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SEISMIC ANALYSIS AND COMPONENT DESIGN OF REFINERY PIPING SYSTEMS

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Abstract. *In petroleum industries, especially in refineries installations, hundreds of miles of pipes are installed to transfer row and refined material (fluid and gas) from a point to another of the plant, connecting all the components involved in the transformation process (tanks, distillations columns, furnaces, etc..).Recent seismic events showed a quite high vulnerability of these structures, where damage ranges from the simple failure of joints to the failure of supporting structures. For these reasons, initially the seismic analysis and component design of refinery piping systems is here analysed. A review of the current approaches imposed by European (EN13480:3) and American (ASME B31.3) standards is illustrated by using a proper case study of a piping system on a pipe-rack. The analysis permitted to identify the limits of the design standards and to identify the critical aspects of the problem, i.e., dynamic interaction between pipes and rack, correct definition of the response factor, strain versus stress approach. Finally, the preliminary phases of an experimental investigation on flanged joints are also illustrated.*

1 INTRODUCTION

The piping systems typically found in a refinery complex contain various components and support structures and operate in a broad range of working environments. Some common components usually used in piping systems include straight pipes, elbows, Tee-Joints, various valves, flanged joints, pressure vessels, tanks, strainers and reducers. Depending on the nature of the working fluids, piping systems are designed to work over a wide range of temperature and pressure. A typical piping system is presented in Figure 1.

A pipe-way is the space allocated for routing several parallel adjacent pipelines within process plants. A pipe rack is the structure employed for supporting the pipelines and carrying electrical and instrument trays. The pipe rack is usually made of steel or concrete frames, on top of which the pipeline rests.

Pipe racks are necessary for arranging the process and service pipelines throughout the plant, and they are used in secondary ways; principally to provide a protected location for auxiliary equipment, pumps, utility stations, manifolds, and firefighting and first-aid stations. Lighting and other fixtures can be fitted to the pipe rack columns. Air-cooled heat exchangers are often supported above pipe racks for economy of plot space.

Figure 1 Typical piping layout

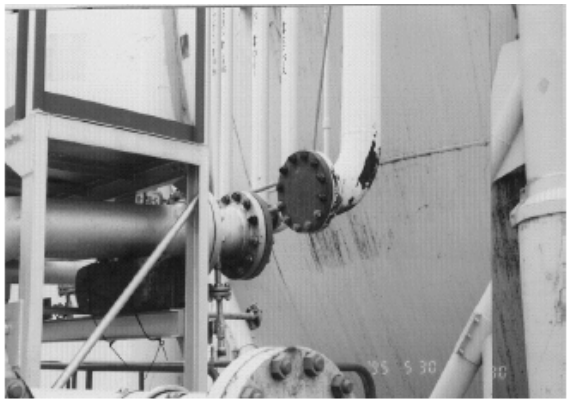


Figure 2 : Breakage of a piping flanged connection

Recent seismic events showed a quite high vulnerability of these structures, where damage ranges from the simple failure of joints to the failure of supporting structures [1],[2],[3],[4]. For example, the failure of a bolted flange connection is shown in Figure 2.

Consequences can be characterized by several degrees of severity, depending on the material delivered by pipes. For dangerous liquids or gases, even a simple failure of a joint can represent the trigger of a significant accidental chain, with severe consequences both for the environment and human lives. Unfortunately, few contributions in the literature are available, in order to clarify the seismic requirements that piping systems have to comply with. In addition, aspects like action and structural modelling have not yet been treated in a satisfactorily manner. Moreover, current American and European standards do not contain enough rules and details for a proper seismic analysis and design of piping systems.

Along these lines, some problems relevant to seismic analysis criteria of piping systems are addressed in this paper. In a greater detail, several aspects that characterise the problem are treated: 1) modelling of pipes and pipe-racks; 2) selection of the analysis method; 3) definition of the seismic action; 4) dynamic analysis of the system; 5) stress analysis of pipes; 6) definition of the ultimate capacity of pipes and joints between pipes.

To this end a representative case study of an actual piping system was analysed. The requirements of American and European codes were compared and important aspects were hig-

hlighted, like: a) dynamic coupling between pipes and pipe-rack, often erroneously neglected; b) definition of proper restraint conditions between pipes and support structures and between adjacent piping systems; c) evaluation of the ultimate capacity of pipes and joints necessary for a correct design of a structure, as suggested by modern approaches like Performance-based Engineering.

About the aforementioned item c) an experimental campaign is currently undertaken at the University of Trento in order to characterise the cyclic behaviour of flanged joints between pipes, particularly important to avoid leakage of dangerous substances during a severe seismic event. Preliminary information about the testing activity is illustrated and commented.

2 EUROPEAN AND AMERICAN STANDARS FOR THE SEISMIC DESIGN OF PIPING SYSTEMS

Nowadays, both European and American codes are available for the design of piping systems under seismic events. The main European contributions is chiefly represented by the standard EN13480, dedicated to metalling piping systems [5]. The Eurocode 8 - part 4, the European seismic code for industrial components, is also devoted to pipelines, but only of above-ground type, which differs from metallic piping system for many aspects, and then useless.

American experience on piping system is very rich, especially in terms of design standardization and seismic design calculation, and the long list of standards and codes available it is a clear demonstration of it. The main standard is represented by ASME B31.3 [6], but many other contributions and guidelines are also available [7],[8].

The seismic analysis of a piping system involves several basic steps that allow defining the proper seismic action, the suitable numerical model and analysis method and the verification format to be used. The European (EN13480:3-2002) and the American standards (ASME B31.3-2006) for piping systems differ for several of these aspects. Thus, in order to understand these differences and the consequences on the seismic response evaluation, in the following both European and American standards are applied to a representative case study.

It is necessary to stress that the American Standard does not contain explicit indications on the seismic analysis of piping systems, but rather refers to the American standard for seismic analysis of structure ASCE7-05, which includes all the required prescriptions. On the contrary the European Standard contains an entire Annex (A) dedicated to the dynamic and seismic analysis of piping systems, but does not contains explicit quantification of the seismic action. At this end the Eurocode 8 (prEN1998:1 2004) should be used.

3 DESCRIPTION OF A CASE STUDY

The piping system here analysed belongs to a refinery, whose plant view is shown in Fig.3. The supporting steel structure is composed by seven transverse moment resisting frames placed every 6 m, realized with commercial HEA/B steel profiles. In the longitudinal direction it behaves like a truss structure, which is reinforced with 6 braces (see Fig. 4, Fig.5). Horizontal bracings are also installed to avoid excessive relative displacements between the pipe supports.

The piping system presents a typical piping layout with pipes having different diameters. To simplify the analysis, only the structural contribution of 8’’ pipes has been considered, whose layout is shown in Fig. 6. The remaining pipes are considered only as weight. Several flanged elbows are present within the pipe-rack and at both the ends of the piping system.

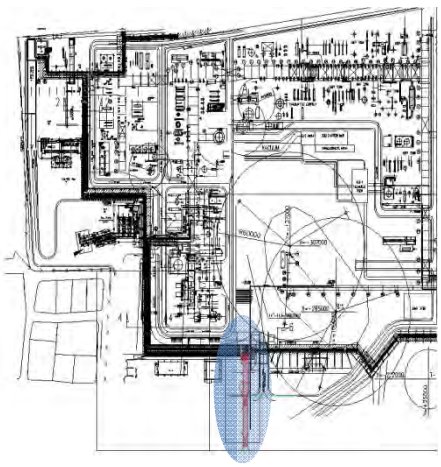


Figure 3 – Plan view of the refinery

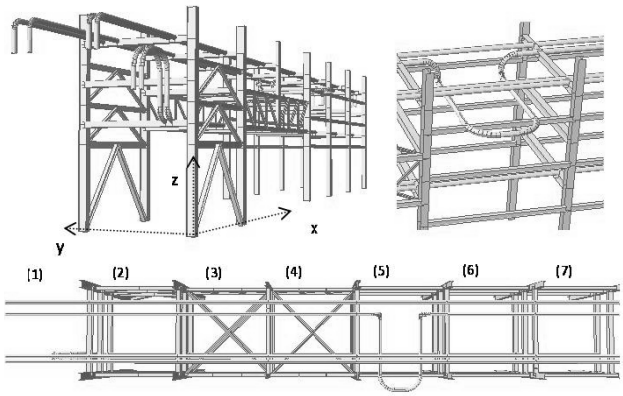


Figure 4 – The piping system

The fluids contained in the pipes are several, but essentially Amine, cooling water and high to medium pressure steam. The vertical loads corresponding to the weight of the pipes, insulation and fluid are considered as uniformly distributed equal to 12 kN/m. The main characteristics of the piping system are the following:

