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INVESTIGATION OF THE SEISMIC RISK OF INDUSTRIAL PIPE RACK – PIPING SYSTEMS ACCOUNTING FOR SOIL-STRUCTURE INTERACTION

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ABSTRACT

Earthquake events have shown that industrial pipe racks lack of a completed design framework that encompasses contemporarily a number of uncertainties such as modelling, seismic action, design and analysis procedures as well as soil conditions. That being said, the seismic behaviour of piping systems has not been assessed up to par recognizing the potential effects of nonbuilding – nonstructural components interaction as well as soil conditions that constitute a decisive parameter particularly for structures that lie on alluvial deposits. In the present work, after reviewing European and American standards and technical literature upon design parameters, the seismic reliability analysis of two pipe rack – piping systems in decoupled and coupled case considering near- and far-field records as well as soil deformability is addressed. As it is illustrated, the classic nonlinear static analysis may overestimate the resistance of racks, common limit states of interstorey drift ratio cannot be applied and the behaviour factor selection may be unjustifiable. Also, soil-structure interaction affects detrimentally the response both of rack and piping system as depicted by the fragility functions.

Keywords: piping system; pipe rack; modelling; dynamic interaction; soil-structure-interaction (SSI); fragility curves.

INTRODUCTION

Mid- and down-stream facilities constitute indispensable links of energy supply network where the gas or oil is temporarily stored, converted or processed in order to reach the market afterwards. The soaring energy demands along with the increasing population density forces authorities to give due consideration in the safe construction of oil and gas process plants since the smooth operation in there signifies the safety and financial robustness of communities at regional and supra-regional level. Also, the resilience of those plants was, is and will be a top priority issue on the agenda of societies given the high repercussions a Natural Technological (NaTech) -which are increasing as a result of climate changes- or specifically in this study a seismic event might cause. Industrial accidents within units that comprise steel or concrete pipe racks and complex piping systems, among others, occur rather frequently. There are ample examples of incidents that the interested reader can find on the news even on a weekly basis pertaining to failures of piping complex units. Even though incidents were mainly related to human and organizational errors according to the inquiry of [1] upon 364 chemical process industry accidents, failures were also attributed to the layout and fabrication of the piping and supporting structure system. That being said, nonbuilding structures similar to buildings e.g. pipe racks as well as nonstructural components by which the racks are outfitted constitute a vulnerable to seismic hazard system. Pipe racks constitute primarily steel structures in mid- and down-stream facilities such as Liquified Natural Gas (LNG) terminal and Petrochemical Plant (PP), respectively, and support pipes that transfer hazardous materials from one unit to another. The construction of seismic-resistant pipe racks is evident taking into consideration that several oil refineries are located in high seismic-prone countries e.g. Italy, Greece, Turkey, USA, Canada, China, Taiwan, Japan and many others.

There are numerous key parameters when modelling, designing and assessing a pipe rack – piping system that increase considerably the risk due to uncertainties included in; the type of pipe elements, the dynamic interaction of nonbuilding structure – nonstructural components and the boundary conditions of pipes, to name just a few. The European (EN) codes present clear paucity of information in many of the aforementioned parameters, whereas the American (AM) ones deal with the design of system in a more comprehensive manner, however, the structural peculiarity of pipe racks along with attached complex piping systems could make the adoption of e.g. behaviour factor or analysis methodology questionable and unjustifiable. Although, the matter is rather crucial for the safe design of industrial facilities, the technical literature is rather limited. Both codes and bibliography are reviewed in the following section.

Practicing and professional engineers should design pipe racks dealing with challenges that come from different engineering disciplines. For instance, another touchstone of pipe rack – piping system design refers to the effects of soil deformability. The majority of oil and gas plants are placed close to seacoast where the soil is quite weak e.g. liquefiable sand and if pipe racks or tanks are massive and exhibit stiff response, the effects of SSI phenomenon on the seismic response could be substantial as it has been proved for common-building structures.

Scope

The present paper aims mainly at shedding light on design parameters that characterise the type of structures in-hand and demonstrating the effects of soil on pipe rack and pipelines fragility. For these purposes, two case-studies are addressed; the first pertain to a steel petrochemical plant pipe rack that is designed and assessed in the way of estimating the behaviour factor. The second one refers to a Reinforced Concrete (RC) pipe rack that is assessed for estimating the fragility curves accounting for SSI. Except for the two main goals, attention is also given to the review of current EN and AM seismic codes as well as technical literature applicable to pipe racks so as possible inconsistencies might be found and estimation of Interstorey Drift Ratio (IDR) values in relation to pipe stress. The work presented herein does not aim at covering every aspect involved in the seismic design of process plant pipe racks yet to form the starting point for further research and recommendations.

REVIEW OF CODES AND LITERATURE

Modelling and analysis of pipe racks

The structural type to be used for modelling pipe racks depends on the single case and should always be compatible with design code provisions; for instance, the use of modular pipe racks is extensive in oil and gas industry due to financial reasons and the type of connections is strongly related with the distribution of pipes and/or vessels weight as well as the fabrication cost. Pinned connections are typically utilized for beams in the longitudinal direction (struts) and shear tabs for the transverse (bent) beams. Also, the type and location of bracing is essential for the transportation, lifting and support of permanent and operating loads. As an example, vertical bracing could be used to support side overhang cantilevers that are necessary for pipelines running out of the main pipe rack frame [2].

Modelling and seismic analysis issues that pertain to the interaction between piping system and pipe rack are crucial for these types of structures and could change significantly the global seismic response. For instance, when horizontal pressure vessels are supported on the rack, it is rather possible torsional effects to be present depending on the tank mass, the position and the frame section profiles. Furthermore, the idiosyncrasy of piping systems response refers to the way, rigid or flexible, that they are attached on the rack. For example, a number of parametric analyses on a piping system and its supporting structure was conducted in [3] by considering different configurations such as number of supported pipes or different diameter of pipes, end conditions and diameter of link connections (U rings). The authors concluded that the frequency of the system can significantly be affected by the type of link elements (diameter of links). At least to the Authors' knowledge, there are very few research efforts that undertake the seismic design of pipe racks ([2], [4], [5]) and even these publications not in a comprehensive manner since parameters e.g. behaviour factor selection, nonbuilding structure – nonstructural components interaction and analysis methodologies or soil effects are not dealt with sufficiently. Also, the limited research on the dynamic interaction proves that the assessment of structures included in process plants have been of concern at a component level without considering the system of pipe rack – pipework as a whole.

When it comes to codes, the main EN contribution for seismic design issues ([6]) does not make reference to seismic design requirements of industrial structures yet only to irregular, which differentiate in many ways compared to the petrochemical pipe racks. On the contrary, American code [7] or the guideline [8] (the latter makes reference specifically to petrochemical plants) stipulate seismic design criteria and analysis methodologies for steel pipe racks in particular based upon the rigidity of the connection and the weight of each system. The AM codes stipulate that if the non-building structure similar to building and nonstructural components weight W_p is less than 25% of the weight of the entire system W_t, then, the interaction could be neglected, and each structure could be designed and analysed separately. On the contrary, if the supported system weighs more than 25%, then, the coupled system should be considered either by considering the nonbuilding structure only as a rigid element with appropriate distribution of each seismic weight (rigid response with $T<0.06s$) or modelling the whole system in the same model (flexible response with $T>0.06s$). This statement by the code gives mainly attention to the weight ratio and rigidity of individual supported nonbuilding structures without considering the case of multiple-secondary components e.g. pipelines ([3]) for which the relative stiffness of connection (ring to pipe diameter) as well as the end-conditions may affect the frequency of the system.

