Post-Emergency, Multi-Hazard Health Risk Assessment in Chemical Disasters

PEC

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Definition of the structural models and seismic fragility analysis techniques available for the specific case study
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1 INTRODUCTION

Scope of this document is to describe the seismic analyses developed for the elements at risk included in the Plant A and Plant B. In order to focus the attention on the elements that stored the biggest quantity of material, the analyses reported herein are relative to: tanks, horizontal vessels and vertical vessels. Analyses reported for vertical vessels are deemed to be representative also of other slender vertical elements such as distillation columns and reactors. The document describes the analyses developed to derive fragility curves for the elements listed above: the intention of the report is to obtain frequencies and consequences of the different damage states to be used in further risk analyses in the project but also to propose methodologies for seismic fragility curves evaluation that can be used in other applications of seismic assessment of petrochemical plants.

1.1 TANKS

After a seismic event, in order to evaluate the amount of material stored in the tanks that spills into the environment, the first element required are the fragility curves elaborated for these structures. The fragility of the storage tanks has been described as the probability of reaching or exceeding a certain damage limit condition for a given severity of the ground shaking. Fragility curves have been derived selecting as representative parameter of the ground motion severity the Peak Ground Acceleration (PGA). The curves produced in this study are mechanics based and have been calculated through a simplified model, which describes the population of structures by means of random variables. Fragility curves have been calculated for different types of storage tanks. The sub-classification of the storage tanks under consideration has been done posing particular emphasis on their geometric characteristics. Therefore, to each storage tank of the test area a type and a set of fragility curves has been linked. Fragility curves have been validated by the comparison with curves published in the technical literature and by a sophisticated mechanical FE model.

1.2 HORIZONTAL AND VERTICAL VESSELS

Also for horizontal and vertical vessels the first element required to define material stored in vessels that spills into environment after a seismic event is the fragility curves elaborated for these structures. In particular in this document are presented fragility curves for horizontal and vertical vessels. Fragility curves have been derived selecting as representative parameter of the ground motion severity the Peak Ground Acceleration (PGA). The curves produced in this study are mechanics based and have been calculated through a simplified model of the structures described by means of random variables. Fragility curves have been evaluated with simplified but still reliable models in order to allow the engineers to replicate the analyses for a large variety of equipments. Practice oriented seismic vulnerability approaches have been proposed in this study. Fragility curves have been calculated for different classes of vessels. The sub-classification of the vessels has been done on the bases of geometric dimensions.
2 TANKS

2.1 DAMAGES OBSERVED IN PAST AND RECENT (EMILIA) EARTHQUAKES

2.1.1 Damage to tanks in past earthquakes
As described in Beilic (2013), the susceptibility of storage tanks to earthquake damage has been demonstrated in numerous seismic events occurred in Alaska (1964), Japan (Nigata 1964, Kobe, 1995) and California (San Fernando 1971, Imperial Valley, 1979, Loma Prieta 1989, Northridge, 1994).

Different types of damages or failure have been observed during the past earthquakes for steel tanks; the more common ones are showed in Figure 2-1 and listed below:

a) Elastic-plastic buckling of the tank shell (“Elephant foot” buckling)
b) Elastic buckling of the tank shell (“Diamonds shaped” buckling)
c) Failure of the piping connected to the tank
d) Roof damage due to the convective wave
e) Secondary buckling at the top of the shell
f) Settlements of the foundations
g) Toppling around a corner at the base
Common forms of damage for steel tanks involve buckling of their shells. The “Elephant foot” buckling (Figure 2-2) involves outward displacements of the bottom shell courses; experimental studies (Niwa & Clough, 1982) demonstrate that this form of shell buckling is induced by the combined action of circumferential tension near the plastic limit of the material and of vertical stresses of compression. This form of buckling is more frequent in full tanks with high aspect ratio (height/diameter), however also squat tanks and containment structure only partially filled suffered damages in past earthquakes because of “Elephant foot” buckling. Typically, the “Elephant foot” buckling extends over a large portion of the circumference of the tank. Sometimes the buckling of the lower course of the shell led to loss of the tank content, because of weld or piping failure and in some cases has caused the collapse of the entire structure.

![Figure 2-2: “Elephant foot” buckling occurred after the 1964 Alaska earthquake (Karl V. Steinbrugge Collection, University of California, Berkeley)](image)

**b) “Diamonds shaped” buckling**

A different form of buckling has been recognized for tanks with very thin shells, such as the stainless steel reservoirs that are commonly used for foodstuffs like wine, milk or beer. This form of buckling is called “Diamonds shaped” buckling (Figure 2-3) since involves the formation of diamond shaped buckles in the shells. The buckles, usually, occur at a certain distance above the base of the tank because the elastic (“diamond”) buckling is caused by high vertical stresses and low circumferential tensions acting simultaneously (Niwa & Clough, 1982).
c) Piping system failure

The failure of the piping system of the tank (Figure 2-4) is one of the most common causes of loss of their content during earthquakes. The fracture of the piping is mainly caused by the large settlement, the buckling of the shell or the uplift of the tank wall that determine unsustainable vertical displacements for rigid piping systems. Failures of piping connecting adjacent tanks have been also observed in past earthquakes because of the relative horizontal displacements of the tanks. These types of failure can be avoided protecting brittle elements such as the valves of the piping system and providing sufficient flexibility in other elements of the system.

Figure 2-4: Failure of the piping system occurred during the 1987 Edgecumbe (New Zealand) earthquake (Edgecumbe archive, EERC Berkeley)

d) Roof damage

The roof damage (Figure 2-5) is another common form of damage for steel tanks subjected to earthquakes. The ground excitation induces the sloshing motion of the fluid contained in the tank; the amplitude of the liquid waves in vertical direction along the tank circumference has been estimated, based on some observations, to have exceeded several meters during some seismic events. For tanks that are near to be full the fluid sloshing motion could result in an upwards pressure distribution acting on the roof. This pressure, frequently, lead to failures of joints between the roof and the shells of the tanks.
e) Secondary buckling of the top shells

In squat tanks with low height to radius (H/R) ratios, a secondary buckling effect has been recognized at the top of the shells (Figure 2-6). This form of buckling is caused by the suction that could occur within the tank at the top because of the sloshing motion of the water. The damages observed induced by this form of buckling are similar to those caused by an external wind pressure.

f) Settlements of foundation and failure of welds at the base

Petrochemical tank storage farms are frequently sited close to harbors or rivers because their content is mostly transferred via sea with tankers. These sites consist of cohesionless soil with liquefaction potential and this lead to poor foundation condition for the tanks. Base rotations and large settlements have been observed for tanks after many earthquakes. These settlements (Figure 2-7), sometimes of the order of meters, are mainly induced by the combined action of the liquefaction of the soil beneath the tank and of the lateral accelerations on the structure. The lateral accelerations induced on the tank by the seismic action result in huge base overturning moment and in large uplift displacement on the tension side of the tank base. As a consequence, the hydrostatic pressure of the tank content holds down the base plate and counteracts the uplift. Because of this combined action, the base weld is subjected to large stresses that may result in the collapse of the connection and in the spillage of the tank content.
g) Toppling around a corner at the base

The last typology of failure observed is the toppling of the storage tank around a corner at the base of the structure. In the toppling assessment the storage tank is considered to be a rigid body that topples around a corner at the base.

2.1.2 Field observation of failures for Emilia earthquake

As described in Brunesi et al. (2015), on May 20th and 29th, 2012 two earthquakes of magnitude (Mw) 6.11 and 5.96 hit the Emilia region in the Po Valley, one of the most industrialized zones of Northern Italy. The majority of structures severely damaged by the seismic events were industrial facilities: one-story precast reinforced concrete structures (Magliulo et al. 2013; Bournas et al. 2013; Belleri et al. 2014) and nearby storage steel tanks.

The poor seismic performance provided by storage steel tanks, as a consequence of lacks in the past Italian design practice and in early design in general, confirms the high vulnerabilities of these structures as already shown by other destructive earthquakes in other areas (Manos 1991; Niwa and Clough 1982; Swan et al. 1985; Stepp et al. 1990; Zareian et al. 2012; González et al. 2013).

The most common types of failures observed were fracture of anchors and “Elephant foot” buckling near the base of the tanks. Generally, “Elephant foot” buckling was experienced in squat tanks, while some of the slender tanks surveyed developed “Diamonds shaped” buckling. Total and partial collapse of legged tanks was also commonly observed, induced by shear failure and/or buckling of their legs due to axial forces resulting from tank overturning moment. In some cases, flat-bottomed steel cylindrical tanks, typically larger than legged tanks, failed in tension at the bottom of the tank wall, in correspondence of the anchor rods or massive concrete pads.

Some of the failure modes encountered during the surveys are shown in Figure 2-8. In particular, Figure 2-8(a) shows “Elephant foot” buckling and Figure 2-8(b) shows “Diamonds shaped” buckling. This damage pattern, less common than the previous, occurs at small hoop stress levels and, hence, is particularly sensitive to internal pressure and imperfection amplitude: the buckling strength decreases, as the former reduces or as the latter increases. A less common collapse mechanism, not yet included in modern Codes, is documented in Figure 2-8(c), where an example of the earthquake-induced secondary diamond-shaped buckling, observed in the May 2012 sequence, is shown. In this case, the elastic mechanism, due to an effective inward acting pressure, is uplifted in correspondence
of the circumferential welds, where the amplitude of the imperfections are larger and their effects, concomitantly with wall thickness reduction along the height, more visible.

Base-anchorage failures in flat-bottomed systems are collected in Figs. 5(d), (e), and (f). In particular, these mechanisms are usually associated with the elastic diamond-shaped buckling of the tank wall, as presented in Figure 2-8(d). In many cases, excessive inelastic strain demands took place in the anchor bolts, causing their fracture or debonding from the concrete pads. Figure 2-8(e) and (f) show other two anchoring system-related modes, encountered during the surveys; the former shows the spalling of concrete, induced by insufficient distance between the anchor bolt and the edge of the foundation and low resistance of the concrete, while the latter presents a flexural failure occurred in the anchor plates. Hence these systems, poorly anchored and detailed to sustain earthquake-induced demand, collapsed because of lack of proper steel reinforcement around the anchor and inadequate resistance at the concrete foundation. Failures were induced by sliding and rocking of the tank, and they were observed to occur in the weakest link of the anchoring system.

![Figure 2-8: Observed failure mechanisms: (a) elephant’s foot buckling; (b) diamond-shaped buckling; (c) secondary diamond-shaped buckling; (d) combined diamond buckling and failure of anchors; (e) concrete spalling at the anchorage; (f) bending of anchor plates; (g) shear-buckling of a leg; (h) sliding of an unanchored system](image)

The leg-supported tanks, usually smaller than flat-bottomed systems, mostly failed in their legs if anchored, or they were observed to slide when unanchored. An example of these two behaviors is shown in Figure 2-8(g) and (h), respectively. In particular, Figure 2-8(g) presents a typical case of damage by combined shear and buckling of the tapered tank legs. The coupling between heavy static loads and the effect of horizontal and vertical seismic excitation, resulting into prominent stress/strain concentrations at the base of the stocky legs, where the area and moment of inertia of the section are lower, induces this mechanism.
Therefore, the leg slides, after failure of the anchorage, and then visibly buckles, losing its verticality; the rest of the tank does not exhibit any appreciable deformation and, hence, the seismic performance of this system typology is mainly governed by the response of its legs and anchorages.

On the other hands, unanchored legged systems performed considerably better; no damage in the tanks or falling off their foundations was detected, due to a combination of rocking and sliding. If the legs were strong enough to rock and the piping system was flexible enough to accommodate the seismic displacement demand, a sort of safe base-isolation mechanism is provided. In the majority of the cases, displacements of the order of about 10 cm were measured, as shown in Figure 2-8(h) and roughly in accordance with the spectral demand given in Figure 2-8(b).