- Structural steel S-275 JR according to EN 10025 (2005)
- Pipe steel - ASTM A106 Grade B
- pipes with diameter of 8’’
- Pressure of the pipes: 0.5÷5 Mpa
- Temperature range 47 °C ÷360 °C.
- Seismic category of Importance Ip=1.5, PGA=0.24 g, Soil conditions: D

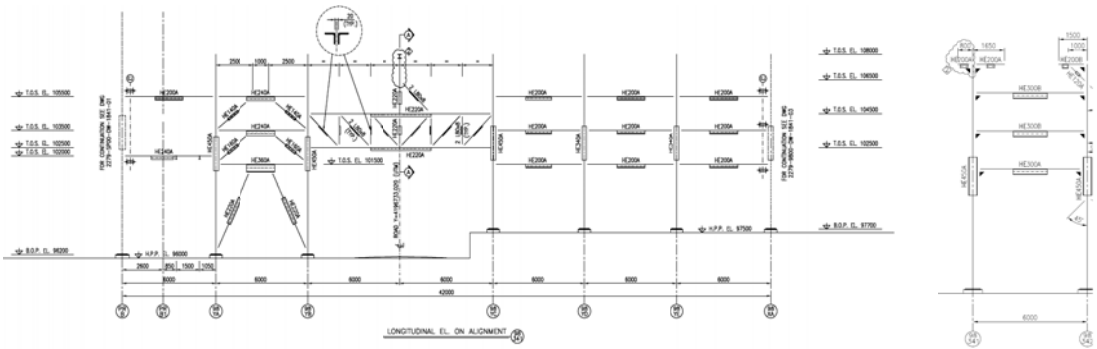
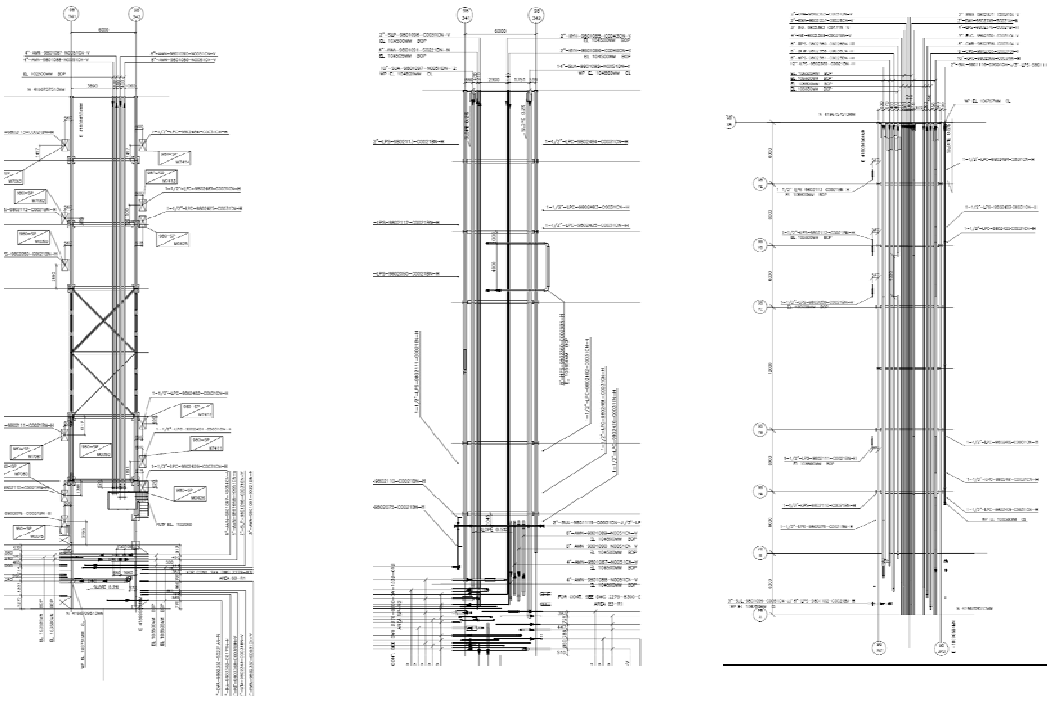


Figure 5 - The analysed piping system



First floor Second floor Third floor

Figure 6 - The analysed piping system

4 MAIN ISSUES IN SEISMIC DESIGN OF PIPING SYSTEMS

The seismic design of a piping system entails many issues. As stated above, they are essentially related to the modelling of the structure, to a correct definition of the seismic action, to a proper analysis method to be applied, and finally, to a appropriate design method to be used. In addition, nowadays, the Load and Resistance Factor Design method (LRFD) is certainly the standard method for designing structures and the allowable stress design method is by now abandoned because often considered too much conservative. Unfortunately, this latter is still the current approach for designing a piping system. This represents a limit to be overcome. But to the purpose, it is firstly necessary to understand the limit of such an approach; and the more direct way is to apply the codes prescriptions to a case study, trying to individuate limits and drawbacks. In doing this, it seems proper to compare European and American experiences, applying both the EN13480:3 and ASME B31.3 to the case study described above along with a comparison of the results regarding all the treated aspects.

4.1 Definition of a numerical model

A synthetic scheme of what European and American standard prescribe for a correct numerical modelling of a piping system and the definition of the seismic conditions is reported in Table 1.

The table clearly shows that the suggested numerical model to use in seismic analysis is always elastic both for EN13480 and ASME B31.3. This choice comes certainly from the old way to evaluate the safety level of a structure: the allowable stress method, still diffused in designing of piping systems. Usually, only the piping system is modelled, using the supporting structure only to evaluate the seismic action at pipes level (e.g., in-structure spectra). The supporting structure (e.g. pipe-tack) is treated as elastic too. The assumption of elastic beha-

vior would not be a strong limitation if a correct value of the behavior factor were adopted. Some comments on this aspect will be provided afterwards.

Another important aspect, often related to the assumption on the numerical model is the analysis method to be adopted. This ranges from the very simple equivalent static method to the time-History analysis. But, this aspect will be treated in the next section.

A key point in modelling a piping system is the possibility to neglect the interaction (static and dynamic) between the pipes and the supporting structure. EN13480 does not provide any indication about it, whereas ASME B31.3, by means of ASCE-07, prescribes a crude rule based on the ratio WR between the weights of pipes and supporting structure. In particular, if WR < 25% the interaction can be excluded and the piping system can be considered as a non-building structure, loaded by a seismic action coming from the supporting structure at pipes level.

This rule has been recently analysed by several authors. For example in [9] the rule has been analyzed using time-history analysis. From the results and discussion the author concluded that in some cases this decoupling rule could produce gross errors in the evaluation of the dynamic behavior of piping systems. In particular, it seems that in dynamic assessment of such systems, in addition to the primary-secondary system weight ratio criteria, attention should be paid to other aspects as “end conditions of pipes”, “relative stiffness of supporting structure to piping system” and “relative stiffness of pipes to pipe-supports”, even though only partial conclusions where reached by the authors, that suggested more investigations on this matter. In the literature many works have been dedicated to the problem of dynamic coupling between primary and secondary systems [10],[11],[12], but the results are difficult to be extended to the case of piping systems for several reasons, but mainly because in case of piping systems the secondary system is composed by more sub-systems (pipes with different diameters, different end conditions, different supports, etc..). Therefore, assumption of a single secondary system could lead to gross errors in response prediction of piping systems.

(1) elastic calculation shall be used although some part might be exhibit plastic deformations (p. 4.1), (2) SSE=safe shutdown earthquake, DBE= Occasional operation condition, (3) FF=Flexibility Factor, SIF= Stress Intensification Factor, (4) G=ground motion spectra, IS=In-structure spectra.

Code	**Model type**	**Analysis**	**Seismic condition**	**Dynamic interaction**	**Relative motion**	**Pipe modelling**
EN13480:3	Elastic (1)	Equivalent Static Modal (G,IS[4]) Time-History	SSE (2) OBE	NO	YES	Beam elements (FF and SIF)
ASME B31.3	Elastic	Static Modal Time-History	SSE	YES/NO	YES	Beam elements (FF and SIF)(3)

Table 1 – Code prescriptions for numerical modelling as well as seismic conditions

Concerning the case study, applying the weight ratio rule, dynamic interaction has been considered, because WR > 25%. Moreover, for comparison, this rule has also been considered in applying the European standard. In order to evaluate the effectiveness of the weight rule, the behavior of the piping system without pipe-rack has also been analysed.

The pipes are usually connected to the pipe-rack by mechanical supports, often flexible. They are usually designed to accommodate thermal and pressure effects, avoiding excessive stress in the pipes. Unfortunately, no indications on how to model them are provided by European and American standards. Nevertheless, this is an important aspect that have to be treated with particular attention because can cause changes in dynamic behavior of the system.