Soil-structure interaction

The soil deformability is probably the most considerable parameter during the oil refinery structures design process given that many plants are located in high seismicity regions and at coastal sites with liquefiable soils. Considering the effects of SSI as they have been proved for common building structures, the soil deformability can affect either detrimentally or beneficially the seismic response depending on structural types and soil characteristics. Soil has higher impact on stiff and massive structures and lower the force demand (beneficial effect) ([9]); however, there are cases for which the response spectra of recorded ground motions can cause higher demand for longer periods. The primary factor that alters the structural period and damping is the ratio $h/(V_s \cdot T)$, where h is the effective structural height, V_s is the shear wave velocity and T is the fixed-base natural period ([10]).

The SSI effects on process plant structures has not received the required attention by researchers so far, although the cruciality of structures involved in process plants, the complexity of pipe rack – piping system interaction, the harsh environmental conditions and the intricacy of SSI phenomenon demand the assessment of SSI should always be conducted so as to give a better insight of soil effects on the seismic response of structural and nonstructural components. There are cases where pipe racks are massive and stiff due to the supported nonbuilding not similar to building structures and nonstructural components e.g. elevated tanks or complex pipework, and as a result the SSI may affect the response and cause fracture of the most vulnerable and flexible apparatuses.

It is considered essential the assessment of oil refinery pipe racks to account contemporarily for as much uncertainties as possible including those that refer to soil towards minimizing the seismic risk. We mention herein that the SSI effects on industrial structures e.g. pipe racks or tanks have not included in a fragility analysis framework as it has been done partially for common-building structures or bridges ([11], [12] & [13] among others). This framework is necessary in order to highlight the response of nonstructural components e.g. pipes in relation to structural ones accounting for the effects of SSI. Fragility Functions (FFs) is an essential tool particularly for industrial structures; primarily, the most vulnerable components can been highlighted towards taking strengthening measures for existing or under-design structures, and secondly, mitigation and emergency response plans can be adopted in the aftermath of earthquake events [10]. In the second Case-Study (CS2) that follows, an attempt is made to evaluate probabilistically the effects of SSI on rack-piping system response by deriving FFs both for structural and nonstructural components.

With regard to code provisions, the two main EN contributions on the design of geotechnical structures [14] and [15] may encourage the incorporation of SSI into the analysis model, however, they fail to state as much practically as the AM [12] does the modifications on the design process when the soil deformability is accounted for. The last code specifies within a chapter along with a commentary one modifications e.g. on design base shear value during static analysis or the site-specific response spectrum during a nonlinear response analysis. In particular, both modifications rely on the factor B_{SSI} that takes the foundation and structural damping into account based upon the period lengthening.

Behaviour factor

Following the design process of nowadays, it is known a priori that structures are going to experience damage due to middle-to-severe earthquake events. By introducing a behaviour or reduction factor, the elastic spectrum acceleration decreases, and structures are designed to withstand lower forces than probably those will experience during their reference life. Consequently, it is up to the structural engineer to decide the type, location and extent of damage based upon the risk that a structure exhibits to human life and environment [10]; this is also called dissipative design approach. The global inelastic response is not to be confused with the local one. The structural design always relies on the reduction factor (global response), whereas the method of assessing the local response has changed the last decades with the introduction of Limit States Design (LSD).

The behaviour factor (q-factor) or response modification factor (R-factor) as named in EN and AM codes, respectively, is given as the product of ductility and overstrength. The former parameter is a function of members detailing, whereas the second refers to the redundancy that a structure reserves. Depending on the energy dissipation capacity of structural systems, which is defined as low, moderate and high, both concrete and steel frames are categorized in AM code [7] as ordinary, intermediate and special, respectively. The value of q-factor or R-factor varies in codes due to primarily the structural material, concrete or steel, and secondly the different structural type, namely braced frames e.g. Concentrically Braced Frame (CBF), Moment Resisting Frames (MRFs) or dual systems due to the various performance levels to be achieved, the detailing of connections, the number of stories, the soil conditions or the seismic design method considered. Several attempts have been made to assess the value of behaviour factor of common steel structures ([16], [17]). Through these investigations many aspects upon the correct estimation of structural response quality factor have been clarified such as the limitation of the traditional pushover analysis to account for higher mode effects and member stiffness changes [18]. As it is demonstrated in the exact following Case-Study (CS), the higher mode contribution could be substantial when nonbuilding structures are under examination.

EN codes do not make reference to pipe racks yet only to irregular steel structures that differentiate from the racks due to loading type and operational purposes. To make feasible the comparison of the factor prescribed in two codes for steel pipe racks -the concrete is omitted for brevity-, it is assumed that the irregular structures mentioned in the EN code [6] could represent petrochemical steel pipe racks given that they are usually irregular. In any case, this assumption cannot confirm that the values are proper for steel pipe racks.

Table 1. q-factors for pipe racks as specified in [6] & [7].

Structural type		Ordinary	Intermediate	Special
EN	MRF	-	3.20	$4.00a_u/a_1$
	CBF	-	3.20	3.20
AM	MRF	2.33	3.00	5.33
	CBF	2.17	-	4.00

a_u/a_1: overstrength ratio as specified in [6]

The values of q-factor as prescribed in EN code (Table 1) come after a reduction by 20% of the relevant values for regular structures as the code specifies. The overstrength ratio a_u/a_1 is introduced for High Ductility Class (DCH). The value of overstrength ratio varies between 1.1 and 1.3 for MRF and is equal to 1.1 for CBF. Greater values of the overstrength ratio can be adopted in case nonlinear pushover analysis is used, however, the value cannot be higher than 1.6. Also, the values proposed in the AM code (Table 1) refer to the Maximum Considered

Earthquake (MCE, probability of collapse 2% in 50 years), and therefore, they have decreased by 67% to refer to the design earthquake. The values of reduction factors are comparable being greater or lower in one code compared to the other. The AM code does not specify values of R-factor for intermediate ductility CBFs. Also, it is worth mentioning that, in contrast with the EN code, the AM one proposes R-factor for ordinary pipe racks; this is reasonable considering the low ductility demand for this type of structure and may be an indication that the values by the EN code cannot be adopted for pipe racks with high confidence. The R-factor values shown in Table 1 do not consider increase of pipe rack height, since it is not considered in EN1998-1 (2004), however, they do account for the overstrength factor included in the pertinent table of AM code, which is equal to 3 and 2 for MRF and CBF, respectively.