2.2 INTERNATIONAL CODES

In the vulnerability assessment of the storage tanks the definition of their structural properties is not very difficult since storage tanks are highly standardized in terms of construction and design. After an analysis of the international codes is noted that the design of these structures evolved very slowly over the course of the past decades. For this reason, the tanks are very similar around the world and their vulnerability is almost independent from the construction period.

For the structural design of the steel storage tanks, the worldwide references are essentially 4 standards: 650 the American Petroleum Institute (API 650), the American Water Work Association D100 (AWWA 100), the Eurocode 3 (EC3 in the following) and the Eurocode 8 (EC8 in the following). The use of US standards is much consolidated because they were introduced in the 30s.

The difference from the American codes to the Eurocodes is that the first use a design based on very simple empirical methods. In particular, the main differences for the static design are:

- The American codes use empirical rules for defining the thickness of the tank components, while the Eurocodes define the design rules based on the shells theory;
- The American codes use the method of the admissible tensions, while Eurocodes refer to the limit states method.

In the following paragraphs the seismic design for the American codes and Eurocodes are briefly described.

2.2.1 Seismic design of the storage tanks according to the American codes

The response of the structure for a given severity of the ground shaking is assumed as a combination of two components:

- The amplified high-frequency response of the fundamental vibration mode of the tank and of the liquid part that moves jointly to it (impulsive mode);
- The amplified low-frequency response of the liquid part that moves according to its fundamental period (convective mode or “Sloshing” mode).

As an example, the relation suggested by API 650 for the definition of the fundamental period of the Sloshing mode is given below:

\[ T = k \cdot D^{0.5} \]

where D is the diameter of the tank and k is a coefficient which can be defined as a function of:

- the ratio height/diameter of the tank;
- the lateral stress coefficients to be applied to the tank;
• the portion of liquid which moves with the tank;
• the mass of liquid which generates the Sloshing.

The last problem is defining what portion of fluid moves together with the tank (impulsive mass) and which participates in the Sloshing motion (convective mass). In general, the standards provide relationships, tables or curves that define the portion of impulsive and convective masses and the height of their center of mass, useful for evaluating the moment applied to the base of the tank.

The seismic design should take into account: (i) the stability of the tank when the axial forces increase in the mantle as a consequence of the overturning moment and the vertical component of the seismic action, (ii) the increase of the liquid height in the tank as a result of the Sloshing.

2.2.2 Seismic design of the storage tanks according to the Eurocodes

For the design of the storage tanks the Eurocodes refer to the following definition of limit states:

• **Serviceability Limit State**: under the design seismic action the tank seal should not be compromised as well as the running of the systems connected to the tank such as pipes. The local buckling should not cause irreversible damage;
• **Ultimate Limit State**: under the design seismic action the sliding or the overturning of the tank as a rigid entity should not occur. The nonlinear behavior should occur locally rather than globally. Loss of content should not occur. The foundation can show an overstrength with respect to the tank structure.

When the tank can be considered as rigid, the Laplace’s equations for the motion of the fluid inside the tank can be expressed by the sum of two contributions: motion of the rigid-impulsive mass and motion of the convective mass (sloshing mode). In case of dynamic analysis, the contributions due to the inertia of the two masses are added. If, instead, a response spectrum approach is adopted, in the combination of the modal contributions it should be noted that the two modes have very different periods of vibration. Therefore, the combination according to the rule of the square root of the sum of squares (SRSS) could not be conservative and for this reason the EC8 suggests to sum the peaks of two modal contributions.

When the tank cannot be considered rigid, as this is the case of steel tanks, a third term to define the fluid motion must be added. The impulsive mass creates a rigid-impulsive motion and a deformation motion. The third component may also be determined independently from the others. In order to evaluate the mass components for each mode, also in the Eurocodes tabulated functions that for the convective component and for the rigid-impulsive component are provided. These tables match perfectly with those of the American codes, because come from the solution of the same equations.

Lastly, in the Eurocodes more emphasis to the soil-structure interaction is given.

2.3 FRAGILITY CURVES

The storage tanks have been sub-classified as a function of the ratio between diameter (D) and height (H), D/H, since such a ratio has the highest influence on the seismic performance of storage tanks on the bases of the damage observed during past earthquakes (Eidinger 2001). The storage tanks of the industrial plants herein considered have been sub-classified as follows:

• class 1: $0.7 \leq \frac{D}{H} \leq 1$;
• class 2: $1 < \frac{D}{H} \leq 1.5$;
• class 3: $1.5 < \frac{D}{H} \leq 2$;
• class 4: D/H > 2.

The following damage mechanisms are taken into account in order to assess the vulnerability of the storage tanks:

✓ The buckling of the wall as a consequence of elastic or elasto-plastic mechanism known in the technical literature as “Diamonds shaped” buckling or “Elephant foot” buckling”, respectively;
✓ The toppling of the storage tank.

The aforementioned mechanisms are the ones that more frequently have been observed in the storage tanks when subjected to earthquake. In the toppling assessment the storage tank is considered to be a rigid body that topples around a corner at the base. The assessment for buckling of the tank walls has been analysed using the simplified method proposed by Malhotra et al. (2000), which is one of the methods suggested by the Eurocode 8 (1998). Since the compression exceeds the buckling stress before the tension reaches the resistance of the welded connection between wall and bottom plate, by controlling the buckling the danger of leaking of the liquid stored is preserved. When assessing the buckling of the tank wall, it should be considered that when anchorage is not in place, the compression in the wall could be considerably increased by the tank uplifting from the foundation. Therefore, in the buckling assessment, the increase of compression due to the unanchored tank uplift is taken into account according to New Zealander regulation (Priestley et al. 1986) that follows the method proposed by Clough (1977). In the fragility curves herein considered the collapse mechanism due to differential displacements between the wall and the pipes is not taken into account. This damage condition has to be neglected because it is strongly dependent on the relative stiffness of the connection wall-pipe which does not follow a standard, and therefore a suitable distribution through a random variable cannot be found. The fragility curves produced in the current study are presented in Figure 2-9 for the case of tanks not anchored on the ground with (a) soil characteristics representative for the site and (b) rock, i.e. without soil-structure interaction. Figure 2-9 shows that the curves are not particularly different for the two soil conditions as the soil of the area is still hard, with an average value of Vs around 600 m/s². However, for the tanks on the soil, for which the effects of soil-structure interaction are taken into account, the vulnerability is smaller, as the soil-structure interaction is a phenomenon that produces an increase of the periods towards the branches of minor amplification of the spectrum and a higher amount of energy is dissipated.

![Figure 2-9: Fragility curves proposed in this study for unanchored tanks a) on rock and b) on soil](image_url)
For the comparison with the curves coming from literature, the curves on rock are selected. Figure 2-10 shows the comparison between the fragility curves calculated in this study and the ones proposed in Hazus (FEMA 1999) for the limit state of moderate damage (MD) and of severe damage (SD), that here correspond to the activation of the buckling of the tanks. Hazus (FEMA 1999) is a project in which mechanic based methodologies have been adopted to define fragility curves for a large number of structural typologies. From the comparison can be observed that there is a good agreement between the fragility curves produced in this study and the ones suggested by Hazus (FEMA99). A further validation of the results of this study is that the fragility increases when the ratio D/H reduces and the evidence of past earthquakes confirms this trend, where higher level of damage has been observed on slender structures.

The tanks mechanical model presented in this study for the curves generation allows only the calculation of curves for moderate/severe damage, since no data are available to quantify the activation of other failure mechanisms.

The curves corresponding to other damage limit states have been generated calculating the mean and the standard deviation parameters as specified below:

- The calculated curve is assumed to be corresponding to the limit state of severe damage;
- The average of the curves relative to other limit states was calculated preserving the ratio between the average values of the curves proposed in Hazus (FEMA99);
- The standard deviation was calculated assuming for each limit state the same dispersion of the curve really calculated. This choice is consistent with what can be observed in the curves proposed by Hazus (FEMA99), which have very similar coefficient of variation for each damage limit state.

![Fragility curves proposed in this study and Hazus (FEMA99) for the limit state corresponding to the activation of the mantle Buckling for tanks a) anchored and b) not anchored](image)

Figure 2-10: Fragility curves proposed in this study and Hazus (FEMA99) for the limit state corresponding to the activation of the mantle Buckling for tanks a) anchored and b) not anchored

As an example for non-anchored tanks, Figure 2-11 and Figure 2-12 show the fragility curves produced for each limit state and for all the classes in which the tanks were classified, respectively for the case of rock foundations and soil foundation, with characteristics generated based on the mechanical properties of the soil of the area. The information on the anchor conditions is not known, therefore the tanks are assumed as not anchored, which corresponds to the most common practice.
Figure 2-11: Fragility curves for unanchored tanks proposed in this study for each limit state and for each identified class for tanks on rock.
2.4 MECHANICAL MODEL VALIDATION

Several computational strategies, either based on high-definition finite element models or simplified mechanical idealizations, can be easily used to reproduce the complex response of industrial storage structures, whose behavior is governed by a combination of many interacting phenomena. A brief overview of the prevailing contributions is given hereafter, aiming at providing the rationale behind selection and definition of suitable prototype systems.

The ground shaking that excites the base of a tank during an earthquake causes a change in the set of pressures exerted by the contained liquid on the tank itself, pressures which are thus visibly different from those corresponding to the static condition. Obviously, distribution and intensity of these hydrodynamic pressures are function of earthquake intensity, geometry of the tank and type of liquid contained. For a horizontally excited upright containment structure, a portion of the fluid close to the upper free surface tends not to displace laterally with the tank walls, experiencing vertical sloshing: the arising of fluid waves associated with “convective” modes of vibration. Nearer the floor base and along the walls at the bottom, the fluid is unable to move out of the way as the tank displaces and moves synchronously with the wall acting as an added mass rigidly attached to the wall. This mass with the inertial mass of the tank contributes to the “impulsive” mode of the system. The proportion of the two masses of fluid contributing to the two vibration modes depends on the geometry of the tank: height of the free surface of the fluid and the radius. The portion of fluid that acts as a convective wave increases as tank aspect ratio (height/radius) decreases, with the impulsive mode becoming more predominant.

Accordingly, two different geometrical configurations were considered to investigate the response of vertical anchored cylindrical steel tanks that represent one of the most common form of storage structures in the Italian scenario. In particular, the slenderness ratios H/R was assumed as reference parameter in order to identify different behavioral aspects for this type of system. As such, the behavior of both squat tanks (0.5 < H/R < 1.5) and slender tanks (1.5 < H/R < 6) was explored. A resume of their peculiar characteristics is given below, whilst further and more detailed information can be found later on. Figure 2-13 and Figure 2-14 show the two prototype system under study.

Figure 2-12: Fragility curves for unanchored tanks proposed in this study for each limit state and for each identified class for tanks on soil
• Case-study 1 is a vertical cylindrical tank made of steel. The radius is \( R = 15 \) m and the height \( H = 15 \) m, therefore the tank aspect ratio is \( H/R = 1 \). The ratio of radius \( R \) to the thickness \( t \) is equal to \( R/t = 750 \).

![Figure 2-13: Case-study 1, squat tank \((H/R = 1)\)](image)

• Case-study 2 is a vertical cylindrical anchored tank made of aluminum. The radius is \( R = 1.18 \) m and the height \( H = 4.72 \) m, therefore the tank aspect ratio is \( H/R = 4 \). The ratio of radius \( R \) to the thickness \( t \) is equal to \( R/t = 500 \).

![Figure 2-14: Case-study 2, slender tank \((H/R = 4)\)](image)

High-fidelity models were thus developed to predict the dynamic response of the two case-study structures, which were tested in past experimental programs. Comparisons between experimental and numerical estimates may serve as a validation of this modeling approach, which were then used to
perform a set of nonlinear dynamic analyses aimed at exploring behavioral changes in the seismic response of similar structures as a consequence of parametric variations in their geometry. Those findings were finally used to correlate estimates resulting from detailed and simplified modeling approaches. A review of well-known mechanical/phenomenological techniques was provided in the following as a background of more advanced numerical procedures.