Dissipative supports can be used instead of traditional ones in order to introduce artificially more damping and reduce the response of the pipes.

The analysed case present a quite stiff support systems, modelled as elastic spring in the transverse direction (Y), leaving free the relative displacements in longitudinal direction (X) and using fix restraints conditions in vertical direction. Moreover, as usual, all the rotations between pipe and pipe-rack have been unrestrained.

Another relevant aspect about modelling of piping systems is the adoption of a proper model for pipes and fittings (elbows, tee-joints, nozzles, etc..). At this regards, usually beam elements with hollow section are used for straight pipes (Fig.7). The fittings are also modelled using beam elements, but modifying the stiffness for the effect of geometry. At this regards both European and American codes define a flexibility factor (k >1) using which the moment of inertia of the pipe is reduced. In addition, to take into account the stress concentration effect, the Stress Intensification Factor (SIF) is used to increase the stress calculated using the beam theory. The value of k and SIF calculated according to EN13480 and ASME B31.3 are very similar.

In presence of high pressure condition the stiffness of the pipe could increase. To account for this effect the flexibility factor k is reduced. Unfortunately, only ASME B313.3 provides an explicit expression of the pressure reducing factor. For the analyzed case and for a pressure of 5 MPa this factor halves the flexibility.

Alternatively, it is possible to use shell elements to model fittings [13]. This approach is appropriate to account for ovalization of the section and stiffening pressure effect. For this reasons this model has been used for the case study and a comparison with beam model has been carried out (Fig. 8). We can anticipate that the numerical simulations have shown a similar behavior of both the models and the reliability of modified beam element, at least for standard fittings, like the one here analyzed. Therefore, in what follows only the results of the more refined one will be shown.

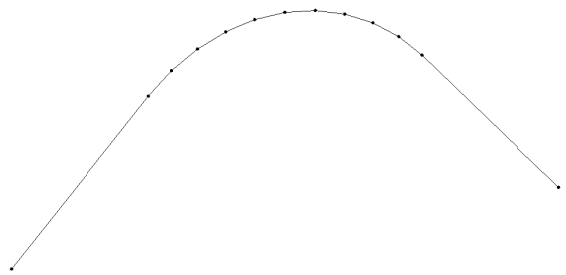


Figure 7 – Beam FEM for an elbow

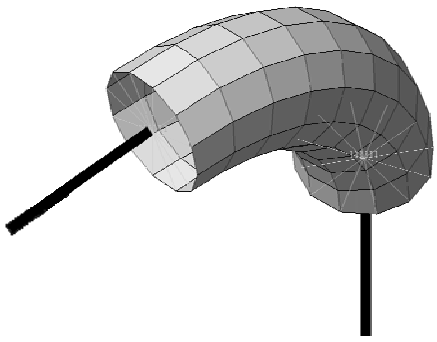


Figure 8 – Shell FEM for an elbow

A last but not less important aspect regards the boundary conditions of the pipes. In fact, because a piping system is realized by hundreds of miles of pipes, the analysis involves necessary a limited part of the structure. Consequently proper boundary conditions have to be accurately adopted to simulate the remaining part of the structure. Also for this delicate aspect no indications are provided by the European and American standards. As already shown in literature, the correct boundary conditions may cause important modifications of the dynamics of the piping system but the correct choice depends on the single case [9].

Because one of the aim of this work is to compare European and American standards, it has been decided to limit the possible cases adopting as restraint conditions of the pipe ends only hinges. In this respect, one can look at Fig. 9.

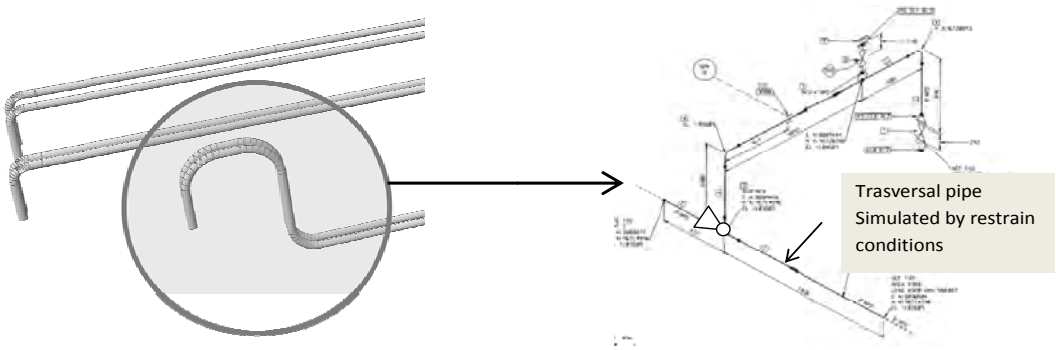


Fig. 9 – Boundary condition in one if the pipe ends

The model of the piping system and the support structure has been realized in the general purpose software MIDAS Gen [14]. It is shown in Fig. 10.

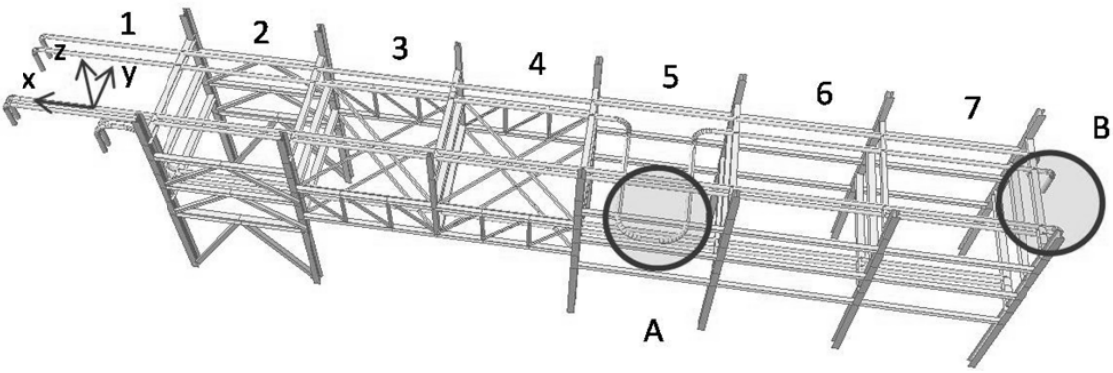


Figure 10 – The FE model of the piping system

4.2 Seismic actions and analysis methods

Both European and American standards assume the following two types of analysis mandatory for the pipes:

- Movements due to inertia effects
- Differential movement of the supports (within the supporting structure or between adjacent pipe-racks)

The first type of analysis is essentially related to the effects of the absolute acceleration on the pipe mass. The second one is due to the relative movements between two supports, within the supporting structure or belonging to adjacent structures. Often the relevant effects are due to the displacement effect rather than acceleration effects.

Concerning the case study, the entire model here considered (pipe + pipe-rack) allow identifying both the effects. On the contrary the model without considering the supporting structure, here also considered, allows identifying only the inertia effect of the pipes, unless a multi-support excitation would be used.

The seismic action for pipe-racks is usually represented by design response spectra or accelerograms (natural records or synthetic accelerograms). For the analysis of pipes only, "In-structure" spectra or "filtered response spectra" are used.

Design response spectrum method

The design spectra are the main representation of a seismic action and usually are defined by the seismic codes in terms of hazard conditions of the site, the level of dissipation capability of the supporting structure and pipes (response or behavior factor), the right level of damping to be employed, and the level of structure reliability to impose, identified by the importance factor.

For what regards the support structure, hazard conditions apart, the damping usually adopted is equal to 5%, as suggested by Eurocode 8 and 3 for steel structures, whereas the behavior factor q depends on the type of structure used for the pipe-rack. While for building-type structure this aspect has been well identified and quantified, for structures like pipe-racks, that may often be considered as non-building structure (ASCE-07:2005), the problem may be quite different.

The current American and European seismic codes provide a q factor for steel racks equal to 3 ½ and 4 respectively (Tab 2). This choice probably derives from the hypothesis of no-coupling between the rack (primary system) and the pipes (secondary system). In fact, usually the level of dynamic coupling between pipes and rack can be neglected. But in other some cases its influence cannot be excluded a priori [9].

In order to compare the results of EN13480 and ASME B31.3 the same elastic spectrum, has been here adopted defined according to prEN1998:1, and modified using the behavior factor of Tab. 2. It is shown in Fig. 13.

The spectral response of the modal oscillators and then combined to obtain the resultant response of the system. Moreover, the resultant forces and displacements from bi-directional analysis are typically obtained by the square root sum of square of the response in each direction, or by applying the so-called 100-30 rule.