Although common-building structures are designed for earthquake events with recurrence period of 475 years (or probability of occurrence 10% in 50 years), it is possible to experience earthquake with higher recurrence period. To keep the structures safe enough, codes propose lower values of q-factor than the real product of ductility and overstrength; this is the reason the analytical calculations of [19] yielded values of q-factor up to 6 times the one proposed by the codes for MRF. When it comes to structures of higher importance e.g. oil refinery pipe racks, the values prescribed by the codes could be non-risk related and thus unjustifiable. Recently, a new risk-targeted design method has been introduced ([19], [20]) that accounts for different limit states in order to take the seismic risk in a more justifiable way into account. The method introduces a risk-targeted safety factor (γ_{im}), and the behaviour factor that accounts for risk is given by (eq. 1 has been included in the new generation of EN codes):

$$q = r_s \cdot r_\mu \cdot C_p \qquad (1)$$

where r_s is the overstrength factor, r_μ is the ductility factor and C_p pertains to the correction factor due to the risk-targeted definition being equal to the inverse of safety factor (γ_{im}). The last factor is equal to the ratio of seismic intensity corresponding to a designated return period (e.g. T_{LR}=713 yrs in the CS1) and the mean value of seismic intensity that causes collapse; although, the first value can be found in seismic hazard maps, the second one is calculated numerically from the risk-equation, which holds as follows:

$$P_c \approx \lambda_C = \int_0^\infty P(C|S = S_a) \cdot \left|\frac{dH(S_a)}{dS_a}\right| \cdot dS_a \qquad (2)$$

P_c is the probability of collapse, λ_C is the mean annual frequency of collapse, P(C|S=Sa) is the fragility function of collapse given the random variable S (e.g. spectral acceleration at the fundamental period) and $H(S_a)$ is the hazard function which describes the annual rate of exceedance of S_a. To estimate the mean value of S_a that causes collapse, a common value of P_c equal to $2 \cdot 10^{-4}$ as considered by building codes and a generally accepted value of fragility curve dispersion (β=0.4) are adopted. The risk-targeted factor is estimated in the following CS.

CASE-STUDY #1

The present case-study concerns the design, analysis and assessment of a three-floor petrochemical plant steel pipe rack (Figure 1). The rack is 12 m high and is outfitted with a piping system (Figure 1a), which runs along the length and the height of the third floor. The pipe rack is made of different H- and I-shaped cross-sections as well as circular or rectangular concentric bracing (X-crossing or inverted V) in vertical and horizontal direction. The construction material is elasto-plastic steel grade S275 with strain hardening. The piping system constitutes 8" and 6" (Nominal Pipe Size, NPS) pipes with nominal yield and ultimate material strength 418 and 554 MPa, respectively and two horizontal vessels of 2 cm thickness. It is assumed that the piping system transfer a hazardous but not toxic material (Directive 67/548/EEC, 2004), namely propylene, with unit weight γ=5.42 kN/m^3 ([21]) at zero internal pressure in order to stay on the safe side (more information can be found in [22]). To account for the liquid, the density of pipes and tank material has increased.

The piping system includes nine (9) elbows, one T-joint and two nozzles (Figure 1b). It is common in oil refinery industry, horizontal expansion loops of pipes to be formed in order to minimize the internal pressure and this is the reason that pipes run out of the main frame of the rack [2]. Also, it is a common industry practice, the location of pipe supports e.g. anchors to be clearly shown on isometric drawings and provided to structural engineers prior to pipe rack layout configuration, thus, more information on the geometry and the configuration of the pipe rack and piping system can be found in [23]. Finally, engineers adopt a uniform load on pipe rack beams, which are referred as bents and struts in the transverse and longitudinal direction, respectively, to account for small pipes and future installation, which practice has been adopted herein by considering 4.5 kN/m (it refers 1.5 times the maximum concentrated load on the rack by the existing piping system).

Modelling and design

Two types of elements can be used for modelling a piping system, namely shell and beam elements. The main EN contribution [24] and the AM code [25] make reference only to beam elements. The primary difficulty when modelling pipes with beams elements concerns the pipe bend (elbow). To take the higher flexibility of this critical component into account, the thickness of a straight beam that substitutes the curved pipe decreases according to a rough code-related rule as specified in [23] and [24]. Also, the seismic codes propose conservatively the use of a stress intensification factor (i), which depends on the seismic level considered, namely design or safe shutdown. There is also another way of modelling the pipe bend as proposed by [22] that has been assessed herein by using beam elements. The method (known as 'Equivalent Straight Elbow') is based on analytical calculations and try to form a straight pipe that has the same mechanical characteristics with the flexible pipe bend by using the Euler-Bernoulli theory. In doing so, the original elbow is subjected into axial, shear and bending moment so as the individual stiffnesses to be found which are set equal to the one of a straight beam. In fact, the only parameter that changes in the end is the pipe thickness of the straight element. The reader who is interested in this methodology can find more details in [22]. Of course, it is pointed out that the beam elements proposed by the codes are not accurate and definitely inappropriate to capture the nonlinear deformation of the pipe (the so-called ovalisation phenomenon), however, neither EN nor AM codes deal with shell elements.

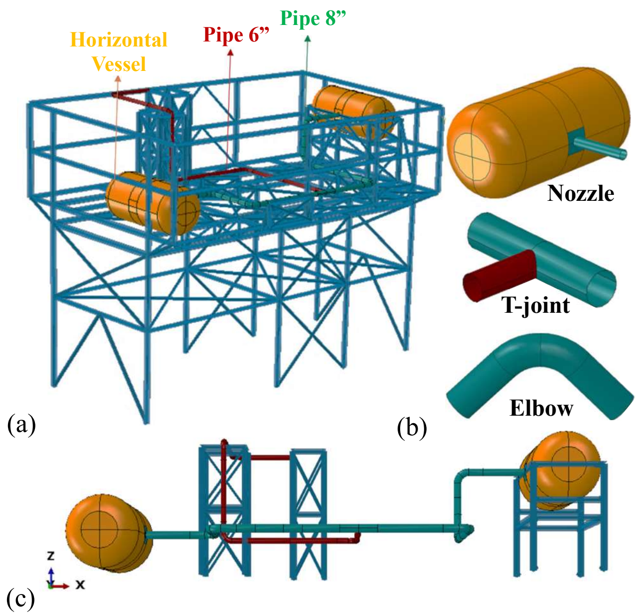


Figure 1. The three-floor petrochemical plant steel pipe rack in CS #1 (a) the overall, (b) the critical components of the piping system, and (c) piping system layout.

Beam elements may not be capable of capturing the exact dynamic characteristics that can be found by shell elements or experimental tests. A modal analysis has been conducted on ABAQUS software [26] for the pipeline shown in Figure 1a&c by using shell elements. As it is illustrated in the first row of Table 2, the beam elements that have been examined in [22] lose the first fundamental frequency of the pipeline, whereas the results from the experimental [27] and ABAQUS analyses seem quite similar. Given the consistency found with experimental tests as well as the small scale of piping system, shell elements will be used exclusively for the present CS.

Table 2. Evaluation of frequency (in Hertz) of the piping system.

Beam elements (1)	Experimental Tests (2)	Shell Elements (3)	Variation (1) *vs.* (3)	Variation (2) *vs.* (3)
-	3.70	3.47	N.A.	6.6%
6.50	6.40	6.56	0.9%	2.4%
7.14	Not given	7.31	2.3%	N.A.*
8.22	Not given	8.21	0.1%	N.A.

Furthermore, the pipe rack – piping system response will be examined in the coupled case since W_p>25% (W_p=28.58 tonnes and W_t=54.91 tonnes) and T>0.06. It should be mentioned that the analysis for estimating the fundamental frequency of the piping system (vessels and pipes) includes the towers on which are supported on as specified in [7] since the elevated mass of tanks and pipes cannot be considered rigidly supported on the rack (Figure 1c). Thus, piping system during seismic analysis constitutes not only the pipes and the vessels but also the supports that the piping system is attached to.

