rack independent of modelling and therefore reduce the dispersion of damage. Finally, both beams and columns failure was observed due to shear (meso-scale).

The investigation of pipes fragility and correlation of stress-strain values with LSs of the structural components is considered essential in order to confirm that common LSs cannot apply for nonbuilding structures like the present LNG rack. Also, another important parameter in the design process will be the evaluation of peak floor acceleration that is usually proposed by the codes when nonstructural components exist. Finally, records in near-field conditions as well as compatibility for the vertical component should be considered as well to examine possible variations of the pipe rack-pipes damage. The research on the aforementioned topics is still under investigation by the Authors.

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