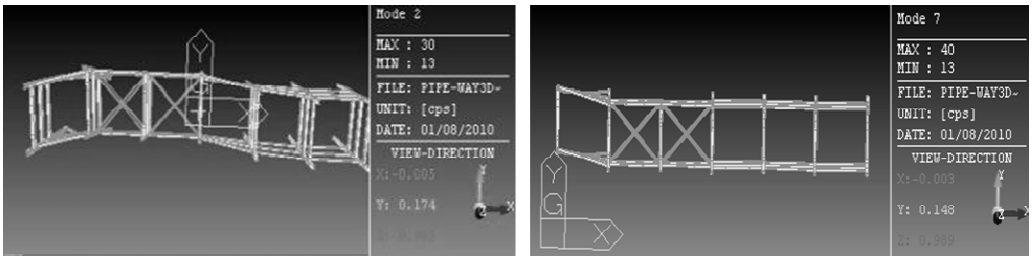


Figure 11 First two vibration modes of the pipe-rack model

In Tab.3 the main dynamic characteristics of the system are shown. As mentioned before, the modal analysis on the entire system allows highlighting the important role of the pipes in realizing structural coupling between the several frames of the pipe-rack. For example in Fig. 11 and 12 the vibration modes of the rack with and without the pipes is shown. They are quite different especially in terms of excited mass. For example, the period of the first mode of the

rack with and without pipes is very similar, whereas the excited mass is higher in the first case, showing the coupling effect of the transverse frames due to the pipes.

All the results here presented have been carried out for the elbows modelled using shell elements. The results using the simpler beam elements (here not presented) are quite close to previous ones, showing their reliability.

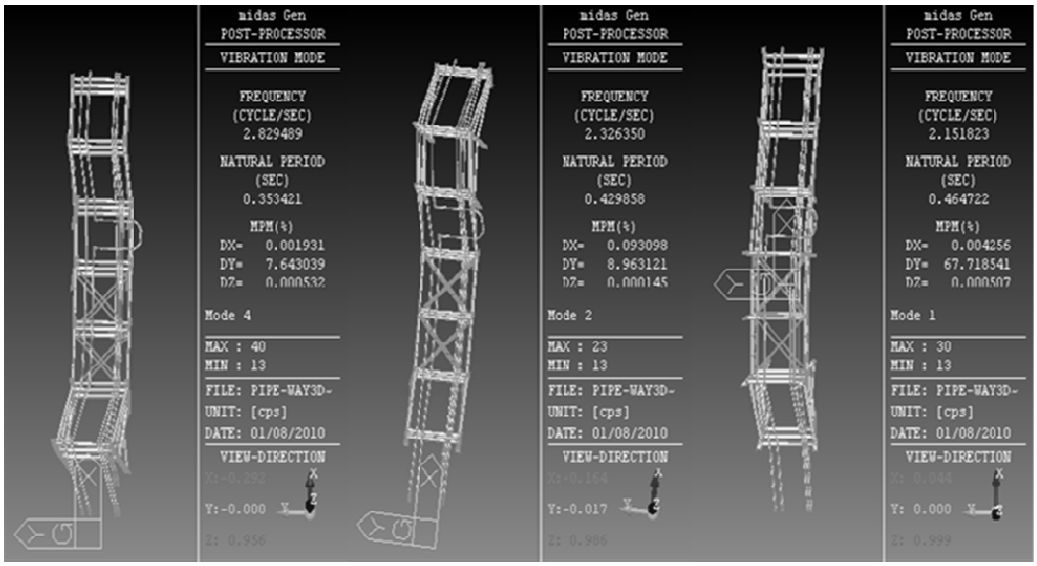


Figure 12 First three vibration modes of the pipe plus pipe-rack model

Code	Behavior factor R (pipe-rack)	response factor R_p (pipe)	Damping (%)	Importance F.
EN13480:3 (EC8)	4	Not indicated	5	1.5
ASME B31.3	3 ¼	6÷12	5	1.5

Tab. 2 – Behaviour factor, damping and Importance Factor

Case	Mode 1			Mode 2			Mode 3		
	f (Hz)	T(sec)	MPS(%)	f (Hz)	T(sec)	MPS(%)	f (Hz)	T(sec)	MPS(%)
TD-PR (*)	2.17	0.459	28.29	2.33	0.456	47.62	2.93	0.440	5.68
TD-PRP(*)	2.15	0.456	67.61	2.33	0.429	8.96	2.93	0.353	7.64
LD-PRP(*)	2.81	0.355	53.27	5.15	0.194	36.35	---	---	---

(*) TD-PR=Transversal direction – Pipe-rack, TD-PPR=Trans. dir. – Pipe-rack+pipes, TD-PR=Longitudinal dir. – Pipe-rack+pipes

Table 3 – First three vibration modes of the entire piping system

In-structure spectra

The in-structure spectra allow a seismic action to be defined for the single pipe at several floors of the pipe-rack, in which the pipe is placed. Both European and American codes give their explicit expressions. They are defined as the spectrum acceleration multiplied by the amplification factor AF shown in Fig. 14. The expression of the amplification factor expressed by EN13480 (EC8) and ASME B31.3 (ASCE-07) differ for the presence of the period

ratio Ta/T1, presents only in the seismic European Standard which takes into account the dynamic interaction between pipes (Ta) and pipe-rack (T1). (see tab. 4).

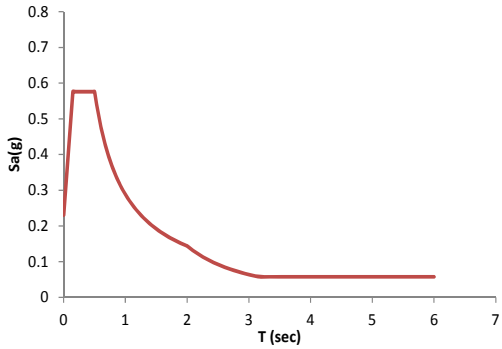


Figure 13 Eurocode 8-1 free-field elastic spectrum (Soil D)

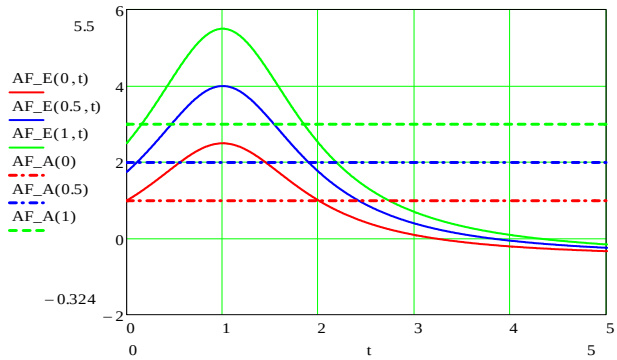


Figure 14 Amplification factor of in-structure spectra v/s T_a/T_1

In case of piping, because is generally avoided any yielding phenomenon in the pipes, the dissipation capability is generally restricted only to the supporting structure and the relative response modification factor depends on its structural configuration. In some cases dissipation phenomena may have place also in the pipes and a specific behavior factor have to be defined if the elastic analysis with response spectra is used.

The behavior factor provided by the codes, especially by the American one, seems to be overestimated. For example, ASCE-07 prescribes the use of a behavior factor 6 or 12 according to the deformability of the material used. In some cases this hypothesis may not be totally true.

For example Okeil and Tung in 1996 [15] using an idealized piping system have suggested a closed-form formula that provides the reduction factor q as function of the ductility of the support structure, μ, and the ratio f of the piping frequency and the frequency of the seismic action. The reduction q increases with the ductility, μ. Moreover, the stiffer is the piping system (low values of f) the higher is q. In any case q is never greater than 3-4. These results are in contrast with the provisions of ASCE-07.

Code	Expression	note
ASME B31.3 (ASCE-07)	$F_p=I_p\frac{0.4a_pS_{DS}W_p}{R_p}AF$ $AF=\left(1+2\frac{z}{H}\right)$	*a_p=2.5 (Tab. 13.6-1)* *S_{DS}=Maximum spectral acceleration* *W_p=Weight of the pipe* *z=height where the pipe is placed* *H=total height of the pipe-rack* *I_p=Importance factor* *R_p=response factor*
EN13480:3 (EC8)	$F_a=I_p\frac{a_gSW_p}{R_p}AF$ $AF=\left[\frac{3\left(1+\frac{z}{H}\right)}{1+\left(1-\frac{T_a}{T_1}\right)^2}-\frac{1}{2}\right]$	*a_g=Peak Ground Acceleration* *S=soil factor* *T_a=fundamental period of the pipes* *T_1=fundamental period of the pipe-rack*

Table 4 In-Structure spectra definitions

Other authors reached similar conclusions as well. For example in [16] the authors investigated on the dynamic response of a piping system on a rack with gap and with friction. By using dynamic harmonic analysis on a simple system they found that in presence of nonlinearity (friction between pipes and rack) the reduction of the acceleration with respect to the elastic case could be of the order of 2-3, value suggested also by the American Lifelines Alliance [7] and FEMA 450.