2.4.1 A review of mechanical mass-spring analogies

Earthquake-induced actions produce a change in shape and intensity of pressure distributions acting on the tank wall. The so-called hydrodynamic pressures vary with time and cause a significant variation in the internal actions recorded in the shell. When a ground supported cylindrical tank is subjected to a base excitation, the fluid in its upper portion tends to displace vertically and not horizontally (see Figure 2-15). This type of displacement incompatibility that occurs between the fluid and the tank wall results in the development of fluid waves associated with “convective” modes of vibration. Such a behavior is known as sloshing.

![Figure 2-15: Sloshing wave and pressures induced on above-ground cylindrical tanks](image)

The vibration period of the first mode convective wave is usually quite long, and is mainly function of the aspect ratio of the tank (i.e. height of the fluid in the tank normalized by its radius). The portion of liquid closer to the bottom of the tank tends to move in unison with the shell and behaves as an additional mass attached to the tank walls. This mass contributes to the “impulsive” mode of the system, in combination with the inertial mass of the tank itself. In slender tanks characterized by large aspect ratios, the impulsive component is more significant than the convective one, while the convective component is the most pronounced one in shallow or broad tanks having low aspect ratios. More in general, the portion of fluid that acts as convective wave increases as tank aspect ratio decreases; for very broad tanks only 30% of the tank content behaves as impulsive mass. It must be also noticed that, in tanks completely filled and closed at their top, convective waves cannot develop, and hence the entire fluid mass behaves according to the impulsive mode (Priestley et al. 1986).

The hydrodynamic pressures related to these two types of vibration have different variations in time because of different vibration periods and have a different distribution up to the height of the tank. The hydrodynamic pressures associated with the impulsive component reach their maxima close to the base of the tank and decrease up to the free surface of the fluid. The variation with time is characterized by high frequency vibrations caused by small vibration periods that are usually associated
with the impulsive mode. By contrast, long periods associated with the sloshing mode result in low-frequency vibrations of the convective pressure distributions. These pressures show their maxima at the liquid surface and decrease along the depth. Shape and amplitude of both pressure distributions depend on the aspect ratio of the tank.

Mathematical models were derived in the past to represent the behavior of fluid contained in a tank under dynamic excitations (see e.g. Jacobsen 1949; Yang 1976; Haroun 1980), and they constitute the theoretical background of seismic design methods currently implemented in different codes and standards. Classical potential flow theory can be thus integrated with the use of Fourier series, and advantage can be taken of Bessel functions. Despite this, mechanical models appeared to be an attractive method of analysis for such liquid-filled systems due to the complexity of the problem. One of the first mechanical models was suggested by Housner (1963), considering rigid-walled cylindrical tanks. Convective and impulsive hydrodynamic pressures induced by seismic action on the rigid tank wall were computed and an equivalent spring-mass system able to develop the same forces and moments on the tank was proposed.

This model was then generalized by Wozniak and Mitchell (1978) distinguishing between squat and slender tanks. Similar results were obtained by Veletsos and Young (1977). To take into account the wall flexibility, improvements of the aforementioned models were proposed (Haroun and Housner 1981; Veletsos 1984) adding a further degree of freedom. Additional improvements were suggested by Malhotra et al. (2000). Figure 2-16 shows a schematic of the model derived by Housner (1963) for rigid-walled cylindrical tanks, while Figure 2-17 presents an extension in case of flexible tanks.
The main assumptions behind these equivalent mechanical models are listed as follows:

- The mass of contained fluid is categorized into two components corresponding to impulsive and convective behavior;
- Identification of vibration frequencies of these two components;
- Assessment of the influence of wall flexibility on the dynamic response of the system.

In general, mechanical spring-mass analogies represent the fluid by means of lumped/concentrated masses, which are connected to the tank walls rigidly or through springs. This type of spring-mass analogies provides a satisfactory method to compute shear force and overturning moment at the base of the tank, which in turn can be used to obtain vertical and shear stresses in the wall just above the base of the tank. Therefore, these approaches permit one estimating the global seismic response of tanks accounting for the effects on foundations as well. In addition, the circumferential stresses induced by an earthquake can be easily computed based on codified provisions that allow the assessment of hydrodynamic pressures exerted on the wall of the tank. A review of well-established procedures based on different international standards can be found in the literature (see among others Beilic 2013; Brunesi et al. 2015).

Even though simplified modeling techniques are attractive approaches for fragility analysis of similar systems, detailed finite element models with explicit consideration of fluid-structure interaction may give more insight than mechanical analogies, as they are also able to associate sloshing motion and internal actions with stress/strain distributions, which in turn may be more appealing in terms of identification of damage states of interest. A review of recent advances regarding feasible modeling approaches and simulation procedures used for seismic analysis/design of tanks was provided in the following. Finally, the prevailing hypotheses assumed in order to carry out the set of nonlinear dynamic analyses shown here were listed and discussed in detail.

2.4.2 Finite element modeling and simulation techniques

Over the last decade, a significant number of research efforts (Virella et al. 2006; Sezen et al. 2008; Ozdemir et al. 2010; Moslemi and Kianoush 2012; González et al. 2013; Buratti and Tavano 2014; Brunesi et al. 2015) has demonstrated the feasibility of detailed finite element models to accurately simulate the seismic response of liquid storage steel tanks. Virella et al. (2006) investigated the fundamental impulsive modes of vibration of cylindrical tank-liquid systems anchored to the foundation. Sezen et al. (2008) assessed the dynamic behavior of above-ground liquid-containing tanks damaged during the 1999 Kocaeli, Turkey earthquake. Ozdemir et al. (2010) validated their nonlinear dynamic numerical procedure in comparison with the results of shake table tests of past experimental program on both anchored and unanchored systems. Moslemi and Kianoush (2012) carried out a series of parametric analyses on concrete cylindrical open top ground-supported water tanks, with the main focus of identifying the major parameters that affect the dynamic response of such structures, quantifying their interaction as well. González et al. (2013) reproduced the collapse mechanism of a leg-supported tank, affected by the 2010 Maule, Chile earthquake, while Buratti and Tavano (2014) proposed seismic fragility curves of an anchored steel tank by the added mass method. Finally, Brunesi et al. (2015) predicted damage mechanisms observed in the aftermath of the 2012 Emilia seismic sequence by using high-definition finite element that explicitly account for sloshing phenomena.
In light of this scenario, finite element analyses appear an attractive tool for seismic performance assessment of such structures, being able to capture stress/strain concentrations, crucial in interpreting damage patterns and failure modes. Therefore, a series of high-fidelity models have been developed to reproduce the seismic response both of flat-bottomed steel tanks. Geometrically and materially nonlinear dynamic analyses have been prepared into a general purpose finite element package, with fluid-structure interaction capabilities; LS-DYNA is adopted to play out the numerical procedure. Arbitrary Lagrangian Eulerian (ALE) method is used to allow for large structural and liquid deformation (see Figure 2-18).

In particular, penalty coupling method for shell and solid, which was defined within the framework of CONSTRAINED_LAGRANGE_IN_SOLID, is chosen to act in compression only. In the proposed numerical approach, compressible Navier-Stokes equations are assumed to describe the motion of the fluid, which was represented by a mesh of 8-noded, 1-point integration ALE solid elements; 4-noded, 2-points integration, Belytschko et al. (1984) shell elements are selected to materialize the wall of the tank. This shell element type represents a computationally efficient solution, since it is based on a combined co-rotational and velocity-strain formulation. Bilinear elastic-plastic constitutive law (MAT_003), with a combination of isotropic and kinematic hardening, is used to capture the cyclic permanent deformations exhibited by the wall of the tank under seismic excitation. Null material is chosen to simulate the fluid that was implemented through its bulk modulus; in addition, the case study tanks are analyzed as completely filled, which is the normal operation condition. An explicit solution strategy is adopted to perform the series of dynamic simulations, with automatic mesh-dependent integration time step of the order of 10-5.

Hence, the numerical technique proposed herein is able to account for many sources of nonlinearity, such as large amplitude nonlinear sloshing of free surface liquid and yielding/buckling mechanisms of tank wall. Such a detailed finite element approach was considered to predict the experimental response of two case-study tanks. Comparisons between numerical estimates and experimental results were provided in the following.
2.4.3 Numerical analysis of tested specimens: case-study 1 – Manos and Clough (1982)

The first case-study structure is the one tested in the pioneering experimental research carried out by Manos and Clough (1982). In particular, an anchored open-top tank resting on rigid foundation was analyzed numerically according to previously presented modeling techniques.

The tank prototype used in the experiments was a 1/3 scaled structure having a radius of about 1.83 m and a total height of 1.83 m. The system was filled with water (1000 kg/m$^3$) up to a height of 1.53 m. The tank was made of aluminum with a density of 2700 kg/m$^3$. The corresponding elastic modulus and yield stress were equal to 71000 MPa and 100 MPa, respectively. The thicknesses of the base plate was 2 mm. The same thickness was used for the tank wall, which however has a second course with a thickness equal to 1.3 mm. An L-shaped steel girder was placed onto the top of the second shell course, as shown in Figure 2-19, where a schematic presenting the geometry of tank is provided.

In the experimental shaking table test, the input motion was derived from the horizontal component of the El Centro 1940 earthquake assuming a peak ground acceleration (PGA) of 0.50 g. The input was then scaled with respect to time by 1/3 because of similitude requirements.

Figure 2-19 shows the numerical model developed for explicit nonlinear dynamic analysis. The same tank dimensions and material properties assumed in the experimental study were used to carry out the numerical simulation. The tank was assumed to be fixed at its base and the base plate was modeled to behave as a rigid surface. The fluid was considered to be incompressible. Gravity loads were applied to the model by means of gravity acceleration in a long timeframe so that the induced perturbations were observed to vanish. The seismic input consisting of real accelerograms was applied to the base of the tank, as done in the experimental investigation. The resulting finite element model had 5793 nodes, 890 shell elements and 4320 solid elements and the computational time required for the analysis was up to 16 hours on a multiprocessor mode.
A comparison between numerical predictions and experimental data is given in terms of pressure time histories at different locations up to the height of the tank wall, as shown in Figure 2-20. In the experimental study carried out by Manos and Clough (1982), measurements were taken of the pressure recorded at key position, and this parameter was considered to be crucial for validation of numerical models able to reproduce sloshing phenomena, as well as damage mechanisms. In particular, the pressure time histories shown in Figure 2-20 refer to three specific locations up to the height of the tank wall (i.e. R = 1.83 m at heights of 0.05 m, 0.45 m and 0.86 m above the tank base). Numerical predictions were found to be in close correlation with experimental results, as almost negligible differences can be observed for each position under consideration. Response graphs revealed that peak pressures were predicted in a very consistent and accurate manner, and the same applies to the post-peak behavior. Similar considerations can be drawn for the shape and amplitude of sloshing wave, as small discrepancies (< 5%) were determined from the comparison between numerical and experimental data.
2.4.4 Numerical analysis of tested specimens: case-study 2 – Haroun (1980)

The aforementioned numerical techniques were considered to predict the experimental response of another reference tank, namely case-study 2 (Figure 2-21). In particular, the cylindrical tank tested by Haroun (1980) is an open top, anchored prototype filled with water. The shell is made of aluminum with modulus of elasticity and density equal to 68940 MPa and 2600 kg/m³, respectively. The radius of the tank is 1.18 m; its total height is 4.57 m, while the height of the water free surface was 3.96 m. The thickness of the wall changes along the height, as a value of about 2.3 mm was provided up to 3 m, whereas the upper part of the tank wall consisted of a 1.6 mm-thick shell. The 1994 El Centro earthquake, scaled to a PGA equal to 0.5 g, was assumed by Haroun (1980) to perform the shake table test considered here for validation purposes. Figure 2-22 presents deformed shapes of the finite element model at different integration time steps, thus providing a measure of the sloshing motion predicted during explicit nonlinear dynamic analysis. This finite element model, developed along the lines of that used to predict the tank tested by Manos and Clough (1982) had 8009 nodes, 1236 shell and 6084 solid elements with ALE formulation.