As discussed previously, the structural type to be adopted for pipe racks may vary due to the different configuration of nonbuilding structures – nonstructural components. In the present CS, the petrochemical steel pipe rack is considered as an Ordinary Concentrically Braced Frame (OCBF) with horizontal and vertical bracing and is modelled on ABAQUS software. The hazardous material along with the flexibility of nonstructural components make the assumption of low ductility class (ordinary structure) necessary. The rack is placed in a high seismic-prone area in the north-eastern part of Sicily (near Milazzo city), in Southern Italy, where an oil refinery is located, and designed according to the Italian [28] and the European code [6] for the Safe Life Limit State (SLLS) with recurrence period equal to 712 years (or probability of exceedance 7% within 50 years) as per [28]. The design parameters can be found in Table 3.

Table 3. Design parameters for the pipe rack [28].

Location	Coastal site near Milazzo, Sicily
Soil	C
q-factor	2.17
Importance Class	III (Essential facility)
PGA	0.16g
Recurrence Period, T	712 yrs

The piping system is designed according to the stress-based approach or Allowable Stress Method (ASM) as specified in [24] and [25] and analysed taking both inertial effects and differential movements of the supports into account (coupled case). It is emphasised that in case of decoupled system, such case is examined in CS2, only the inertia effects can be evaluated. Furthermore, the [24] code makes reference to two seismic levels, viz Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE), whereas the latter one only to OBE (or occasional loads as they are defined in the code). It is pointed out that the [7] proposes additional seismic acceptance criteria for nonstructural components e.g. allowable peak spectral acceleration at attachment points or relative displacement between attachment points that are not included in [24] and [25]. The latter codes pertain mainly to the design of pipelines itself without considering dynamic interaction with attachment structures. Following this design methodology, other acceptance criteria could be assessed in order to enhance the design methodology in future publications. Also, it should be mentioned that the lower q-factor value between the piping system and the supporting structure is adopted, since the coupled case is considered. This assumption and the reduction factor come after the [7] (see also Table 1), considering that EN code still does not specify values of q-factor for pipe racks.

Finally, a modal analysis has also been conducted and the first two fundamental mode shapes excite the 42% and 26% of the total mass of the structure in the Y- and X-direction (Table 4), respectively. In contrast with the common building structures, the highest modal participating mass ratio is observed at higher modes e.g. 6th mode, and that makes the use of common design and assessment methodologies questionable.

Table 4. Modal analysis of steel pipe rack.

Mode	Time period (s)	Modal mass (%)
2nd	0.44	42
6th	0.31	26

Assessment of steel rack

The high irregularity of the pipe rack and high uncertainty of the effects of pipe rack-piping system interaction makes the evaluation of system performance factor (q) necessary. In this way, the reliability of the reduction factor proposed by the codes can be assessed and the parameters that affect the seismic response can be highlighted. In the framework of the CS1, the classic nonlinear analysis (Pushover Analysis, PA) is primarily adopted to estimate the behaviour factor of the pipe rack considering uniform loading distribution. The response is monitored at different control points along the perimeter of the third floor of the pipe rack to examine possible variation of the seismic behaviour since the rack is expected to have high torsional effects due to the support of tanks on the third floor; however, in the following only the worst case -point with the highest IDR- is presented, though. Given the structural type adopted, the substantial reduction of rack lateral resistance considering the force-deformation curve is adopted as global collapse limit state for each direction [29]. Also, the criterion used to define the global yield threshold, which is necessary for the behaviour factor estimation, is selected as the yield displacement at 75% of the maximum strength of the original force-displacement curve compared to the equivalent elasto-plastic system. The ductility factor is estimated by using the equal displacement method. More information about the ductility and behaviour factor estimation can be found in [10].

To assess the previous analysis methodology for pipe racks, the Incremental Dynamic Analysis (IDA, [30]) is also adopted. A suite of 7 spectrum compatible records that refer to near-fault conditions (epicentral distance<15kms, [31] &[32]) are used to excite the rack. Since the compatibility is difficult to be attained in the two horizontal and one vertical component simultaneously, the records are compatible only for the two horizontal components. To be consistent with the pushover analysis, the same criterion is adopted for defining global collapse and yielding of the system. The seismic records are scaled up to collapse based upon the maximum Peak Ground Acceleration (PGA) in the two horizontal directions, however, the vertical component is scaled as well in order to keep the V/H ratio constant [29].

According to the results shown in Figure 2, the PA overestimates the lateral resistance in both directions. The overestimation reaches up to 2 times at the maximum IDR value in the Y-direction. Except for the irregularity of the rack by itself, the PA does not activate the tank mass as the dynamic analysis does and this may be the reason of the high inconsistency observed by the two methodologies. For instance, the ductility factor occurred almost 40% higher by PA in the Y direction (Table 5). Considering the type of vessel (horizontal vessel running in the Y direction), the great mass that is concentrated in a narrow range at the edge of the rack and the lack of slab to support the weight in a uniform manner, torsional effects may be created affecting the ductility in one direction compared to the other. The pipe rack is considerably more ductile in the Y- than in the X-direction; with regard to the results from IDA, the ductility factor occurred by 45% higher in the former direction.

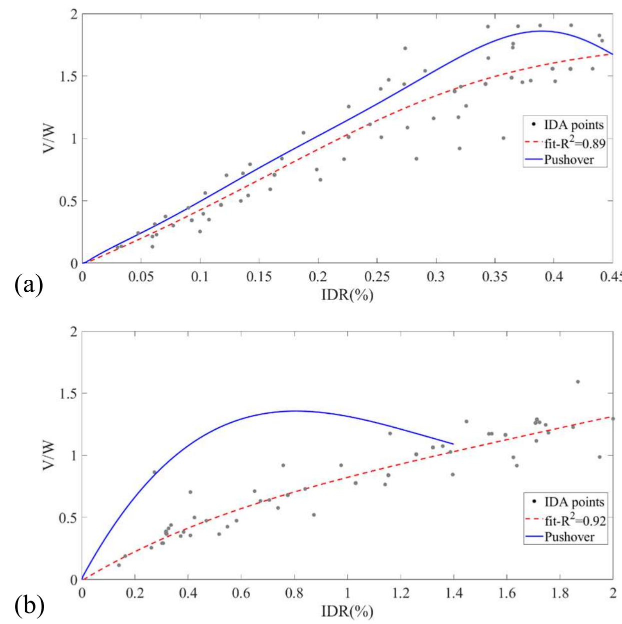


Figure 2. The capacity curves for (a) X-direction and (b) Y-direction and two analyses methods, namely pushover and time-history or IDA

As it was expected, the product of ductility and overstrength factor is extremely high. When the results of IDA are considered, the factor fluctuates from 9 to 15 in both directions. To estimate the factor accounting for the designated risk, the risk-equation (eq. 2) is used by assuming probability of collapse (P_c) equal to $2{\cdot}10^{-4}$, which corresponds to a generally accepted value that has been adopted in the development of building codes and standard deviation of the fragility function equal to β=0.4, which is also a generally accepted value in the literature for code-conforming buildings [33]. Also, REASSES software [34] is used for deriving the hazard curve (Figure 3) at the site under consideration. The results of risk-targeted factor are shown in Table 5 in parentheses. Considering only the values of factor by IDA, the factor occurred by 29% less than the design value in the X-direction and 16% higher in the other direction. The inconsistency of the factor with the design value may indicates that the factors proposed by the code are not always on the safe side and surely unjustifiable.