Instead of using static method employing seismic force of Tab. 4, a dynamic approach is also possible, using generated floor response spectra, whose shape is based on typical Soil-Structure interaction theory. For industrial piping systems few contributions have been found in literature [2], [3], [4], [17].

For the case study of Fig. 4**,** floor spectra have been generated by using time-history analysis. Fig. 15 shows the floor spectra for one the three accelerograms used in Time-History analysis (see next section), from which is clear the filtering effect of the pipe-rack. The maximum amplification is now restricted to the range 2-3 Hz, whereas in the ground acceleration spectra was placed in the range 2-6 Hz. The labels Frame 1, 3 and 5 correspond to the labels of the frames indicated in Fig.10. It is also possible to note the variability of the maximum amplification effect within the structure. The maximum acceleration peak is found at frame 3; this because more mass is placed there with respect to the other frames.

Because the natural frequencies of the piping systems are in the range 15-40 Hz the amplification effect due to the inertia is very limited. In fact, the maximum acceleration applied to the pipes corresponds more or less to the maximum acceleration at floor in which the pipes are placed, some peaks a part, at frequency 7 and 10 Hz.

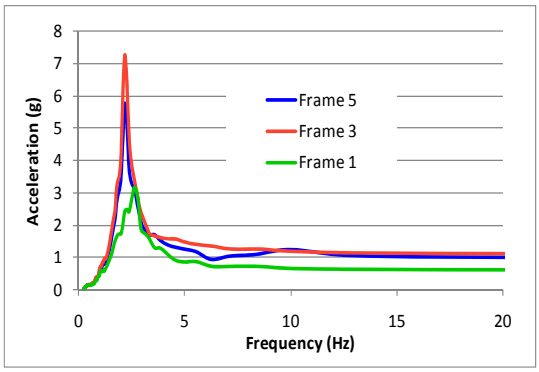


Figure 15 - Case study - Spatial variability of the floor spectra for the El-Centro earthquake

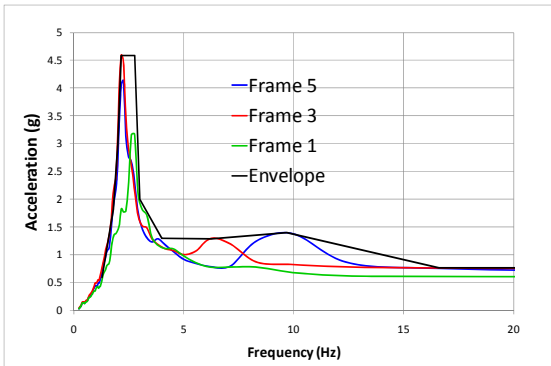


Figure 16 - Case study – Mean floor spectra for several frames and Envelope Floor Spectrum

A possible representation of floor spectra for the case-study is shown in Fig.16 as envelope of the mean spectra at several frames. This allows to account for the spatial variability of the action and its frequency content. This has been used to get the response of the pipes analyzed individually.

Time history analysis

A time history seismic input is rarely used for the design or retrofit of piping systems. Often it is used to generate facility specific response spectra analyses, or as a research tool, to study in detail the behavior of a component or system as a function of time.

Nowadays, the scientific community has widely accepted the use of natural records to reproduce a real input, for several reasons. For many engineering application, the purpose of selection and scaling of real earthquake is to fit the code design spectrum considering the

seismological and geological parameters of the specific site. To comply with the seismic codes set of accelerograms, regardless its type, should basically match the following criteria:

- minimum of 3 accelerograms should be used;
- the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of ag× S for the site in question (S is the soil factor, ag is the Peak Ground Acceleration);
- in the range of periods between 0.2 T_1 and 2 T_1, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied, no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be <90% of the corresponding value of the 5% damping elastic response spectrum.

To help engineers in selecting a proper set of records, some tools have been proposed in the literature. The most recent is REXEL proposed by Iervolino et al [18].

More rarely artificial or synthetic accelerograms are used. The first ones are generated to match a target response spectrum, the second ones form from seismological source models and accounting for path and site effects. For artificial signals, even though, it is possible to obtain acceleration time series that are almost completely compatible with the elastic design spectrum, the generated accelerograms often have an excessive number of cycles of strong motion, and consequently have unrealistically high-energy content. For this reason are less used than natural records. Moreover, to generate synthetic accelerograms there is a need for a definition of a specific earthquake scenario in terms of magnitude, rupture mechanism in addition to geological conditions and location of the site. Generally, most of this information is not often available, particularly when using seismic design codes. This representation of a seismic input is rarely used.

According to the above considerations it has been decided, at least for this preliminary analysis, to use natural records and to select only three accelerograms compatible with the spectra of Fig. 13. For the case study the vertical component has been considered as negligible, so only bi-directional motion has been adopted.

The three records were taken from Pacific Earthquake Engineering Research center database (http://peer.berkeley.edu), using the following hazard and compatibility conditions (Tab. 5). In Tab. 6 the characteristics of all the selected accelerograms are shown. Fig. 17 and 18 show the elastic spectrum of two records.

Magnitude Mw	**Soil conditions**	**Distance (Km)**	**PGA (g)**	**Lower bound**	**Upper bound**
6÷7	D	0÷20	0.24	-10%	+30%

Table 5 – Hazard and compatibility conditions for natural record selection

Event	Data	Mag.	Dist. (Km)	Filter (Hz)	name
Imperial Valley	19/05/1940	7.2	13.0	0.20	El-Centro
Northridge	15/01/1994	6.7	27.6	0.14	Santa Monica
San Fernando	02/09/1971	6.6	24.0	0.50	Old Ridge

Table 6 - List of the three selected natural records

After the identification of the accelerograms, the direct integration of the equation of motion or the integration of the single modal oscillators (time-history modal analysis) can be per-

formed. This latter is used when a limited non-linearity is present in the structure during the seismic event. Most frequently a direct integration scheme is used.

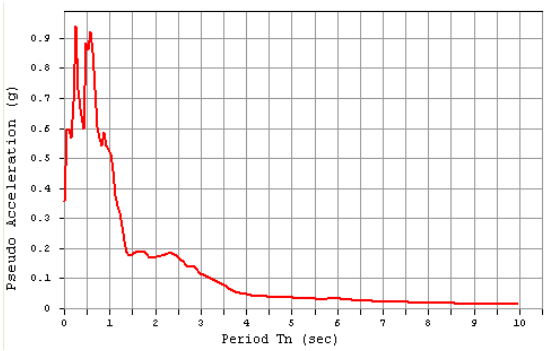


Figure 17 – Elastic spectrum of the El-Centro earthquake

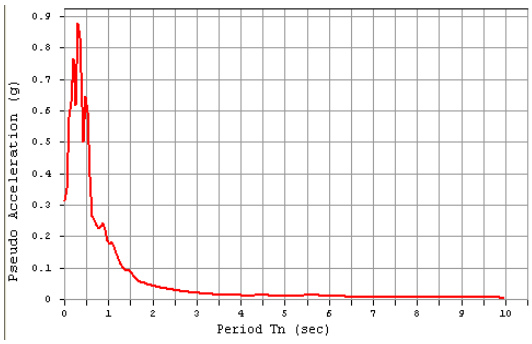


Figure 18 – Elastic spectrum of the San-Fernando earthquake

Differential movement effects

No specific indications are provided by the codes for the differential movement between two supports or between adjacent structures. Only ASCE-07 provides a simplified criterion based on the elastic analysis of the pipe-rack. Chapter 13.3.2.1 suggests evaluating the relative displacements between two connection points within the structure and at the same level using the differential movement for each vibration modes combined using appropriate modal combination procedures.

Unlike the cases of differential movements between adjacent pipe-racks connected by pipes, this effect on pipes within the structure is usually neglected. As observed above, the complete model, pipes + pipe-rack, allows to automatically accounting for this effect.

4.3 Calculation of the seismic response of the case study

The seismic analysis results of the analysed piping system using both static and dynamic methods previously described are illustrated herein.

For each analysis method the results in terms of moments are provided. In particular the maximum local moments M_y and M_z are provided for the straight pipes at each bay (1 to 7) and for the more critical elbows indicated in Fig. 10 with red circles (points A and B)

In Fig. 19 is shown, for the design conditions (PGA=0.24g, q=4, ξ=3%) the seismic response of the entire structure in terms of moments, shear and axial force for soil condition B and E, obtained using response spectrum analysis.