Figure 2-21: Finite element model of case-study 2 (Haroun 1980)
Figure 2-22: Case-study 2 (Haroun 1980) – Sloshing motion at different integration time steps

Figure 2-23 and Figure 2-24 collect the prevailing results obtained numerically, while comparisons with experimental data are systematically summarized in Table 2-1.

Figure 2-23: Case-study 2 (Haroun 1980) – Numerically predicted base shear time history
Figure 2-24: Finite element predictions of case-study 2 (Haroun 1980) in terms of a) maximum meridional compressive stress profile; b) maximum radial displacement profile

Table 2-1: Comparison between experimental data and numerical predictions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Experimental</th>
<th>Numerical</th>
<th>[%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Radial Displacement [mm]</td>
<td>3.32</td>
<td>3.40</td>
<td>2.41</td>
</tr>
<tr>
<td>Max. Meridional Compression [N/m]</td>
<td>63396</td>
<td>62205</td>
<td>1.88</td>
</tr>
<tr>
<td>Max. Base Shear Force [kN]</td>
<td>122</td>
<td>125</td>
<td>2.46</td>
</tr>
</tbody>
</table>

In detail, the variation of base shear with respect to time was provided in Figure 2-23, whilst Figure 2-24 shows meridional compressive stress and radial displacement peak profiles. Also in this case, numerical predictions were proven to be in close agreement with experimental results, as a difference of about 2% was indeed obtained in terms of maximum base shear. Similar discrepancies can be observed as far as the maximum meridional compressive force and the peak radial displacement are concerned.

2.4.5 Parametric simulation of representative storage systems

Once the aforementioned numerical procedures were validated in compliance with experimental data, a set of parametric finite element analyses was performed to explore the seismic vulnerability of these systems. Predictions were collected to point out behavioral changes in the seismic response of storage tanks as a consequence of variations in their geometrical characteristics. The analysis results may thus serve as a cross-validation of the trends emerged from the set fragility curves presented in previous paragraphs.

Figure 2-25 collects representative examples of the finite element models developed to study prototypes that are characterized by different height-to-radius (H/R) and radius-to-thickness (R/t) ratios.
Accordingly, Table 2-2 shows the range of variation of the aforementioned parameters.

Table 2-2: Parametric analysis – Range of $H/R$ and $R/t$ ratios for numerical simulations

<table>
<thead>
<tr>
<th>$H/R$</th>
<th>0.50</th>
<th>0.75</th>
<th>1.00</th>
<th>1.50</th>
<th>2.00</th>
<th>2.50</th>
<th>3.00</th>
<th>4.00</th>
<th>5.00</th>
<th>6.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R/t$</td>
<td>100</td>
<td>125</td>
<td>150</td>
<td>250</td>
<td>400</td>
<td>600</td>
<td>800</td>
<td>1000</td>
<td>1500</td>
<td>2000</td>
</tr>
</tbody>
</table>

As a further validation of the proposed modeling approach, numerically predicted and analytically computed hydrostatic pressure profiles were compared for tanks having different height-to-radius ratios (see Figure 2-26), thus showing an almost perfect match also in this case. Explicit nonlinear dynamic analyses were then performed, and the prevailing analysis results were collected in Figure 2-27, Figure 2-28 and Figure 2-29, where the pressure distributions obtained for prototypes with different $H/R$ are categorized on the basis of $R/t$. 

Figure 2-25: High-definition finite element models for parametric simulation
Figure 2-26: Hydrostatic pressure – comparison between analytical estimates and finite element predictions for case-study tanks having different aspect ratios.

a) H/R = 1.00; b) H/R = 2.00; c) H/R = 3.00; d) H/R = 4.00
Figure 2-27: Parametric simulations of squat case-study tanks having different aspect ratios.
  a) $H/R = 0.50$; b) $H/R = 0.75$; c) $H/R = 1.00$; d) $H/R = 1.50$

Figure 2-28: Parametric simulations of medium case-study tanks having different aspect ratios.
  a) $H/R = 2.00$; b) $H/R = 2.50$
Severe concentrations can be observed in the set of pressure peak profiles determined for tanks having H/R equal to or larger than 3, whilst the other case-study tanks under consideration appear to be less vulnerable to earthquake excitation. The set of fragility curves collected in Figure 2-29 present similar trends. As far as the median of class 1 is concerned (0.7 < D/H < 1.0), collapse was attained for PGA levels of about 0.5 g (see Figure 2-30). By contrast, class 2, 3 and 4 imply PGAs roughly equal to 0.7 g, 0.8 g and 1.0 g, respectively. Similar considerations can be drawn if one refers to an additional set of numerical simulations that was performed for storage tanks having height-to-radius ratio equal to 1.00 and 0.40. Incremental dynamic analyses were thus carried out for these two systems which belong to classes 3 and 4, respectively. Figure 2-31 shows the peak radial displacement profiles determined for increasing PGAs, confirming that buckling took place at peak ground accelerations higher than 0.5 g. Values of about 0.8 g and 1.0 g can be indeed associated with this mechanism, as far as prototypes with H/R equal to 1.00 and 0.40 are concerned. Accordingly, Figure 2-32 presents an example of plastic strains predicted for the former case at a PGA equal to 0.8 g.
Figure 2-31: Maximum radial displacement profiles at increasing PGAs. 
a) H/R = 1.00 – class 3; b) H/R = 0.40 – class 4

Figure 2-32: H/R = 1.00 – class 3. Plastic strain distributions for PGA = 0.8 g
3 HORIZONTAL AND VERTICAL VESSELS

Pressure vessels are probably one of the most widespread equipment within the different industrial sectors. Indeed, there is no industrial plant without pressure vessels. More specifically, pressure vessels represent fundamental components in sectors of paramount industrial importance, such as the nuclear, oil, petrochemical, and chemical sectors.

Among the different equipments that are present in industrial equipments, horizontal and vertical vessels contain usually large quantity of hazardous material. Concerning vertical vessels, different kind of geometry are adopted and reported in the figure below depending on the support typology (vertical vessels supported by legs, skirts and lugs):

![Figure 3-1](image)

Figure 3-1: Example of vertical storage with different supports configuration

Only vessels supported by skirts are considered in this study.

Horizontal vessels considered in this report have steel saddles and vertical rigid supports similar to those reported in the picture below:

![Figure 3-2](image)

Figure 3-2: Example of horizontal vessel with steel saddles and rigid concrete piers
3.1 DAMAGES OBSERVED IN PAST EARTHQUAKES

Analysis of damages observed in industrial plants in past earthquakes is a crucial aspect in the development of seismic vulnerability analyses. This helps us to understand the failure modes to be considered in the different items and the main modeling aspects.

A detailed overview of damages observed in past earthquake is reported in Danesi, (2015) in which information for industrial equipments are collected. The discussion here is limited to horizontal and vertical vessels. The past events examined in Danesi, (2015) are reported in the table below:

<table>
<thead>
<tr>
<th>Quake</th>
<th>Year</th>
<th>Place event</th>
<th>Mw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1984</td>
<td>Morgan Hill, USA</td>
<td>6.2</td>
</tr>
<tr>
<td>2</td>
<td>1985</td>
<td>Algarrobo, Chile</td>
<td>8.0</td>
</tr>
<tr>
<td>3</td>
<td>1988</td>
<td>Spitak, Armenia</td>
<td>6.8</td>
</tr>
<tr>
<td>4</td>
<td>1989</td>
<td>Loma Prieta, USA</td>
<td>6.9</td>
</tr>
<tr>
<td>5</td>
<td>1990</td>
<td>Luzon, Philippines</td>
<td>7.8*</td>
</tr>
<tr>
<td>6</td>
<td>1991</td>
<td>Limon, Costa Rica</td>
<td>7.7</td>
</tr>
<tr>
<td>7</td>
<td>1994</td>
<td>Northridge, USA</td>
<td>6.7</td>
</tr>
<tr>
<td>8</td>
<td>1999</td>
<td>Kocaeli, Turkey</td>
<td>7.6</td>
</tr>
<tr>
<td>9</td>
<td>2001</td>
<td>Bhuj, India</td>
<td>7.7</td>
</tr>
<tr>
<td>10</td>
<td>2003</td>
<td>Bam, Iran</td>
<td>6.6*</td>
</tr>
<tr>
<td>11</td>
<td>2010</td>
<td>Maule, Chile</td>
<td>8.8</td>
</tr>
</tbody>
</table>

Performance of vessels in the previous events is reported in the following table:

<table>
<thead>
<tr>
<th>Quake</th>
<th>Performance of vessels</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Two horizontal boilers housed in a small sheet-metal building slid several inches</td>
</tr>
<tr>
<td>2</td>
<td>Boilers slid from their unanchored support</td>
</tr>
<tr>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>The earthquake's most significant effect on vertical vessels involved anchor bolts. At one refinery, over 20 vertical vessels had stretched anchor bolts</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>There were two instances of anchorage failure and three instances of significant yielding of anchorages for process vessels</td>
</tr>
<tr>
<td>8</td>
<td>Elevated reactor vessel broke lateral supports near the top of the vessel; sliding of vessels were reported</td>
</tr>
<tr>
<td>9</td>
<td>Broad tanks slid off their foundations by as much as a foot</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>Structural damage was observed in boiler steel buildings of power plants. Failures were concentrated in seismic stoppers, which in some cases led to total destruction of the restraining elements and the consequent clashing between boiler and building structure</td>
</tr>
</tbody>
</table>

Note: - indicates that no damage was reported in this type of component
Figure 3-3: Shear failure of anchor bolt (Roche et al., 1995) (left and middle); Sliding of horizontal vessels (Johnson et al., 2000) (right)

Additional information related to damages observed in past earthquakes to vessels is reported in FEMA E74 (2011) and some pictures are reported below.

Figure 3-4: Tank shifted off support curb, Granada Hills Hospital in the 1994 magnitude-6.7 Northridge Earthquake (FEMA E74, 2011)
A list of damages observed in petrochemical facilities after seismic events worldwide is described in Lopez et al (1992): after events in US during the 80’s and 90’s many industrial plants remain out of order for several months. Concerning vessels, damages were mainly observed in connection between piping and vessels, and in foundation anchoring. Effects of seismic events, even if of moderate intensity, were increased by the fact that the vast majority of the industrial plant equipments were designed for gravity and function loads only.

In Paolacci (2013) an overview of the response of different items in petrochemical plants is also described. The most frequent damages in case of seismic event are the anchor bolts failure at the foundation due to the excessive actions, and the loss of contained fluids because of the failure of connected flanges due to excessive relative displacements. Failure due to local buckling at the skirt level was also observed.

From the previous references the following conclusions are possible. Although the performance of a specific plant, structure or component, is case dependent since the number of factors influencing the response is considerable, the repetition of some failure modes makes possible the following summary.

The failure of vessels is rarer than failure of piping. Failure modes have been attributed to:

- Unanchored equipments;
- Yielding of anchor bolts;
- Failure in shear of anchor bolts with consequent move up of vessel;
- Failure in the connection between piping and vessels
- Buckling at the skirt level.
Fragility curves evaluation reported in the present study, aims to consider all the main failure mechanisms reported in the previous references.