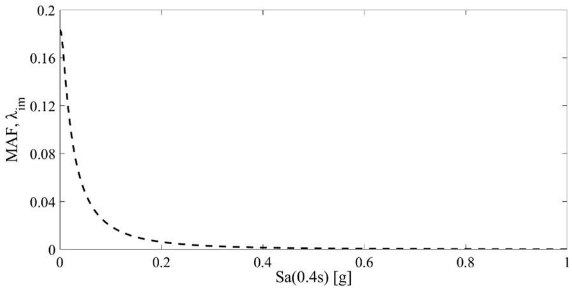


Figure 3. Hazard curve at Milazzo seacoast zone, Sicily (MAF: Mean Annual Frequency of exceedance)

Table 5. Behaviour factors for the pipe rack

	Ductility factor		Behaviour factor	
	X-dir	Y-dir	X-dir	Y-dir
Pushover	1.70	3.76	10.6(1.82)	15.2(2.61)
IDA	1.85	2.68	9.04(1.55)	14.7(2.53)
%	+8.9	-28.7	-14.7	-3.3

Table 6. The coupling response of nonbuilding – nonstructural components for the 3 out of 7 worst time-histories in terms of stress, IDR and PFA.

E(ID) (OBE, SSE)	Von Mises (MPa)	IDR(%) (X, Y)	PFA
EQ1	352 (T) 350 (T)	0.09/0.33 0.10/0.38	0.93g 0.68g
EQ2	372 (N*) 384 (N)	0.02/0.52 0.12/0.41	0.81g 0.98g
EQ3	365 (T) 259(T)	0.06/0.09 0.11/0.32	0.78g 0.59g
Mean	336 325	0.06/0.29 0.1/0.34	0.64 0.74
CV (%)	11 14	56/51 42/28	36 21

After investigating the response of the rack in terms of IDR, the stress/strain distribution on pipes (this examination can be categorized in local scale as discussed in CS2) is also essential in order to critically review the reliability of the design method followed and compare afterwards the pipes response with the IDR values as well as the maximum Peak Floor Acceleration (PFA) as defined by [7]. For that purpose, the two seismic levels, OBE and SSE, are considered. Considering the high dispersion of results along the perimetrical points of the rack, the floor acceleration is recorded at multiple points of the third floor of the rack yet only the maximum PFA is considered herein. According to ASCE/SEI 7-16 (2017), the PFA shall not be exceed three times the PGA of seismic input ($S_{e(T=0)}$=0.26g). The first remark upon the results refers to the linear response of the piping system for the 7 time histories. In Table 6, only the three time-histories with the highest stress found on the T-joint (T) and Nozzle (N) are presented. It is worth noting that the rack has considerably high overstrength and low ductility since IDR values no greater than 0.12% and 0.52% where observed for the X and Y direction, respectively, which makes the common IDR values not applicable for the present rack. The Coefficient of Variation (CV) has also been computed. Regarding the OBE, the CV is equal to 11% for the stress distribution, between 50-60% for the IDR in both directions and 36% in case of PFA. The high values of CV, particularly in the last two cases, may signify the complexity of pipe rack-piping system interaction. It may also verify that the ASD method could be unsafe, since the PFA is greater than the allowable value proposed by the code ($3 \cdot S_{e(T=0)}$) and thus additional design requirements shall be undertaken within the performance-based engineering framework.

Case-study #2

The pipe rack that is examined in this section comes from an existing LNG terminal plant that consist of different process units (Figure 4a, [35]). The pipe rack consists of 2 sub-racks; a 6x9x8.3 m short rack that supports pipelines that come immediately from the LNG storage tank and a 102x6x7.3 long rack that transfer the liquid to nearby units (Figure 4b). The rack is outfitted with 7 pipelines of ASTM A312/TP304L steel grade with yield and ultimate strength 370 MPa and 461 MPa, respectively, and zero internal pressure in order to stay on the safe side.

In stark contrast with the original design during which the rack was placed in a low-seismicity region, the following probabilistic assessment will be conducted assuming that the rack is placed in a high seismic-prone area of Priolo Gargallo, southeast of Sicily, Italy. This is justified by the need of highlighting and acquiring additional information of most vulnerable components. The supporting structure is modelled as decoupled with the pipework since the weight of nonstructural components is less than the 25% of the total weight of the system as the [7] specifies. The coupled case is being investigated by the Authors to evaluate the degree of dynamic interaction and will not be discussed hereafter.

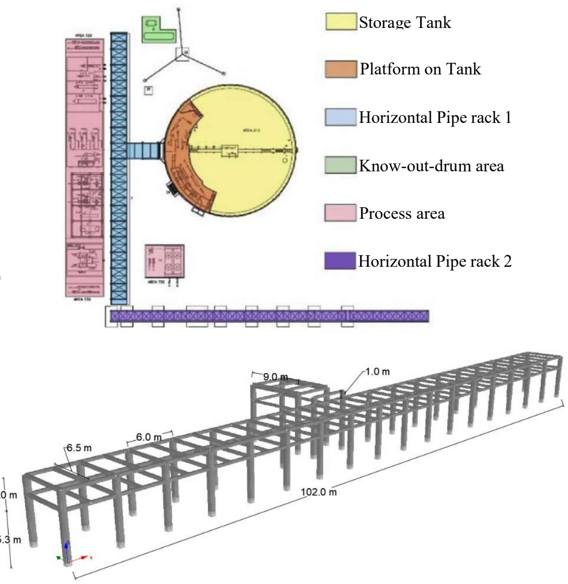
The assessment process of the rack that follows includes a number of steps. After the material and element modelling, a modal analysis of the rack is conducted, and the weakest direction is specified with PA. The seismic records are selected based upon the fundamental period of the weakest direction and the IDA follows for the evaluation of structural damage.

Assessment of RC rack

The rack is modelled in the finite element analysis software Seismostruct [36] (Figure 4b). The inelastic response of beams and columns is described by inelastic force-based frame elements that rely on the nonlinear fibre section method. 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 with isotropic hardening. A five-element non-uniform subdivision is adopted both for columns and beam members.

The modal analysis of the rack yielded the two fundamental modes shown in Figure 5. The capacity curves were derived for both directions and two load distributions (uniform and 1st mode), however, the results are not quoted for brevity. The weakest direction was used for the selection of spectrum compatible records within the time period range as the [6] specifies. Before the seismic analysis, acceptance criteria should be determined. Three Limit States (LSs) for two failure modes, namely shear force and chord rotation, are adopted according to [37] (Table 7).

(a)



(b)

Figure 4. (a) The LNG terminal plant and pipe rack in CS #2; (a) The LNG terminal layout ([35]), and (b) the RC pipe rack modeled by Seismostruct code.