The maximum moment in the steel columns is about 220 kNm, very far from the ultimate moment of the steel elements. Similar results have been obtained by using T-H analysis. Therefore the support structure can be considered safe against the ultimate limit state. The soil conditions have also a limited effect both in the rack and in the pipes.

For the above observation, in what follows only the soil condition D, relative to the design conditions of the case study, have been considered.

In Tab. 7 the result of the seismic analysis of the case study of Fig. 1, in terms of bending moment in the pipes are shown. They have carried out at the several bays of the steel frame and at top floor and have obtained using all the aforementioned methods. In particular the values of damping (5%), behavior factor (q=4) and Importance factor (I_p=1.5) provides by the EN13480:3 standard and prEN1998:1 code have been used.

Firstly, a marked underestimation of the bending moment in the pipes using in-structure spectra is noticed. This is probably due to the limited mass of the pipes, which amplifies the relative displacement effects between the different support points of the pipes. In fact, by using the response spectrum method the bending moments have in mean double values.

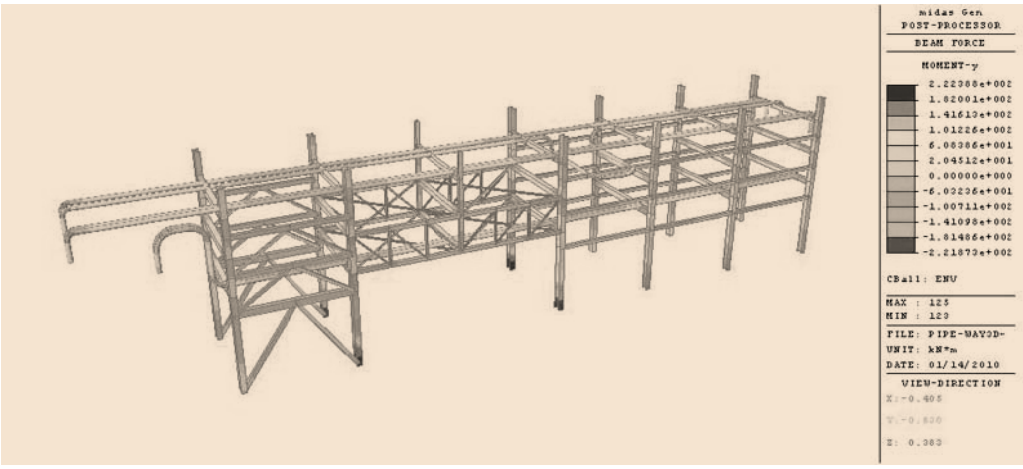


Figure 19 - Response Spectrum Analysis: Moment distribution for the Eurocode 8-1 Spectra with ξ=3%, ag=0.24g, q=4 and Soil D.

The use of dynamic analysis instead of the static one produces slight differences when in-structure spectra are used. This is due probably to the uniform distribution of the seismic force in the first case, whereas, in the second case, the distributions related to the several vibration modes is used. But substantially they provide similar results.

The time-history analysis on the elastic system produced greater values of bending moments. This is due to the absence of non-linear behavior of the structure. Because by applying to the response a reduction factor of 4, the results becomes not far to the results obtained with the response spectrum analysis and q=4, they should be considered reasonable. Further analysis will be dedicated to this aspect, modelling the analyzed structure in the non-linear range.

Method	Bay/Moment	1	2	3	4	5	6	7
Static analysis: In-structure spectra method	M_z	2.88	2.40	2.52	2.64	2.76	2.76	2.88
	M_y	1.56	1.56	1.20	1.32	1.44	1.20	1.44
Dynamic analysis: In-structure spectra method	M_z	2.10	2.30	2.30	3.10	3.30	3.00	2.30
	M_y	2.20	2.10	2.10	2.20	1.90	1.90	1.00
Dynamic analysis: Response spectrum method	M_z	5.30	5.50	5.50	3.50	2.90	4.80	5.80
	M_y	2.80	2.60	3.60	3.20	2.20	0.70	3.80
Dynamic method: Time-History analysis (elastic)	M_z	13.50	14.60	14.10	5.05	6.25	14.15	12.32
	M_y	7.30	6.65	4.45	4.65	3.05	1.70	3.85

Table 7 – Results relevant to the seismic analysis of the case study

4.4 Piping stress analysis and checks

One of the fundamental steps for the qualification of a pipe system is the fulfillment of some limits of the pipe stress or strain, for a given working condition. For a seismic action, usually two working conditions are considered:

- Operating condition or design basis earthquake condition: (OBE) is that earthquake which, considering the regional, local geology and seismology, could be reasonably expected to affect the site during the operating life of the plant. It is the earthquake during which the operating conditions of the plant can be still assured.
- Safe shutdown earthquake condition: (SSE) is the maximum ground motion for which some critical components of the plant must be designed to remain functional.

In order to evaluate the safety level stress-based or strain-based approach can be used. The first approach intends to evaluate the maximum stress in the pipes and the calculation is usually based on elastic analysis of the structure.

While stress based approach for pipelines is acceptable for a material with a well defined yield point and with a well defined yield ductility and strength, this design criteria becomes invalid when the stress in pipelines exceeds the limit under some displacement control loads, such as earthquakes and landslides [19]. In this case, strain based approach provides the design rule where the strain in the pipeline is allowed to exceed the specified yield strain provided that the safe operation can be ensured under displacement load. This method allows selected extensions to the stress-based design possibilities to take advantage of steel's well-known ability to deform plastically, but remain a stable structure. Codes and Standards are available for the strain based design approach, see [19] for reference. With the strain-based approach maximum strain in the pipes is calculated and compared with specified strain-limits related to limit states usually identified with buckling or ovalization of the pipes. Unfortunately, this approach needs of the calculation of the seismic response in the non-linear range. This is one of the reasons because the stress approach in more used.

Only EN13480 contains explicit indications for calculating the pipe stresses limits, considering both the above conditions (OBE and SSE), whereas ASME B31.3 indicates only occasional load conditions that can be identified as OBE condition.

For the verification of the pipes against earthquake, the allowable stress approach is usually adopted and will be also adopted in the following.

Load combinations and stress calculation

The response to seismic and other loads (sustained, thermal, pressure, etc..), have to be combined. European and American code prescribe similar combinations. In this respect the seismic load prescribed by the seismic codes (prEN1998:1 and ASCE-07) can be considered as an exceptional seismic action. Under this condition, usually Load and Resistance Factor design (LRFD) approach is adopted. If the allowable stress approach is used, the seismic action has to be reduced, as usual, of a certain safety factor, typically 1.4 (see ASCE-07).

Because the allowable stress approach is wildly diffused in piping design in the following section will be used for the calculation of the stresses and the verification of the safety level. In doing this, the ASCE-07 combination formula will be adopted.

ASME B31.3 does not provide an explicit equation for calculating the longitudinal stress, whereas EN13480 provides at point 12.3.3 the formula to evaluate the longitudinal stresses due to sustained, occasional and exceptional loads (e.g. the earthquake). This comes from the beam theory and includes internal pressure P and the resultant moments due to the sustained loads (MA) and the earthquake (MB):

$$\sigma = \frac{pD}{4t} + 0.75 \times SFI \frac{M_A + M_B}{Z} \tag{1}$$

where *SIF* is the stress intensification factor.
Because ASME B31.1 provides a similar expression of the stress, in the following this will be also adopted for ASME B31.3.
ASME B31.3 at point 319.4.4 indicates the way to calculate the resultant moments, similar to that of EN13480:3. In particular:

$$M_R = \sqrt{\left(SIF_y \times M_y\right)^2 + \left(SIF_z \times M_z\right)^2} \tag{2}$$

This definition has also been adopted for calculating the resultant moments using EN13480.
The resultant moment and the maximum stress in the pipes using the results of spectrum analysis have been calculated for both European and American standards. In particular, the maximum stress obtained is lower than 71 MPa in the straight pipes (bay 7) and 135 MPa in the elbows (B). The stress level here calculated is greater than that determined using in-structure spectra. In any case the stresses are limited and the piping system can be considered as over-designed.
Studies have shown that the present standards for piping system design under seismic loads are over conservative and modifications have been proposed to relax this over conservatism [20],[21]. For example, [21] has applied the present codes to calculate the stress limit on piping systems and experimental results showed significant discrepancy from the real behavior. This aspect will be treated in further studies.