### 3.2 INTERNATIONAL CODES AND LITERATURE

#### 3.2.1 International Codes

International standards dealing with horizontal and vertical pressure vessels are considered in this section. The vast majority of the standards regarding pressure vessels mainly addressed the design from the static and functional point of view. There are not specific standards that deal with seismic design of pressure vessels.

Regarding static and functional design of pressure vessels there are two main standards in United States and Europe. In United States the main standard dealing with pressure vessels is the ASME BPVC “Boiler and Pressure Vessel Code”. In Europe the reference standard is the EN 13445-3:2009. EN 13445-3 gives a detailed description of the stress analysis for non-pressure loads for horizontal pressure vessels on saddle supports in chapter 16.8 and for vertical pressure vessels on skirt in chapter 16.11. Alternatively, “Design by Analysis” Annex B and C may also be considered. The method described in Annex B deals with various design checks, whereas the method described in Annex C, referred to a “stress analysis”, involves the interpretation of stresses calculated on an elastic basis at any point in a part of a vessel, and then verification of their admissibility by means of appropriate assessment criteria. Seismic analysis of pressure vessels is only occasionally treated in the two previous codes. In Europe, other two standards for the design of vessels walls and skirts are: Eurocode 8 part 4 and Eurocode 3 part 6. Both codes give rules for the buckling design of steel tank walls under internal pressure and these rules can be used also for the assessment of pressure vessel walls and vessel skirts. The critical buckling strength of a skirt can be determined also using a number of published sources. The two most common methods for determining the critical buckling strength of a skirt in US are ASME BVPC Section VIII, Division 2, Paragraph 4.4 and AWWA D100 Section 13.4.3.4.

As far as seismic analysis of pressure vessels is concerned, in Europe the main code that gives rules for the estimation of the seismic actions in industrial equipments is the Eurocode 8 part 4. Eurocode 8 part 4 Annex A provides a methodology for calculating hydrodynamic forces due to a seismic event. The provision for vertical containers can be employed for the vertical pressure vessels. However, when the height to diameter aspect ratio is large, sloshing effects can be neglected, considering also the presence of equipment inside the vessel. A methodology for horizontal pressure vessels is also presented in A.5. It should be noted that the calculation of sloshing frequencies and masses in horizontal cylindrical containers is not trivial, due to the lack of closed form analytical solution of the corresponding hydrodynamic problem. In United States, ASCE/SEI 7-10 (ASCE, 2010) includes pressure vessels in Chapter 15 for non-building structures. In that Chapter, guidance is given for design requirements of pressure vessels and the selection of seismic factors for calculating the base shear. Various types of vessels are addressed, such as elevated vessels on leg or skirt supports, horizontal saddle-supported welded steel vessels and vessels supported on structural towers similar to buildings and the behavior factors are given. Some of the rules reported in ASCE 7-10 are also explained in FEMA P750 (2011) and FEMA P751 (2011). In these two documents detailed rules are given for the estimation of the fundamental period of vibration of vertical pressure vessels and for the definition of the seismic base shear and overturning moment.
In Italy, the Italian Society for the Industrial constructions provides indication for the seismic vulnerability analysis of industrial equipments with reference to the requirements reported in the Direttiva 2012/18/UE “Seveso III”.

3.2.2 International References

A vast number of papers and books are available for the design of pressure vessels especially for the static and functional design. Regarding the seismic design, all the references deal mainly with the seismic behavior of specific cases. Only a few references deal with fragility curves evaluation. An overview of some of the main references is reported herein.

Di Carluccio in his Phd thesis deals with seismic hazard definition, structural fragility component, accident – sequence analysis in a plant and consequence analysis. The document treats in depth steel tank but there are some tables about standard dimension/ anchors type of pressure vessels.

The Pressure Vessel Design Manual, Moss (2003), deals with the design, mainly static of pressure vessels. Supports design is also included for all types of vertical and horizontal vessels.

Hossein Kazem and Mahmood Minavand (Kazem and Minavand, 2008) give indications for the retrofit of horizontal and vertical pressure vessels: equipments are modeled with shell elements.

In Di Carluccio (2008) the seismic response of horizontal vessels for a single case is analyzed: the model is developed with shell and solid elements and the analysis includes the response of the supports.

In the European Project INDUSE (2013) and specifically in the work package 6 “seismic behavior and design of industrial pressure vessels” seismic design guidelines and recommendations are developed. In Deliverable D6.2 recommendations for the definition of the seismic input at the base of horizontal and vertical vessels are reported. In Deliverable D6.1 the seismic behavior of horizontal and vertical pressure vessels are discussed. For vertical pressure vessels three specific geometries are analyzed with the aid of non linear static and dynamic analysis. The models are developed with shell elements and the safety criteria are defined in terms of shell strains. For horizontal pressure vessels the models are again developed with shell elements and three different support conditions are investigated.

In ASCE (2011) a complete set of recommendations for the seismic design and assessment of equipments in petrochemical facilities is reported. The guidelines start from the definition of the seismic hazard through the methods to estimate the base shear and overturning moment. Simplified equations, similar to those reported in ASCE 7-10, are reported for the estimation of the fundamental period of vibrations of vertical vessels and assembling of components.

Considering specifically references that deal with fragility curves evaluation for horizontal and vertical vessels, just a few studies are available.

In Hazus (FEMA 1999), specifically in Chapter 8.5.8 and Appendix 8.C (Table D8.8 and D8.9), are reported the results of damage algorithms for anchored and unanchored subcomponents of generation facilities (after G&E, 1994). Some parameters for fragility curves for horizontal and vertical vessels are given.
In Moharrami and Amim Amini (2012) fragility curves for vertical process towers are discussed. Fragility curves are evaluated with incremental dynamic analysis: all the main material and geometrical properties are assumed to be deterministic and the only source of uncertainty is in the ground motion definition.

3.3 FRAGILITY CURVES FOR HORIZONTAL VESSELS

3.3.1 General method description
The procedure used to determine seismic fragility curves for horizontal vessels that will be described in the following section is based on these five classes of input:

- Damage states fixed by data available in literature;
- Geometrical and quantitative vessels characteristics: defined a random variable starting from a range relative to data received for Plan A and Plant B;
- Mechanical characteristics are assumed as deterministic;
- Seismic characterization of the site based on acceleration response spectra defined for different return periods;
- Geological and Geotechnical data of the site: assumed as deterministic.

Input data are then processed by mechanical methods and vulnerability curves are determined, representing damage as a function of the seismic intensity. Equivalent linear soil structure interaction has been included in the fragility curves evaluation: this is done because horizontal vessels are mainly founded on shallow foundations and due to the high stiffness of the vessel/supporting system the soil foundation structure interaction may play an important role.

The following flowchart summarizes the procedure adopted to determine seismic fragility curves for horizontal vessels.
Figure 3-6: Flow chart for the definition of i-th statistically independent horizontal vessels
FRAGILITY FUNCTIONS (Incremental Equivalent Linear Static Method)

STEP 1
Development of numerical $i$-th model (Equivalent linear model)

STEP 2
Response spectrum definition for different return periods of the seismic intensity

STEP 3
Development of Equivalent Linear Static Analyses for all the statistically different models and for all the seismic intensities.

STEP 4
Identification of performance Levels for all the sub systems (foundation, anchorage, piping)

STEP 5
Identification of the seismic risk indicator $\rho$ for each vessel sub-system given a selected Performance Level:

$$\rho_{n,Dir,i2/PL,i} = \frac{D_{n,Dir,i2/Dir}}{C_{i2/PL}}$$

Where:
- $i_1$: index for seismic intensity (spectrum)
- $Dir$: direction considered
- $i_2$: index for failure mechanism
- $D$: rotation/force demand (curvature demand)
- $C$: subsystem displacement / vessel capacity
- $i$: $i$-th statistically independent vessel

STEP 6
Identification of the seismic risk indicator $\rho$ from the envelope of seismic risk indicators for all the sub-systems for a given direction:

$$Y_{n,Dir/PL,i} = \max(\rho_{n,Dir,i2/PL,i})$$

STEP 7
Definition of the number of failures, $Y_{n,Dir/PL,i} \geq 1$, for a given direction and a given performance level

STEP 8
Lognormal fit of the number of failures following the approach reported in Baker J. W. (2013)

Figure 3-7: Flow chart for the definition of fragility curves for horizontal vessels
Fragility curves evaluation starts with the definition of the properties of 90 statistically independent vessels. Vessels are defined considering the following properties:

- 10 different radius defined based on a uniform distribution function with average \( R = 1.0 \text{ m} \) and a variation of +/- 25%;
- Three different connection typologies: 2+2 bolts, 4+4 bolts and 6+6 bolts;
- Three different moment/rotation curves with lower, medium and best estimate soil parameters.

For all the statistically independent vessels a simulated design has been performed as described in the following parts.

### 3.3.2 Definition of the vessel properties

The wall thickness of each vessel has been estimated with the following equation (Moss, 2004):

\[
T_p = \frac{P \cdot R}{S \cdot E + 0.4 \cdot P} + C
\]

Being:

- \( P = \) internal pressure of the fluid containment = 190 kPa;
- \( R = \) radius of the vessel;
- \( S = \) allowable steel stress assumed equal to 95 MPa;
- \( E = \) joint efficiency factor = 0.85;
- \( C = \) corrosion allowance assumed to be 3 mm.

The properties of the 10 statistically independent vessels are reported in the following table:

<table>
<thead>
<tr>
<th>Vessel</th>
<th>Radius</th>
<th>L/R</th>
<th>Saddle's height</th>
<th>Length</th>
<th>% heads</th>
<th>Volume</th>
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</thead>
<tbody>
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<td>3.567</td>
<td>1.2</td>
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</tr>
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<td>3.837</td>
<td>1.2</td>
<td>23.66</td>
</tr>
</tbody>
</table>
Table 3-4: Masses for the 10 statistically independent vessels

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
<th>Fluid density (kg/m³)</th>
<th>Pressure (kPa)</th>
<th>Mass acrylonitrile (kg)</th>
<th>S-allowable stress (kPa)</th>
<th>Joint Efficiency factor</th>
<th>Corrosion Allowance (mm)</th>
<th>Vessel Thickness (mm)</th>
<th>Percentaage connections</th>
<th>Vessel Mass (kg)</th>
<th>Total Mass (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>190</td>
<td>810</td>
<td>3998</td>
<td>95148.2</td>
<td>0.85</td>
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<td>1917</td>
<td>21081</td>
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</tbody>
</table>

3.3.3 Definition of the connection properties

Three different connection configurations have been considered: 2+2 bolts, 4+4 bolts and 6+6 bolts per each curb. A schematic view of the connection configurations is reported in the following sketch.

![Figure 3-8: Foundation stiffness degradation](image)

For both the directions, the shear resistance of the supporting system has been estimated as the minimum between the pure shear resistance of the bolts and the shear resistance due to the flexural strength. The first case leads to a “shear mechanism” while the second case leads to a “flexural mechanism” of the connection.

In the longitudinal direction the connection capacity has been estimated with the assumption of a rigid behavior of the steel vessel and that the connections react with a push–pull mechanism.
Under these assumptions the following relationship has to be verified for the longitudinal direction:

\[ V_{RD,\text{long}} = \min \left( V_{RD,\text{bolts}}, \frac{M_{rd,\text{conn},\text{long}}}{H} \right) \]

In which:

- \( V_{RD,\text{long}} \) = shear resistance of the supporting system in the longitudinal direction;
- \( V_{RD,\text{bolts}} = nA_{s,bolt}f_{yk} \)
- \( n \) = total number of bolts;
- \( A_{s,bolt} \) = shear area of the single bolt;
- \( f_{yk} \) = allowable shear stress in the bolt material.