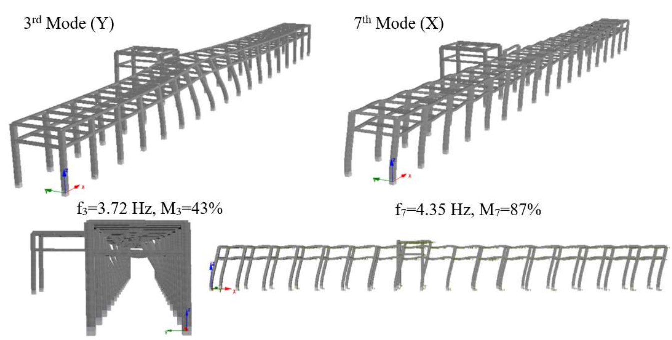


Figure 5. The first two principal modes in the X and Y direction

Table 7. Limit states for concrete members as per [37]

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)	$V_E \leq V_{Rd.EC2}$	$V_E \leq 0.75 \cdot V_{Rd.EC8}$	$V_E \leq V_{Rd.EC8}$

Assessment of steel pipelines

A critical point of rack-piping system modelling is the type of Boundary Conditions (BCs) for the pipes. Usually, pipes are modelled as unrestrained for the longitudinal and all rotational degrees of freedom. To be consistent with the initial design of the rack, fixed points have also been considered. Also, a conservative assumption of pinned connection at pipe edges has been made (Figure 6). The damage of pipes in the decoupled case was monitored and compared with two failure modes -fatigue cracking failure is excluded- as defined in [38] (see also Table 8). To facilitate the forensic investigation of pipes failure, the scalar Equivalent Plastic Strain (PEEQ) as defined on ABAQUS software was used to describe the global development of strains on pipes both in tension and compression. In particular, according to [38], the strain levels of ε_P=0.5% defines the first damage state on pipes in tension, whereas the corresponding strain value in compression is given by:

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

where t is the pipe thickness, D refers to the pipe diameter, E is the elastic modulus and σ_h is the minimum between the hoop stress due to internal pressure and the 40% of yielding stress. The last parameter in eq. 3 is important only when the pipe pressure is considerable, which has a positive influence against pipe buckling. Given that the metric PEEQ is used in this study, the LSs are computed by considering the minimum strain that comes after compression and tension for each pipe. Finally, the conservatively strain value of 2% is considered as ultimate tensile resistance.

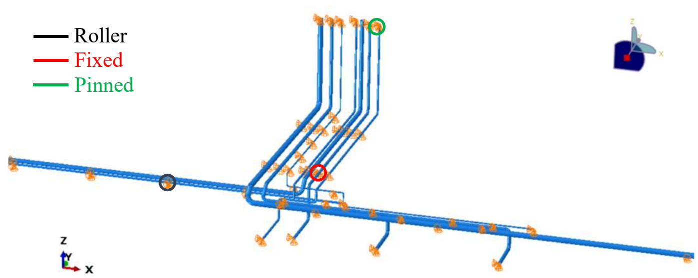


Figure 6. The piping system and the BCs considered

Table 8. Limit states for RC members as per [38]

Mechanism	EDP	Performance Level	Limit States (LSs)
Tensile fracture	tensile strain, ε_T	$\varepsilon_Y < \varepsilon_T \leq \varepsilon_P$ $\varepsilon_P < \varepsilon_T \leq \varepsilon_{Tu}$ $\varepsilon_T \geq \varepsilon_{Tu}$	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 assessment of pipes should be conducted by using accurate and at the same time simple models. Indeed, there is always a balancing act between accuracy and time constraints, particularly when large and complex models are dealt with. In the present study, in order to overcome this obstacle, stick models are adopted for the straight pipes and the special purpose element ELBOW32 as defined on ABAQUS for the pipe bent. The last is a complete elbow element that captures the ovalization and warping of the pipe section ([38]). The piping system that outfits the rack is shown in Figure 6.

Soil modelling

In the present preliminary examination of the SSI effects, a typical foundation which constitute concrete and connection (strip) beams is designed according to [39] (Figure 7) in Seismostruct software. The foundation design is conducted for an alluvium deposit, mostly sandy clay to clayey sand, which is categorised as Soil Type C (STC) in EN code [6] (V_s=210 m/s^2) in order to be consistent with the type of soil that is found at coastal sites. The soil comes after a microzonation study that refers to alluvium and surficial coastal or river deposits [40]. Initially, linear springs have been calibrated to model the soil ([41]), which are placed both under the footings and strip beams on Seismostruct. In order to acquire a better understanding of soil nonlinearity effects, the Ramberg-Osgood (RO) model included in Seismostruct tool set is calibrated for the STC. With the intention of describing the soil shear modulus (G) and damping ratio (D) as a function of shear strain (γ), the equations developed by [42], which account for soil plasticity, are used. Furthermore, to calibrate the RO model, a code is developed in MATLAB [43] that tries repeatedly to find the best fit of G-γ-D curves. Two methods are addressed for that purpose, namely the root mean square error and the coefficient of determination (R^2). Since the first priority is the G-γ curve to be captured as much as possible, the results that come after the former method are selected as shown in Figure 8.

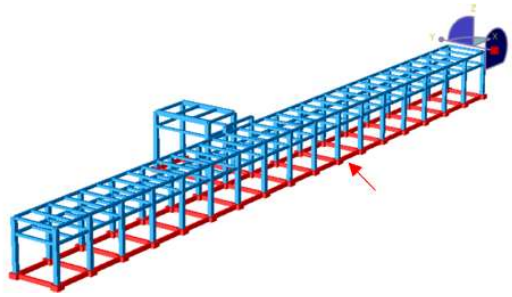


Figure 7. The foundation of RC rack in the decoupled case

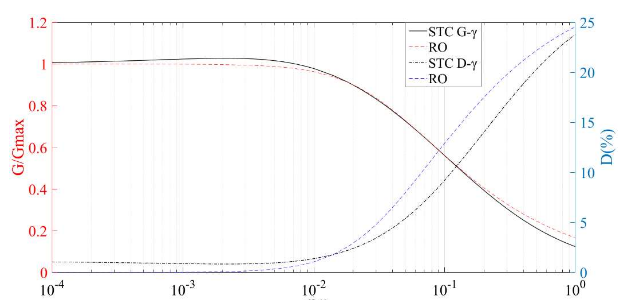


Figure 8. The G-γ-d curve for STC and RO model

It was found that the soil deformability increased the two fundamental periods shown in Figure 5 by 17% (T_3) and 20% (T_7) and decreased the participating modal mass by 41% and 56%, respectively.

Derivation of Fragility Functions (FFs)

It has been proved by numerous research efforts in the literature, that the lognormal Cumulative Distribution Function (CDF) describes well the Intensity Measure (IM) that causes structural damage. This general accepted function is given by:

$$P(C\,|\,IM = x) = \Phi\left(\frac{ln\left(\frac{x}{\theta}\right)}{\beta}\right) \tag{4}$$

where P is the probability of collapse (C) or LS exceedance given the IM, Φ() is the lognormal CDF and finally θ and β is the median and dispersion of the distribution. The smaller the β, the more appropriate the IM for the analysis of a structural system.

The rack is excited incrementally with a suite of 7 far-field spectrum compatible records (epicentral distance>15kms, [31]) till exceedance of predefined LSs. The scaling method followed is the same as described for the #CS1. It is pointed out that exceedance of LSs were not observed in all cases and particularly for the flexure failure mode. In that case, the truncated IDA was adopted to minimize the computation effort and keep the IM within practical limits ([44]).