Definition of the allowable stress
EN13480 includes par. 12.3.3., which is dedicated to the definition of the allowable stress calculation. They are defined according to the conditions previously recalled: Operating Basic Earthquake (OBE) and Safe Shutdown Earthquake (SSE). The basic inequality to be respected is:

$$\sigma \leq k f_h \tag{3}$$

where

σ = maximum stress in the pipes due to the sustained and seismic loads.
k = 1.2 for design basic earthquake
k = 1.8 for safe shutdown earthquake
f_h = is the basic allowable stress given by code

ASME B31.3 does not provide explicit differentiation between OBE and SSE but refers to a Design Earthquake that can be identified with the OBE. It is prescribed that the maximum stress cannot be greater than 1.33 times the basic allowable stress for pipes, indicated in the Appendix A of the same code.

Code	OBE	SSE
EN13480	165.60	248.40
ASME B31.3	183.50	---

Table 8– Pipes allowable stress (MPa) for the analysed case study

Tab. 8 illustrates the capacity of the pipes evaluated using both OBE and SSE seismic condition. The comparison capacity and demand show that the verification is satisfied with a different capacity/demand ratio ρ. In particular, for EN13480 ρ=1.39 whereas for ASME B31.3 is equal to 1.36. This values of ρ show, at least in the analyzed case, a slight difference between the two codes in the evaluation of the safety level.
The above calculations confirm that using allowable stress approach and then comparing the capacity with the demand (OBE), the system is highly overdesigned. The attempt of some researchers to modified the equation (1) [21],[22] is not enough fully exploit the plastic properties of pipes. In this sense the strain-based approach is more promising, especially if the Performance-Based approach would be used to design new pipelines or to assess the seismic behavior of existing ones. This is the reason that has induced to explore the plastic behavior of pipes and pipe-joints using the way of experimental investigation. This activity has been just started and in the following section some details of on-going tests on flanged joints will be provided.

5 EXPERIMENTAL INVESTIGATIONS

5.1 Previous experimental tests

A good number of experimental tests have been performed to characterize piping systems under seismic loading and they are available in literatures, e.g., [20],[21],[22],[23],[24]. It has been found that even strong seismic excitation is not severe enough to significantly damage piping systems unless there is large differential motion of the anchorage [20], and an extremely high level of seismic input is required to introduce damage in the components of a piping system [21]. Additionally, the use of isolation systems reduces seismic demands to the piping systems [22]. Experimental tests, e.g., [20],[23] clearly show that the present design criteria of pipeline under seismic loading are overly conservative and modifications are suggested [20].
Some experimental tests have also been devoted to bolted flange connections in order to assess their behaviour under external loading, e.g., axial and bending loadings [25],[26]. Results show that flanges with lower weights, e.g., a compact VERAX VCF flange joint, have the advantage over the flanges with higher weights, e.g., an ANSI flange joint [25].

5.2 Tests to be performed in the University of Trento

The University of Trento (UniTn) will perform some experimental tests on a number of bolted flanged joints (BFJ) in order to investigate the behaviour of the joints (BFJ) under monotonic and cyclic loadings. These tests are parts of the research works of a European project, INDUSE.
Instead of using conventional thicker joints, two different thinner joints have been designed for the tests due to the fact that thinner joints are expected to exhibit better performance for regular seismic events and for relatively lower working pressure. The two designed flanges have thicknesses 18 mm (Design 01) and 27 mm (Design 02), respectively and have been designed for a pressure of 15 bar. A matlab code has been developed by UniTn to check the joints according to the Eurocode EN 1591-1. One of the joints (Design 02) satisfies the Eurocode EN 1591-1 while the other (Design 01) does not. UniTn aims at performing these experimental tests on these thinner flanges in order to characterise their behaviour in real operating conditions and to check the validity of the design rules of the Eurocode for the flanged joints.

5.3 Test set-ups and programs

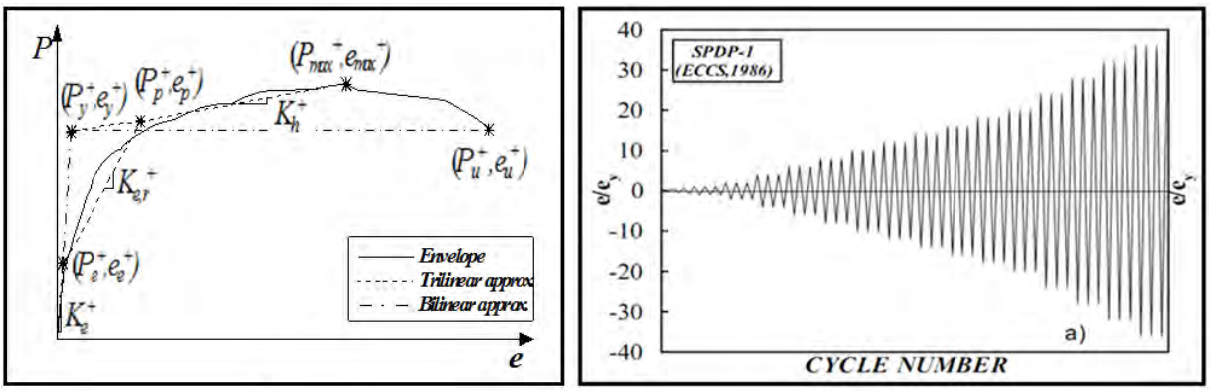
Eight experimental tests on BFJs will be performed by UniTn. The test programs and loading protocols are presented in Table 9. On each type of joint four tests will be performed, two of them will be in bending loading and the other two will be in axial loading. In both of the axial and bending tests, there will be monotonic and cyclic loadings. The cyclic loading will be chosen according to ECCS 45 loading protocols [27]. The monotonic and cyclic loading protocols are presented in Fig.21. The load in the BFJs will be applied via 100 ton actuators.

(a) (b)

Figure 20: Two plate-flanges designed by UniTn for the experimental tests: a) Design 01 for a flange thickness of 18 mm, and b) Design 02 for a flange thickness of 27 mm.

Design Code	Flange Type	Type of loading		No of Tests	
Non-Standard (Design Code-EN 1591)	Plate (Design 01*)	Axial	Monotonic	2	4
			Cyclic (ECCS 45)		
		Bending	Monotonic	2	
			Cyclic (ECCS 45)		
	Plate (Design 02*)	Axial	Monotonic	2	4
			Cyclic (ECCS 45)		
		Bending	Monotonic	2	
			Cyclic (ECCS 45)		
Total No of Tests					**8**

Table 9: Number and types of tests on bolted flanged joints to be performed by UniTn



a b

Figure 21: (a) Monotonic and (b) cyclic loading protocols for the tests on bolted flanged joints.

The test set-ups for bending and axial tests are presented in Fig. 22 and 23 respectively while the real test specimens are shown in Fig. 24 and 25.

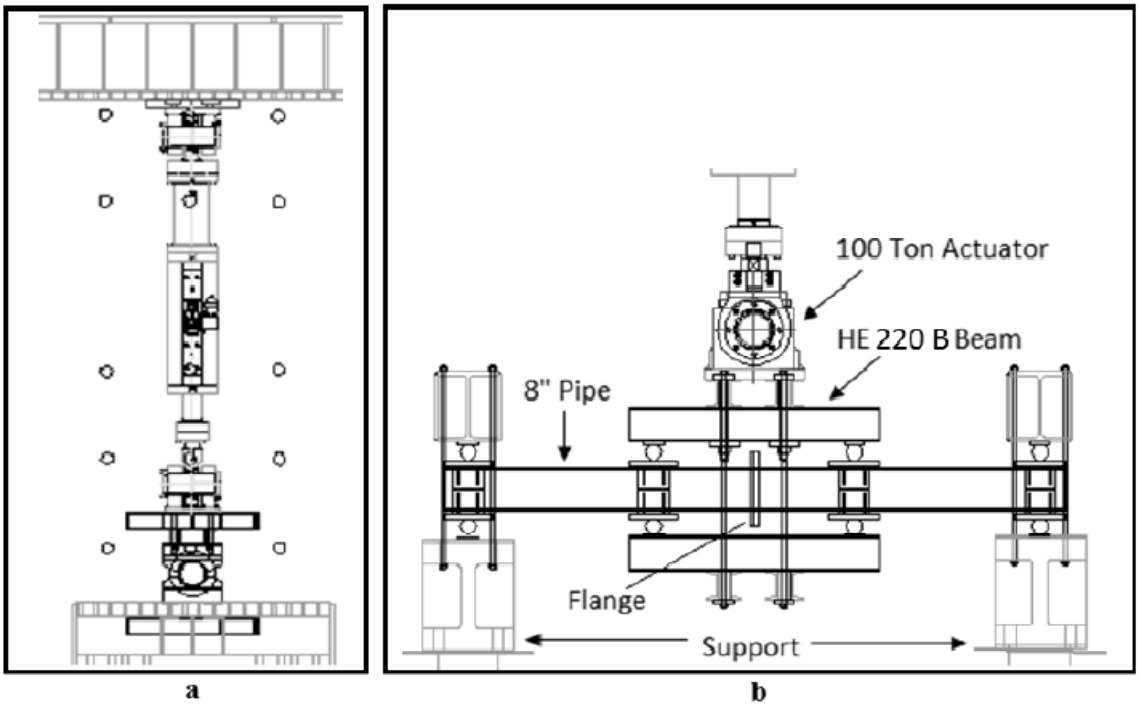


Figure 22: Set-up for bending tests: (a) front view; (b) side view.