The resisting moment of the connection in the longitudinal direction, \( M_{rd,\text{conn},\text{long}} \) has been computed with a push and pull mechanism between the two curbs:

\[ M_{rd,\text{conn},\text{long}} = 2P L_s \]

\[ P = \min \left( N_{RD,t}, N_{RD,c} \right) \]

- \( L_s \) = distance between the two curbs;
- \( N_{RD,t} \) = tensile resistance of the connection in a single curb:
  \[ N_{RD,t} = nA_{s,bolt}f_{yeq} \]
- \( n \) = number of bolts;
- \( f_{yeq} \) = allowable tensile stress in the single bolt based on the embedment length, on the distance from the edge of the connection and on the concrete compression strength. The estimated value is \( f_{yeq}=42 \) MPa and 28 MPa for the performance level 1 and 2 respectively;
- \( N_{RD,c} \) = compressive resistance of the connection in a single curb:
  \[ N_{RD,c} = a b f_c \]
  being \( a \) and \( b \) the dimensions of the curbs.

In the transverse direction, the connection capacity has been estimated with the assumption that the applied seismic shear and moment are equally distributed to the two curbs.

Under this assumption the following relationship has to be verified for the longitudinal direction:

\[ V_{RD,\text{trasv}} = \min \left( V_{RD,\text{bolts}}, \frac{2M_{rd,\text{conn},\text{trasv}}}{H} \right) \]

In which:

- \( V_{RD,\text{trasv}} \) = shear resistance of the supporting system in the transverse connection;
H = vertical distance between the vessel centre of gravity and the supports level.

The resisting moment of the connection in the transverse direction, \( M_{RD,conn,transy} \), has been computed considering the single curb as a reinforced concrete section in which the reinforcement is due to the bolts. The yielding tensile strength of the bolts is substituted with the allowable tensile stress in the single bolt \( f_{y}\text{eq} \).

In order to design the required bolt diameters for all the 10 different statistically independent vessels and for all the three different connection configurations, a simulated design of the connection has been first developed based on the seismic load obtained by the application of the National Italian Code DM 1996. This assumption has been done in order to reflect the case of the assessment of existing vessels designed with older Italian Seismic Codes. The seismic acceleration applied to all the vessels in accordance with the DM 1996 has been set equal to 0.1 g.

For the definition of the resistance of the connection in the two different limit states, PL1 and PL2, the following assumption has been considered: the resistance for the PL1 has been estimated with characteristic properties for bolts and concrete divided by the safety factor 1.5, for the PL2 the safety factor is assumed to be one.

The following table shows the connection properties for all the statistically independent vessels:

### Table 3-5: Connections for the 30 statistically independent vessels

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<th>R</th>
<th>hour</th>
<th>H</th>
<th>f</th>
<th>fc,concrete</th>
<th>fad,concrete</th>
<th>a</th>
<th>b</th>
<th>L</th>
<th>c</th>
<th>fd,V</th>
<th>e</th>
<th>nbolt,TO</th>
<th>As,bolt</th>
<th>fyeq, PL1</th>
<th>fyeq, PL2</th>
<th>W</th>
</tr>
</thead>
<tbody>
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<td>m</td>
<td>m</td>
<td>mm</td>
<td>MPa</td>
<td>MPa</td>
<td>m</td>
<td>m</td>
<td>mm</td>
<td>mm</td>
<td>MPa2</td>
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<td>1.04</td>
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<td>150</td>
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Table 3-6: Connections resistance for the 30 statistically independent vessels: transversal direction

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Table 3-7: Connections resistance for the 30 statistically independent vessels: longitudinal direction

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3.3.4 Definition of the foundation properties

Considering the type of equipment, a rectangular shallow foundation has been assumed. The length of the foundation has been assumed equal to the length of the vessel and the width of the foundation has been set equal to the diameter of the vessel. The foundation height and embedment has been defined equal to 0.5 m for all the vessels. Considering a friction angle of the soil of 25° and a unit
weight of 18 kN/m³, the allowable maximum soil pressure has been assumed equal to 230 kPa. This value, multiplied for the foundation area, gives an estimation of the allowable vertical load on the foundation: the effect of the eccentricity on the bearing capacity of the foundation has been neglected. Given the weight of the vessels and of the foundations, the ratio between bearing capacity and applied vertical load, \( \nu = \frac{N_{rd}}{N_{sd}} \), has been found close to 7.5 for all the vessels.

Based on the previous parameters, the foundation horizontal \( K_{H,0} \), and rocking stiffness, \( K_{\theta,0} \), have been defined:

\[
K_{H,0} = \frac{G \cdot L}{2 - \nu} \quad K_{\theta,0} = \frac{G \cdot I_{0.75}}{1 - \nu}
\]

being: \( L \) = foundation dimension in the direction under consideration, \( G \) = soil shear modulus, \( \nu \) = Poisson’s coefficient for the soil, \( I \) = second moments of area of the footing about the axis orthogonal to the direction under consideration. The dynamic stiffness of the foundation has been assumed equal to the static stiffness in this study.

In the inclusion of soil structure interaction effect in the fragility evaluation of horizontal vessels, in addition to the foundation stiffness, the radiation damping needs to be defined.

The radiation damping, \( \xi_s \), has been defined in accordance with the Eurocode 8 Part 4 (1998):

\[
\xi_s = \frac{2 \cdot \pi^2 \cdot m_{TOT} \cdot a}{K_{H,0} \cdot T_0^2} \left[ \frac{\beta_s}{\alpha_s} + \frac{K_{H,0} \cdot h_{tot}^2 \cdot \beta_\theta}{K_{\theta,0} \cdot \alpha_\theta} \right]
\]

where:

\[
a = \frac{2 \cdot \pi \cdot R}{V_s \cdot T} \approx 1
\]

in our cases these values are considered as constant:

\[
\alpha_s = 1.0; \quad \beta_\theta = 0.1; \quad \beta_s = 0.5; \quad \alpha_\theta = 0.8
\]

\( T_0 \) first fundamental period of the equivalent single degree of freedom system (Veletsos, 1990):

\[
T_0 = 2 \pi \sqrt{\frac{m_f + m}{K_{H,0}} + \frac{m \cdot h_{tot}^2}{K_{\theta,0}}}
\]

In which \( m_f \) = foundation mass and \( m \) = vessel mass and \( h_{tot} \) = total height from the vessel centre of gravity to the foundation centre of gravity.

In order to account for the foundation non-linearity, the elastic initial rotational stiffness has been reduced depending on the foundation rotation and on the axial load ratio \( \nu \). Different research studies have been developed with the aim to estimate moment-rotation curve for shallow foundations. In this study, the approach for medium and dense sand proposed in Paolucci (2009) has been considered.

The study reported in Paolucci (2009) develops moment-rotation curves depending on the axial load ratio: a value of 7.5 has been assumed for this study. Three different moment/rotation curves have been adopted considering the epistemic uncertainty of the soil parameters: lower, medium and best.
estimate soil parameters have been considered. The following figure shows the foundation stiffness
degradation adopted in this study.

![Figure 3-9: Foundation stiffness degradation](image)

In order to compute the actual rotational stiffness, an iterative procedure has been developed starting
with the initial stiffness at zero rotation.

### 3.3.5 Definition of the engineering demand parameters for the horizontal vessels

In order to evaluate the fragility curves from the equivalent linear static simplified analyses, the en-
gineering demand parameters, EDPs, needs to be defined. The following EDPs have been assumed
in this study: foundation rotation, vessel rotation, base shear. The EDPs have been defined for all the
90 statistically independent vessels and for all the seismic intensities considered. Response spectra of
acceleration for the site have been considered for $T_1=30$ years, $T_2=50$ years, $T_3=475$ years, $T_4=
975$ years, $T_5=1400$ years, $T_6=2475$ years.

The following steps have been developed in the EDPs definition:

1. Definition of initial foundation rocking and horizontal stiffness;
2. Definition of the equivalent SDOF period;
3. Definition of the radiation damping;
4. Definition of spectral acceleration for the given return period, for the given period of vibration
   and radiation damping $S_a = f(\xi, T_0)$;
5. Determination of design data: base shear and overturning moment at supports level and foun-
dation level $M_f$;
6. Definition of the foundation rotation $\theta_{f1}$
   $$\theta_{f1} = \frac{M_f}{K_{\theta0}}$$
7. Definition of the foundation rocking stiffness and error check with the initial estimation
   $$K_{\theta1} = f(\theta_{f1}, \nu)$$
\[
\text{error}_i = \frac{K_{\theta, i+1} - K_{\theta, i}}{K_{\theta, i}}
\]

8. Repeat procedure until \(\text{error}<\text{tolerance}=5\%\)

9. Determine final base shear and overturning moment;

10. Check if the performance levels considered (foundation rotation capacity, piping rotation capacity and connections capacity) respect the fixed limit.

### 3.3.6 Definition of the damage limits for the different Performance Levels, PL

In order to reproduce the outcomes of the expected damages in past earthquakes, the following damage states and performance levels have been assumed.

The performance levels have been defined as follows:

- **PL1**: corresponds to the first leakage of the fluid content and minor damage is expected in the vessel structural system;
- **PL2**: complete release of content and global collapse of the vessel.

For each performance level, the limits for each damage states have been defined.

For the connection capacity, as already mentioned, the resistance for the PL1 has been estimated with characteristic properties for bolts and concrete divided by the safety factor 1.5, for the PL2 the safety factor is assumed to be one. Hence the connection capacity has been modeled with a “force controlled” approach.

For the piping capacity, the expected rotation of the vessel has been assumed equal to the expected rotation of the connected piping.

In comparison to structural components and systems, there is relatively limited information on the seismic performance of piping fittings. Although numerous piping codes and guidelines can be found on the literature, they basically present limiting values expressed in terms of stress-strain which are not useful for this approach. With this in mind, the capacities for each pipe diameter were computed taking into account the experimental research work carried at the University of Buffalo (Tian et al., 2014).

For the first leakage the following equation has been used:

\[
\theta_{\text{piping,PL1}} = \frac{2\bar{s}}{D}
\]

Being \(\theta_{\text{piping,PL1}}\) = rotation capacity of the connected piping for the first leakage, \(\bar{s}\) is the average axial slip (analogous to strain in bending assuming plane sections remain plane) and is a constant value of 0.019in, D=outside pipe diameter in inches.

It is important to highlight that rotational capacity for welded joints adopted from this reference should give considerably conservative results since a few experimental tests done for welded joints indicated a much stiffer behavior than the threaded ones. Based on the previous reference, the pipe rotation for the PL2, global collapse, has been assumed equal to 0.02 rad.

The foundation capacity has been expressed in terms of the rotational stiffness degradation as reported in Sullivan et al. (2012). The stiffness degradation \(K_0/K_{\theta 0}\) has been set to 0.3 and 0.2 for the PL1 and
PL2 respectively. These limits have been defined in order to avoid large non-linear soil deformations during and after earthquake shaking which may lead to collapse of the foundation system.

The following table summarizes the EDPs and the relative limits for all the damage states considered in this study.

<table>
<thead>
<tr>
<th>Performance Levels</th>
<th>Piping</th>
<th>Foundation</th>
<th>Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL1</td>
<td>$\theta = \frac{2S}{D}$</td>
<td>$K_0/K_{60} = 0.3$</td>
<td>Shear and tensile capacity</td>
</tr>
<tr>
<td>PL2</td>
<td>$\theta = 0.03$ rad</td>
<td>$K_0/K_{60} = 0.2$</td>
<td>Shear and tensile capacity</td>
</tr>
</tbody>
</table>

### 3.3.7 Fragility curves fitting

Once the engineering demand parameter has been obtained for all the damage failures, the seismic risk index, $\rho$, has been defined:

$$\rho_{i,\text{Dir}2/\text{PL}/i} = \frac{D_{i,\text{Dir}2/\text{PL}/i}}{C_{i/2/\text{PL}}}$$

where $i_1$ = index for seismic intensity (spectrum);

$\text{Dir} = $ direction considered;

$i_2 =$ index for failure mechanism (piping, connection or foundation);

$D =$ rotation/force demand;

$C =$ subsystem displacement / vessel capacity;

$i =$ i-th statistically independent vessel.