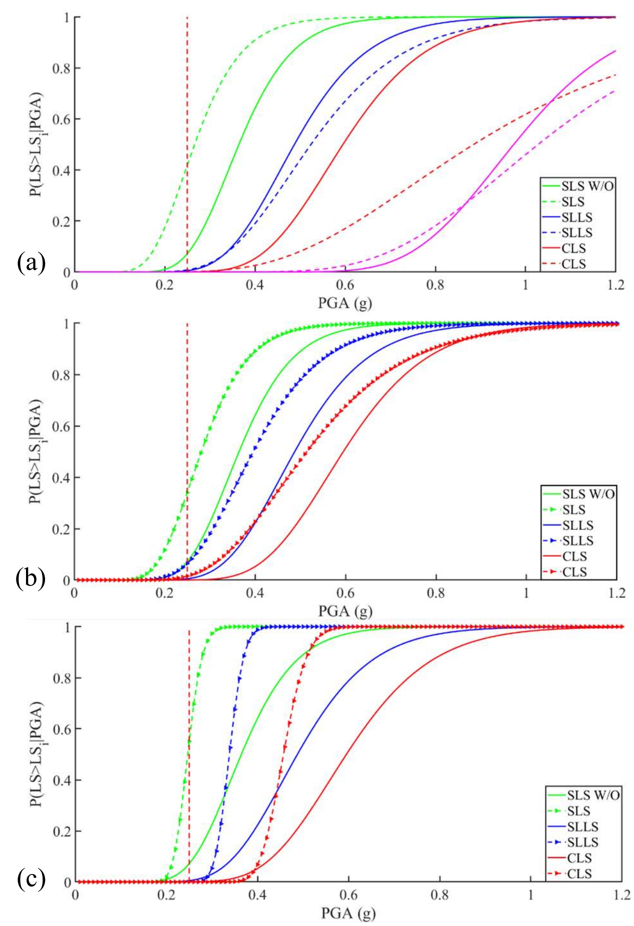


Figure 9. Fragility functions in decoupled case of long rack for: (a) columns and beams W/O SSI, (b) columns W/ SSI and linear soil, and (c) columns with SSI and nonlinear soil (design PGA=0.25 g, dashed line)

Initially, the FFs are discretized to those that refer to columns and beams. When comparing the results, it is obvious that the predominant failure mode of the rack is the shear in all cases (Figure 9a). The beams were found that fail earlier than the columns for the SLS, however, the columns presented higher fragility for the consecutive LSs. Regarding the effects of SSI, it is evident that the soil deformation moves the curves on the left (Figure 9b) and therefore has a negative impact on the rack fragility that comes from 0 up to 7% for the SLLS and linear soil. It is rather interesting that when the nonlinear soil is addressed for, the dispersion of damage both for beams and columns decreases considerably (in Figure 9c only the results for columns are shown for the sake of brevity) and this may indicate that the hysteretic behaviour of soil dissipates energy that makes the structural damage independent of modelling. This behaviour of SSI detrimental effects and lower dispersion due to soil nonlinearity has also been illustrated in [12]. However, the soil nonlinearity is being investigated by the Authors since the suite of seven records may not be sufficient sample to conclude to this statement with utter confidence.

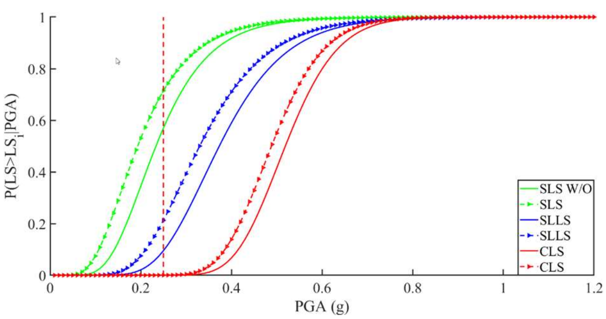


Figure 10. Fragility functions of pipes in decoupled case W/ and W/O SSI (design PGA=0.25 g, dashed line)

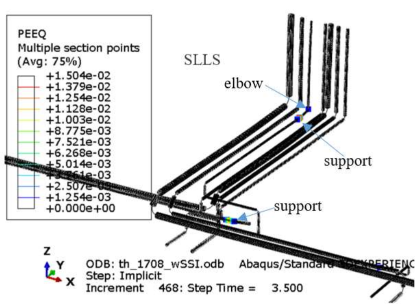


Figure 11. Plastic strain development on pipes corresponding to SLLS in decoupled case: a) W/O and b. W/ SSI (color signifies plastic deformation)

Finally, the response of pipes is examined on ABAQUS software. The pipes are excited at the pipe supports with the time-histories coming from the previous software. The main innovation of the present study constitutes the evaluation of the soil deformability impact on pipes response. The Figure 10 illustrates that the soil influence remains consistent compared to structural elements; however, the fragility of nonstructural components is higher by itself compared to structural ones and the SSI increases further the damage not up to the same degree with that of beams and columns, though. This behaviour can be attributed to the soil deformability that acts as a safe pad for nonstructural components. The fragility increases from 10 to 22% for the SLLS at PGA=0.25g and that might not be acceptable for the plant safety. It should be mentioned that the higher stress/strain is observed either at pipe supports, pipe edges or elbows verifying the significant role the correct definition of BCs plays during the seismic design process. The plastic strain development on the pipe with the worst response (DN 4”-SCH10S) is shown in Figure 11. The strain level corresponds to SLLS W/ SSI, which is by 6.5% higher than the fixed-base case.

CONCLUSIONS

The present research effort has initially shown the inconsistency of codes regarding the behaviour factor and the paucity of provisions that mainly EN codes present upon the design of pipe racks. The behaviour factor is prescribed with different reliability target in codes and the selection of the factor could be unjustifiable and non-risk related when it comes to nonbuilding structures, in particular. Two case studies were considered for the present analytical work and the main findings are summarized as follows:

- the pushover analysis (PA) method overestimates sensationally the lateral resistance of the rack. The behaviour factor was overestimated up to 16% by PA and occurred 29% lower than the design value in the X direction.
- common interstorey drift ratio (IDR) values cannot apply for pipe racks since the did not exceed the value of 0.52% in the Y direction when the maximum pipe stress was close to 80% of the yielding point.
- the CV was found 36% and up to 60% for the PFA and IDR, respectively, making the application of modern design methods necessary.
- the fragility analysis on the RC rack yielded the detrimental influence of soil that increased the SLLS probability of exceedance of columns at design PGA from 0 to 7%. Also, the dispersion of damage was lower in case of soil nonlinearity.
- the fragility of pipes in CS2 was rather high due to the initial BCs considered (the fragility increased from 10 to 22% for the SLLS at the design PGA and may not be acceptable). A modified displacement response spectrum that takes into account the SSI effects could be used to check the pipes stress/strain with respect to IDR values. Also, different BCs than those adopted during the initial design will probably decrease the seismic risk.

The coupled case as well as the nonlinearity of soil is still being investigated by the Authors regarding the effects on structural and nonstructural components and probably more advanced soil model will be used in the future along with pile foundations assuming that the soil beneath the rack is even looser and a surface foundation is not applicable. Finally, the IM to be used for probabilistic analysis of pipe racks should be examined due to the critical response of nonstructural components.

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REFERENCES

[1] Kidam, K. & Hurme, M., "Analysis of equipment failures as contributors to chemical process accidents," *Process Saf. Environ. Prot.*, vol. 91, no. 1–2, pp. 61–78, Jan. 2013.

[2] Bedair, O., "Rational Design of Pipe Racks Used for Oil Sands and Petrochemical Facilities," *Pract. Period. Struct. Des. Constr.*, 2015.

[3] Azizpour, O. & Hosseini M., "A Verification Study of ASCE Recommended Guidelines for Seismic Evaluation and Design of Combination Structures in Petrochemical Facilities," *J. Appl. Sci.*, vol. 9, no. 20, pp. 3609–3628, 2009.

[4] Di Roseto, A. L., Palmeri, A. & Gibb A. G., "Performance-based seismic design of a modular pipe-rack," *Procedia Eng.*, vol. 199, pp. 3564–3569, 2017.

[5] Drake, R. M. & Walter, R. J., "Design of Structural Steel Pipe Racks," *AISC Eng. J.*, pp. 241–252, 2010.