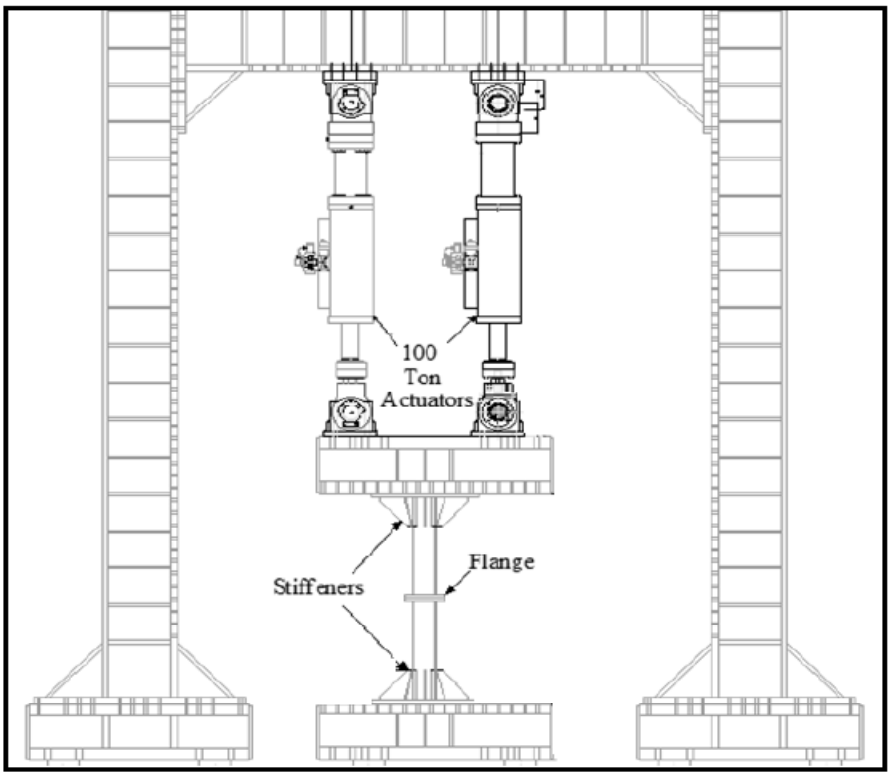
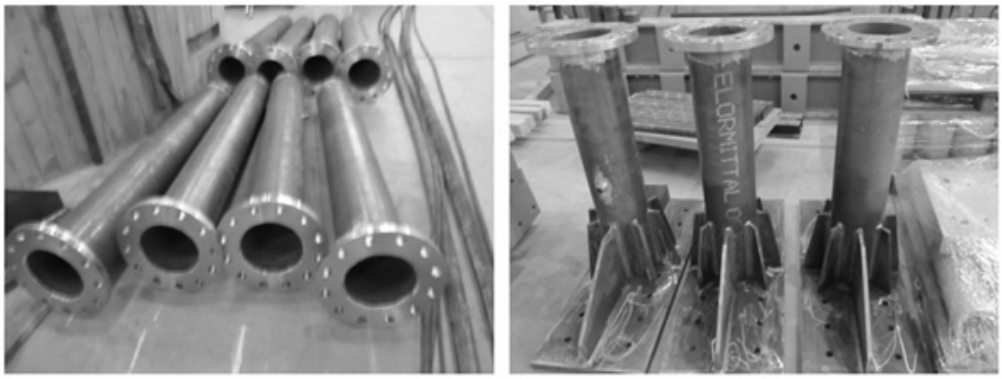


Figure 23: Set-up for axial tests.



a b

Figure 24 Test specimens for: (a) bending tests; (b) axial tests.

a b c

Figure 25 : (a) Flange with 18 mm thickness (Design 01); (b) flange with 27 mm thickness (Design 02); (c) a gasket

6 CONCLUSIONS

The seismic analysis of piping systems is very different from the analysis of like-building structures for which a long experience has permitted to have enough provisions for obtaining a reliable design against earthquakes. Therefore, the present contribution tried to clarify all the design steps of this type of structures and to identify all the aspect that need to be clarified. This was made through a representative case study analysed according to both European (EN13480:3) and American standards (ASME B31.3) devoted to piping systems. The comparison between EN13480 and ASME B31.3 standards showed that the two codes provide similar indications for the evaluation of the seismic response of pipelines supported by a pipe-rack. From the analysis of the results the following conclusions, considered valid for both the standards, can be drawn.

ASCE-07 provides a simple rule to establish when the dynamic coupling between pipes and supporting structure need to be considered. It is based only on a weight ratio. Actually, other ingredients should be included; for example the vibration periods range of the pipe. In the analysed case the single pipe has a limited weight and therefore short vibration periods. This means that, as already shown, the dynamic coupling between pipes and the pipe-rack is limited; whereas the relative displacements between the pipe supports would represent a more relevant effect. The pipes provide a sort of static coupling between the transverse frames. In fact, the models with and without the pipe stiffness contribution have more or less the same period, whereas the excited mass is differently distributed between the several vibration mod-

es. Therefore, the consequence of considering the structural contribution of the pipes is the introduction of a sort of a rigid floor effect that couples the horizontal movements of the nodes of the same floor. For the above-mentioned reasons an extensive analysis on this particular but important aspect is recommended.

The in-structure spectra suggested by prEN1998:1 and ASCE-07 differs for a term that depends on the pipe-structure frequency ratio, included only in the European code. This aspect is strictly related to the previous observation on the dynamic coupling. The exclusion of the dynamic interaction could introduce a high error in the evaluation of the amplification factor of in-structure spectra. Because usually the frequency of the piping system is relatively high with respect to that of the supporting structure, the amplification effect would be relatively low. In any case, to cover all the possible cases, the use of the European formula for in-structure spectra is highly recommended. The result of this investigation suggested at point 1, should also help to clarify this important aspect toward the direction of simplified but more realistic formulas for in-structure action on pipes.

In the analysis both beam element and shell elements were used to model elbows. The comparison showed the reliability of the more simple elements based on the beam theory, at least for the simple case of elbows. For more complicated situations, the use of finite shell elements should be considered.

The behavior factor q is usually indicated by standards. In particular the values provided for piping systems by ASCE-07 seems to be too high, i.e., q=6-12. This is in contrast with the idea of avoiding yielding phenomena in the pipes and to maintain operational conditions also in case of strong seismic events. Certainly, the contribution of the supports in providing dissipation capability should be taken into account; however on the basis of several investigations found in the literature, this exclude the possibility of having so high values of q. For this reason the next step will be an investigation on the reduction factor q also considering the non-linear behaviour of pipes. This could be performed using non-linear beam elements for the straight pipes, and plastic hinges for the simulation of the elbows in the plastic range. In doing so, time-history analysis would be used, even if static non-linear analysis is nowadays very diffused, but difficult to be applied to piping systems for the presence of a uniformly distributed mass of pipes.

The usual way of designing piping systems is based on the allowable stress approach. This means that the structure is considered elastic. Consequently only an operating basis earthquake condition (OBE) can be taken into account for design. Conversely, the modern approach to the seismic design of structures is to differentiate serviceability from ultimate limit states. This latter condition could be represented, for example, by the Safe Shutdown Earthquake (SEE). The problem is that a proper definition of the deformation capability of the pipes and fittings (ovalization, etc..) in the plastic range would be necessary, and on this aspect the research has not reached solid conclusions. But the advantages would be several and first of all the design optimisation. For example in the analysed case, we are very far from the moment capacity of the bolted flanges located in the pipeline. This high level of conservatism seems to be in contrast with the modern performance based-design approach, for which a certain level of yielding in the structure is admitted according to a specific performance. For instance, it would be possible to accept in pipes a certain level of yielding on the condition that leakage would not occur, and the consequence, for example, to accept a reduction of the thickness of the flange. The experimental campaign, briefly described in the paper and that will be performed at the University of Trento in order to evaluate the cyclic behavior of flanged joints, will provide useful information for the design of the flanged joints in the more proper way. Moreover, it should allow one to link the capacity and the demand for several limit states.

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