With the seismic risk index the risk indicator, $Y$, can be defined from the envelope of the seismic risk index for all the damage states:

$$Y_{i,\text{Dir}/\text{PL}/i} = \max(\rho_{i,\text{Dir}2/\text{PL}/i})$$

Once the risk indicator has been defined for all the seismic intensities, for both transversal and longitudinal direction, for all the performance levels and for all the 90 statistically independent vessels, the number of failures, $Y>1$, has been defined. The result is to obtain the number of failures for each PL and for both the directions. The number of failures has been then fitted to obtain fragility curves following the approach reported in Baker (2013). The procedure here proposed is not only suitable to estimate fragility curves but also to estimate the probability of the failure mechanism: piping, connection or foundation.

In the vessels here analyzed, the failure mechanism with the bigger likelihood is the “connection mechanism” and this confirms the damages observed in previous earthquakes.

In addition to the uncertainty defined with the equivalent linear static analysis, $\beta$ analyses, by means of 90 vessels for each intensity level, two other sources of variability have been also included: modeling epistemic uncertainty, $\beta_m$, and ground motion aleatory variability, $\beta_{gm}$. For this regard the approach reported in ATC 58 (2012) has been used with the following equation:
The following table shows the parameters obtained for the fragility curves for longitudinal and transversal direction and for both the performance levels.

Table 3-9: Horizontal vessels fragility curves parameters

<table>
<thead>
<tr>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse</td>
</tr>
<tr>
<td>PL1</td>
<td>PL2</td>
</tr>
<tr>
<td>PGA_m</td>
<td>0.37</td>
</tr>
<tr>
<td>β_analyses</td>
<td>0.26</td>
</tr>
<tr>
<td>β_m</td>
<td>0.40</td>
</tr>
<tr>
<td>β_gm</td>
<td>0.10</td>
</tr>
<tr>
<td>β</td>
<td>0.49</td>
</tr>
</tbody>
</table>

The following diagrams show the fragility curves for longitudinal and transversal direction and for both the performance levels.

Figure 3-10: Fragility curves for PL1 and PL2
As it is possible to see from the fragility curves, the transverse direction is the weakest. In addition to this it is possible to argue that, for vessels similar to those reported in this report horizontal vessels have relatively low seismic vulnerability given that the minimum median PGA among the different directions and performance levels is 0.37 g.

3.4 FRAGILITY CURVES FOR VERTICAL VESSELS

3.4.1 General method description

The definition of seismic fragility curves for cylindrical vertical vessels has been carried out by assuming the following assumptions:

- Mechanical properties assumed as deterministic parameters;
- Geometrical and quantitative properties of vessels defined as a function of a characteristic parameter \( H/R \) randomly variable in two ranges;
- Seismic characterization of the site based on acceleration response spectra defined for different return periods;
- Failure mechanisms fixed by considering recurrent failures available in literature.

Two classes of vertical vessels have been defined by randomly vary the slenderness ratio \( H/R \) into two different ranges, CL1 for vessels with \( H/R \) included in the range \([4;7]\) m and CL2 for vessels with \( H/R \) included in the range \([7;11]\). Starting from these assumptions, the vessels geometrical properties have been derived by a simulated design in accordance with the prescriptions provided by the standards. Numerical models have been implemented with the input data summarized here. The implemented numerical models have been defined as system in series for which the main source of non-linearity is concentrated in the base connection. These models assume a foundation system infinitely rigid with respect to the principal structure. The output data have been processed to define the corresponding fragility curves, expressing the probability of damage occurrence as a function of the seismic intensity.

Figure 3-11 summarizes the main design assumptions while Figure 3-12 shows the main steps of the procedure adopted for the definition of seismic fragility curves for vertical cylindrical vessels.
Figure 3-11: Flow chart for the definition of $i$-th statistically independent vertical vessels
Figure 3-12: Flow chart for the definition of fragility curves for vertical vessels

**STEP 1**
Development of the \( i \)-th numerical model (where non linear effects are in the hinge connection definitions)

**STEP 2**
I.D.A. with record selection for different return periods

**STEP 3**
Development of IDAs for all the records for all statistically different \( i \)-th models and for all the seismic intensities

**STEP 4**
Identification of performance Levels for all the sub systems (connection, piping…)

**STEP 5**
Identification of the seismic risk indicator \( \rho \) for each vessel sub-system for all the records and for all the time steps of the analysis given a selected Performance Level:

\[
\rho_{in,f/PL,i} = \frac{D_{in,f/i}}{C_{f/PL}}
\]

\( in \) = index for seismic intensity (return period)
\( f \) = index for failure mechanism (piping or base connection)
\( i \) = index for the \( i \)-th realizations (80 realizations for each class)
\( D \) = maximum rotation demand for a single \( i \)-th vessel as envelope among the different time steps
\( C \) = vessel capacity

**STEP 6**
Identification of the seismic risk indicator \( \rho \) for a given PL from the envelope of the seismic risk indicators of all the damage mechanism and for all the vessels:

\[
Y_{in/PL,i} = \max(\rho_{in,f/PL,i})
\]

**STEP 7**
Definition of the number of failures, \( Y_{n,De/PL,i} \geq 1 \), for a given performance level

**STEP 8**
Lognormal fit of the number of failures following the approach reported in Baker J. W. (2013):
3.4.2 Definition of the vessel properties and simulated design

Once defined 10 vessels for each class by randomly vary the slenderness ratio H/R, the wall thickness of each vessel has been estimated with the following equation (Moss, 2004):

\[ t_p = \frac{P \cdot R}{S \cdot E + 0.4 \cdot P} + C \]

Where:

- \( P \) = internal pressure of the fluid containment = 190 kPa;
- \( R \) = radius of the vessel;
- \( S \) = allowable steel stress = 95 MPa;
- \( E \) = joint efficiency factor = 0.85;
- \( C \) = thickness for corrosion allowance = 3 mm.

The geometrical properties obtained are reported in the following tables for all the statistically independent vessels by distinguishing between the two fragility classes assumed.

**Table 3-10: Geometry for the 10 statistically independent vertical vessels of CL1**

<table>
<thead>
<tr>
<th>Vessel</th>
<th>Height (m)</th>
<th>Radius (m)</th>
<th>H/R</th>
<th>Skirt Height (m)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.00</td>
<td>2.31</td>
<td>4.32</td>
<td>2.00</td>
<td>164.87</td>
</tr>
<tr>
<td>2</td>
<td>10.00</td>
<td>2.41</td>
<td>4.16</td>
<td>2.00</td>
<td>178.10</td>
</tr>
<tr>
<td>3</td>
<td>10.00</td>
<td>1.63</td>
<td>6.15</td>
<td>2.00</td>
<td>81.46</td>
</tr>
<tr>
<td>4</td>
<td>10.00</td>
<td>2.41</td>
<td>4.14</td>
<td>2.00</td>
<td>179.23</td>
</tr>
<tr>
<td>5</td>
<td>10.00</td>
<td>2.13</td>
<td>4.69</td>
<td>2.00</td>
<td>139.92</td>
</tr>
<tr>
<td>6</td>
<td>10.00</td>
<td>1.60</td>
<td>6.26</td>
<td>2.00</td>
<td>78.53</td>
</tr>
<tr>
<td>7</td>
<td>10.00</td>
<td>1.78</td>
<td>5.62</td>
<td>2.00</td>
<td>97.33</td>
</tr>
<tr>
<td>8</td>
<td>10.00</td>
<td>2.05</td>
<td>4.89</td>
<td>2.00</td>
<td>128.93</td>
</tr>
<tr>
<td>9</td>
<td>10.00</td>
<td>2.46</td>
<td>4.07</td>
<td>2.00</td>
<td>185.84</td>
</tr>
<tr>
<td>10</td>
<td>10.00</td>
<td>2.46</td>
<td>4.06</td>
<td>2.00</td>
<td>186.96</td>
</tr>
</tbody>
</table>

**Table 3-11: Geometry for the 10 statistically independent vertical vessels of CL2**

<table>
<thead>
<tr>
<th>Vessel</th>
<th>Height (m)</th>
<th>Radius (m)</th>
<th>H/R</th>
<th>Skirt Height (m)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.00</td>
<td>1.40</td>
<td>7.14</td>
<td>2.00</td>
<td>60.42</td>
</tr>
<tr>
<td>2</td>
<td>10.00</td>
<td>1.00</td>
<td>10.02</td>
<td>2.00</td>
<td>30.66</td>
</tr>
<tr>
<td>3</td>
<td>10.00</td>
<td>1.53</td>
<td>6.55</td>
<td>2.00</td>
<td>71.75</td>
</tr>
<tr>
<td>4</td>
<td>10.00</td>
<td>1.58</td>
<td>6.32</td>
<td>2.00</td>
<td>77.02</td>
</tr>
<tr>
<td>5</td>
<td>10.00</td>
<td>1.42</td>
<td>7.06</td>
<td>2.00</td>
<td>61.72</td>
</tr>
<tr>
<td>6</td>
<td>10.00</td>
<td>1.47</td>
<td>6.81</td>
<td>2.00</td>
<td>66.27</td>
</tr>
<tr>
<td>7</td>
<td>10.00</td>
<td>1.46</td>
<td>6.86</td>
<td>2.00</td>
<td>65.42</td>
</tr>
<tr>
<td>8</td>
<td>10.00</td>
<td>1.23</td>
<td>8.13</td>
<td>2.00</td>
<td>46.55</td>
</tr>
<tr>
<td>9</td>
<td>10.00</td>
<td>1.40</td>
<td>7.14</td>
<td>2.00</td>
<td>60.40</td>
</tr>
<tr>
<td>10</td>
<td>10.00</td>
<td>1.09</td>
<td>4.06</td>
<td>2.00</td>
<td>186.96</td>
</tr>
</tbody>
</table>
### Table 3-12: Masses for the 10 statistically independent vertical vessels of CL1

<table>
<thead>
<tr>
<th>Vessel</th>
<th>Pressure</th>
<th>Acrylonitrile density</th>
<th>Benzene density</th>
<th>Fluid Mass</th>
<th>Vessel Thickness</th>
<th>Percentage connections weight</th>
<th>Vessel Mass</th>
<th>Total Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>kPa</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kg</td>
<td>mm</td>
<td></td>
<td>kg</td>
<td>kg</td>
</tr>
<tr>
<td>1</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>101131</td>
<td>8.43</td>
<td>1.1</td>
<td>15491</td>
<td>116622</td>
</tr>
<tr>
<td>2</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>109245</td>
<td>8.65</td>
<td>1.1</td>
<td>16435</td>
<td>125680</td>
</tr>
<tr>
<td>3</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>49964</td>
<td>6.82</td>
<td>1.1</td>
<td>9401</td>
<td>59365</td>
</tr>
<tr>
<td>4</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>109935</td>
<td>8.66</td>
<td>1.1</td>
<td>16515</td>
<td>126450</td>
</tr>
<tr>
<td>5</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>85824</td>
<td>8.00</td>
<td>1.1</td>
<td>13704</td>
<td>99527</td>
</tr>
<tr>
<td>6</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>48171</td>
<td>6.75</td>
<td>1.1</td>
<td>9177</td>
<td>57349</td>
</tr>
<tr>
<td>7</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>59703</td>
<td>7.17</td>
<td>1.1</td>
<td>10598</td>
<td>70301</td>
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<tr>
<td>8</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>79081</td>
<td>7.80</td>
<td>1.1</td>
<td>12911</td>
<td>91992</td>
</tr>
<tr>
<td>9</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>113992</td>
<td>8.77</td>
<td>1.1</td>
<td>16987</td>
<td>130979</td>
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<tr>
<td>10</td>
<td>190</td>
<td>876</td>
<td>0.604</td>
<td>114678</td>
<td>8.79</td>
<td>1.1</td>
<td>17066</td>
<td>131745</td>
</tr>
</tbody>
</table>

The design of the skirt thickness has been carried out with the aim to assure a skirt strength higher than the one of the base connection. Both strength and buckling safety have been carried out by following the prescriptions provided by Eurocode 3 – Part 1-6 (1993) and Eurocode 8 Part 4 (1998), respectively.