[6] EN1998-1, *Eurocode 8: Design of structures for earthquake resistance - Part 1 : General rules, seismic actions and rules for buildings*, vol. 1, no. English. 2004.

[7] ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. 2017.

[8] ASCE (2011), *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. Reston, VA: American Society of Civil Engineers*, 2nd Edition.

[9] Stewart, J. P., Kim, S., Bielak J., Dobry, R., & Power, M. S., "Revisions to Soil-Structure Interaction Procedures in NEHRP Design Provisions," *Earthq. Spectra*, vol. 19, no. 3, pp. 677–696, Aug. 2003.

[10] Elnashai, A. S. & Di Sarno, L, *Fundamentals of Earthquake Engineering: From Source to Fragility*, Second. Wiley, 2015.

[11] Anvarsamarin, A., Rofooei, F. R., & Nekooei, M., "Soil-Structure Interaction Effect on Fragility Curve of 3D Models of Concrete Moment-Resisting Buildings," *Shock Vib.*, 2018.

[12] Karapetrou, S. T., Fotopoulou, S. D. & Pitilakis, K. D., "Seismic vulnerability assessment of high-rise non-ductile RC buildings considering soil-structure interaction effects," *Soil Dyn. Earthq. Eng.*, 2015.

[13] Kwon, O. S. & Elnashai, A. S., "Fragility analysis of a highway over-crossing bridge with consideration of soil-structure interactions," *Struct. Infrastruct. Eng.*, 2010.

[14] EN 1997-1 (2004) (English): *Geotechnical design - Part 1: General rules [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC].*

[15] EN 1998-5 (2004) (English): *Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]*. 2004.

[16] Elghazouli, A. Y., "Assessment of European seismic design procedures for steel framed structures," *Bull. Earthq. Eng.*, vol. 8, no. 1, pp. 65–89, 2009.

[17] Asgarian, B. & Shokrgozar, H. R., "BRBF response modification factor," *J. Constr. Steel Res.*, vol. 65, no. 2, pp. 290–298, 2009.

[18] Izadinia, M., Rahgozar, M. A., & Mohammadrezaei O., "Response modification factor for steel moment-resisting frames by different pushover analysis methods," *J. Constr. Steel Res.*, vol. 79, pp. 83–90, 2012.

[19] Celano, F., Žizmond, J. & Dolsek, M., "The evalutation of risk-targeted safety factor and behaviour factor for selected steel structures.," in *16th European conference on Earthquake Engineering*, 2018.

[20] Dolšek, N., Kosič, M., Žižmond, J. & Sinković, N. L, *Development of Eurocode 8, Proposal for Annex F (Informative) Simplifed reliability-based verification format, Rev. 3, University of Ljubljana, 16.6.2017, Ljubljana.* 2017.

[21] Karamanos, S. A., Patkas, L. A. & Platyrrachos, M. A. "Sloshing Effects on the Seismic Design of Horizontal-Cylindrical and Spherical Industrial Vessels," *J. Press. Vessel Technol.*, vol. 128, no. 3, p. 328, 2006.

[22] Bursi, O. S., Reza, M. S., Abbiati G. & Paolacci, F., "Performance-based earthquake evaluation of a full-scale petrochemical piping system," *J. Loss Prev. Process Ind.*, vol. 33, pp. 10–22, 2015.

[23] Bursi, O. S., Paolacci, F., Reza, M. S., Alessandri, S. & Tondini, N., "Seismic Assessment of Petrochemical Piping Systems Using a Performance-Based Approach," *J. Press. Vessel Technol. Trans. ASME*, vol. 138, no. 3, 2016.

[24] EN13480-3 (2002), *"Metallic Industrial Piping–Part 3: Design and Calculation, CEN, Brussels.,"* 2012.

[25] ASME B31.3, *"ASME Code for Pressure Piping, B31 - ASME B31.3-2008 (Revision of ASME B31.3-2006)," Chem. Eng., vol. 76, no. 8, pp. 95–108, 2008.*

[26] ABAQUS, "Analysis User's Manual'. Online Documentation Help: Dassault Systèmes.," 2017.

[27] DeGrassi., G, Nie, J. & Hofmayer, C., Seismic Analysis of Large-scale Piping Systems for the JNES-NUPEC Ultimate Strength Piping Test Program. 2008.

[28] NTC, 2018, *"Norme Tecniche per le costruzioni", DM Infrastructure, 14 January*. (in Italian)

[29] Mwafy, A. M. & Elnashai, A. S., "Static pushover versus dynamic collapse analysis of RC buildings," *Eng. Struct.*, 2001.

[30] Vamvatsikos, D. & Cornell, C. A., "Incremental dynamic analysis," *Earthq. Eng. Struct. Dyn.*, vol. 31, no. 3, pp. 491–514, 2002.

[31] Heydari, M. & Mousavi, M., "The Comparison of seismic effects of near-field and far-field earthquakes on

relative displacement of seven-storey concrete building with shear wall," *Curr. World Environ.*, 2015.
[32] Di Sarno, L. & Karagiannakis, G., "Petrochemical steel pipe rack: Critical assessment of existing design code provisions and a case-study (under revision)," *Int. J. Steel Struct.*, 2019.
[33] Dolšek, M., Sinković, N. L. & Žižmond, J., "IM-based and EDP-based decision models for the verification of the seismic collapse safety of buildings," *Earthq. Eng. Struct. Dyn.*, vol. 46, no. 15, pp. 2665–2682, 2017.
[34] Chioccarelli, E., Cito, P., Iervolino, I. & Giorgio, M., "REASSESS V2.0: software for single- and multi-site probabilistic seismic hazard analysis.," *Bull. Earthq. Eng.*, no. Submitted, 2018.
[35] Bursi, O. S., Di Filippo, R., La Salandra, Vi, Pedot, M., & Reza, M. S., "Probabilistic seismic analysis of an LNG subplant," *J. Loss Prev. Process Ind.*, 2018.
[36] SeismoSoft, "A computer program for static and dynamic nonlinear analysis of framed structures," *Available from URL: www. seismosoft. com*. 2018.
[37] Fardis, M. N., "From Performance- and Displacement-Based Assessment of Existing Buildings per EN1998-3 to Design of New Concrete Structures in fib MC2010. In: Ansal A. (eds) Perspectives on European Earthquake Engineering and Seismology. Geotechnical", 2014.
[38] Vathi, M., Karamanos, S. A., Kapogiannis, I. A. & Spiliopoulos, K. V., "Performance criteria for liquid storage tanks and piping systems subjected to seismic loading," *J. Press. Vessel Technol.*, 2017.
[39] Fardis, M. N., *Seismic Design, Assessment and Retrofitting of Concrete Buildings*, vol. 8. Dordrecht: Springer Netherlands, 2009.
[40] Anastasiadis, A., Raptakis, D. & Pitilakis, K., "Thessaloniki's detailed microzoning: Subsurface structure as basis for site response analysis," *Pure Appl. Geophys.*, vol. 158, no. 12, pp. 2597–2633, 2001.
[41] Gazetas, G., "Foundation Vibrations," in *Foundation Engineering Handbook*, 1991, pp. 553–593.
[42] Ishibashi, I. & Zhang, X., "Unified dynamic shear moduli and damping patios of sand and clay.," *Soils Found.*, 1993.
[43] MATLAB and Statistics Toolbox Release 2018a, "The MathWorks, Inc, Natick, Massachusetts, US" .
[44] Baker, J. W., "Efficient analytical fragility function fitting using dynamic structural analysis," *Earthq. Spectra*, 2015.