The simulated design of the vertical vessels followed the indications provided by FEMA P751 (2011) for the definition of the fundamental period of vibration referring to the case of uniform vessels:

\[ T = 0.0001983 \cdot \left( \frac{H}{D} \right)^2 \cdot \sqrt{\frac{wD}{t}} \]
Where:

H = overall height of the vessel (mm);
D = diameter of the vessel (mm);
t = thickness of the vessel (mm);
w = weight of the vessel (kg/mm), assumed uniformly distributed over the height.

The definition of the seismic actions acting on each vessel followed the prescriptions provided by the National Italian Code (D.M. 16/01/1996) valid in the assumed period of constructions. This assumption reflects the choice to simulate the assessment of existing vessels built in 2000. The hypotheses of shear distribution along the vessel height provided by FEMA P751 (2011) have been assumed, as described in Figure 3-13. Table 3-14 and Table 3-15 report the seismic parameters defined for the vertical vessels of both the fragility class.

Figure 3-13: Hypothesis of shear distribution along the vessel height for seismic actions (FEMA P751, 2011)
### Table 3-14: Seismic parameters for vertical vessels of CL1

<table>
<thead>
<tr>
<th>Vessel</th>
<th>T , sec</th>
<th>R</th>
<th>Seismic protection coefficient</th>
<th>C – Seismic coefficient</th>
<th>K - Distribution coefficient</th>
<th>Lever Arm, m</th>
<th>Base Shear, kN</th>
<th>Bending Moment, kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>0.099</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>116.50</td>
<td>932.02</td>
</tr>
<tr>
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<td>0.096</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>125.55</td>
<td>1004.41</td>
</tr>
<tr>
<td>3</td>
<td>0.133</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>59.30</td>
<td>474.44</td>
</tr>
<tr>
<td>4</td>
<td>0.095</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>126.32</td>
<td>1010.57</td>
</tr>
<tr>
<td>5</td>
<td>0.106</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>99.43</td>
<td>795.41</td>
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<td>6</td>
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<td>8.00</td>
<td>57.29</td>
<td>458.32</td>
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<td>0.07</td>
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<td>8.00</td>
<td>70.23</td>
<td>561.83</td>
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<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
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<td>8.00</td>
<td>91.90</td>
<td>735.19</td>
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<tr>
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<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>130.85</td>
<td>1046.76</td>
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<tr>
<td>10</td>
<td>0.094</td>
<td>1.00</td>
<td>1.40</td>
<td>0.07</td>
<td>1.00</td>
<td>8.00</td>
<td>131.61</td>
<td>1052.88</td>
</tr>
</tbody>
</table>

### Table 3-15: Seismic parameters for vertical vessels of CL2

<table>
<thead>
<tr>
<th>Vessel</th>
<th>T , sec</th>
<th>R</th>
<th>Seismic protection coefficient</th>
<th>C – Seismic coefficient</th>
<th>K - Distribution coefficient</th>
<th>Lever Arm, m</th>
<th>Base Shear, kN</th>
<th>Bending Moment, kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>-</td>
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The results obtained in the simulated design have been compared with the results provided by the numerical models, having particular attention to the fundamental period of vibration. Great differences characterize the fundamental periods of vibration provided by the numerical models with respect to the ones predicted by the simulated design. The motivation of these differences is the assumption regarding the base connection. The simulated design refers, in fact, to a fixed base constraint, while a multi-linear plastic link has been used to simulate the behavior of the base connection in the numerical models. This lead to an increase of the fundamental period in the numerical models of about 3 times the predicted values.
3.4.3 Definition of the connection properties

The base connections of all the vessels have been assumed as composed by a base plate welded to the vessel and anchored to the r.c. foundation by means of steel anchor bolts. The connection design followed the indications provided by Cook et al. (2001). The bolts failure has been assumed as the only admissible failure mechanism since the base plate thickness has been set such that its resistance is bigger than the resistance of the bolts. For these elements, the shear resistance has been estimated as the minimum between the pure shear resistance of the bolts and the shear resistance due to the flexural mechanism of the connection:

\[ V_{RD,\text{conn}} = \min(V_{RD,\text{bolts}}, \frac{M_{RD,\text{conn}}}{H}) \]

The shear resistance of the bolts has been estimated with the following equation:

\[ V_{RD,\text{bolts}} = nA_{s,bolt}f_{yk} \]

Where:
- \( n \) = total number of bolts;
- \( A_{s,bolt} \) = shear area of the single bolt;
- \( f_{yk} \) = allowable shear stress in the bolt material.

Then the resisting moment of the connection has been estimated as the minimum between the resisting moment due to the bolts and the one referred to the base plate:

\[ M_{RD,\text{conn}} = \min(M_{rd,\text{bolts}}, M_{rd,\text{plate}}) \]

In this formula, the resisting moment of the anchor bolts has been computed with the following formula:

\[ M_{rd,\text{bolts}} = A_{bolts} \cdot f_y \cdot (D - 2c_0) + 0.5 \cdot P \cdot (D - 2c_0) \]

Where:
- \( A_{bolts} \) = resisting area of the bolts;
- \( f_y \) = equivalent yielding stress;
- \( D \) = diameter of the vessel;
- \( P \) = acting axial force.

The resisting moment of the base plate has been computed with the following formula:

\[ M_{rd,\text{plate}} = f_y \cdot t^2 \cdot \frac{r_p r_b}{r_b - r_p} \]

Where:
- \( f_y \) = yielding stress of the base plate;
- \( t \) = thickness of the base plate;
- \( r_p \) = external radius of the vessel;
- \( r_b \) = distance from the center of the plate to the centerline of the anchor bolts.
For all the vessels the dominant failure mechanism is the flexural collapse of the bolts, this leads to the choice to model the base connection with a multi-linear plastic link associated to a non-linear moment – rotation law. The moment – rotation law assumed for this element has been a kinematic cyclic law defined in accordance with the indications provided by Cook et al. (2001). The bending moment is assumed as the minimum between the resisting moment of the bolts and the base plate. The rotation capacity of the connection accounts for both the elastic deformation of the anchor bolts and the rotation of the base plate under the applied bending moment:

$$\theta_{conn} = \theta_{bolts} + \theta_{plate} = \frac{2ML_b}{n r_b^2 A_b E_b} + \frac{45M}{E r_b^2} \left( \frac{r_b - r_p}{t} \right)^{1.83}$$

Where:

- \( M \) = applied bending moment;
- \( L_b \) = length of bolt from top of the plate to embedded head of anchor bolt;
- \( n \) = number of anchor bolts;
- \( A_b \) = cross-sectional area of the anchor bolt;
- \( E_b \) = modulus of elasticity of the anchor bolt;
- \( E \) = modulus of elasticity of the plate;
- \( r_b \) = distance from the center of the plate to the centerline of the anchor bolts;
- \( r_p \) = distance from the center of the plate to the centerline of the anchor bolts;
- \( t \) = thickness of the base plate.

The design of the bolted connection for all the 10 vessels of the two fragility classes had been carried out by assuming the seismic input obtained by the application of the National Italian Code DM 1996. The following tables summarize the assumptions for the design of the bolted connection and the results obtained for the moment – rotation law of the base connection. The connection properties have been defined such that their safety factors with respect to the seismic intensity defined by the Italian Code DM1996 are similar for all the vessels.

### Table 3-16: Design of the base connection for vertical vessels of CL1

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<tr>
<th>N°</th>
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<th>( s_{bolts} )</th>
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<th>Bolt Shear Strength</th>
<th>Plate and Skirt Strength</th>
<th>Nbolts</th>
<th>Iplate</th>
<th>Iskirt</th>
<th>M&amp;d,Bolts</th>
<th>M&amp;d,plate</th>
<th>V&amp;d,Bolts</th>
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### Table 3-17: Design of the base connection for vertical vessels of CL2

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<th>Ikirt</th>
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### Table 3-18: Moment – Rotation law for the base connection of the vertical vessels of CL1

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<th>(\theta_{y,plate})</th>
<th>(\theta_{y,Tot})</th>
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### Table 3-19: Moment – Rotation law for the base connection of the vertical vessels of CL2

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<th>Vessel</th>
<th>Minimum (M_{Rd})</th>
<th>(\theta_{y,bolts})</th>
<th>(\theta_{y,plate})</th>
<th>(\theta_{y,Tot})</th>
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A further procedure for the definition of the rotational stiffness of bolted circular flange joints can be found in Couchaux (2010) and Couchaux et al (2011). This method starts from an attempt of simplification of the model proposed by Kozlowski & Wojnar (2008) to determine the rotational stiffness of bolted circular joints. The model proposes a full characterization of the moment – rotation curve for the specific case of bolted circular flange joints by defining both bending resistance and the initial rotational stiffness, as suggested in EN 1993-1-8 (2005). In this approach, the initial rotational stiffness of the joint is estimated by applying the component method, while the bending resistance of joint is estimated by assuming alternatively the occurrence of a fully plastic mechanism or an elasto-plastic regime, as described in the following figure.

![Figure 3-14: Possible failure mechanisms (a – ductile failure mechanism, b – non-ductile failure mechanism) in a bolted circular flange joint according to Couchaux et al. (2011)](image)

With reference to the ductile failure mechanisms, the section is initially studied by defining the position of the neutral axis from the equilibrium of applied axial forces:

\[
\frac{\alpha}{\pi} = \frac{N + N_{T,pl}}{N_{C,pl} + N_{T,pl}}
\]

Where:

- \(N_{T,pl}\) = tensile resistance of the joint;
- \(N_{c,pl}\) = compressive resistance of the tube;
- \(\alpha\) = angle defining the position of the neutral axis.
Then the plastic bending moment can be defined as a function of the angle \( \alpha \) and the acting axial force \( N \) and computed as follows:

\[
M_{j,pl} = \frac{N_{T,pl} \cdot R + N_{C,pl} \cdot R}{\pi} \sin \alpha
\]

In case of non-ductile failure mechanisms, two possible behavior modes are taken into account, one corresponding to the case of dominant bending moment and one corresponding to the case of dominant axial force. In the first case, the joint is subjected contemporarily to compression and tension, while in the second case the joint is completely compressed or tense.

Referring to the case of dominant bending moment, it is possible to compute the bending moment applied by the tube to the supporting flange as follows:

\[
M_{j} = \theta_{j} R^{3} \left\{ (k_{t} - k_{c}) \left( \frac{\sin \alpha}{2} - \alpha \right) + k_{t} \pi \right\}
\]

Where:

\( \theta_{j} = \) rotation acting on the tube cross-section;

\( R = \) tube radius;

\( k_{t} = \) stiffness coefficient per unit length in the tensile zone;

\( k_{c} = \) stiffness coefficient per unit length in the compressive zone.

Thus the initial rotational rigidity of the joint can be computed as follows:

\[
S_{j,ini} = \frac{M_{j}}{\theta_{j}} = R^{3} \left\{ (k_{t} - k_{c}) \left( \frac{\sin \alpha}{2} - \alpha \right) + k_{t} \pi \right\}
\]

Finally, in the case of dominant axial the rotational stiffness \( k \) is the same along the circumference of the joint and equivalent to \( k_{t} \) or \( k_{c} \) in case of joint fully tense or compressed, respectively. The bending moment and the corresponding initial rotational stiffness can be estimated as follows:

\[
M_{j} = k R^{3} \pi \theta_{j}
\]

\[
S_{j,ini} = \frac{M_{j}}{\theta_{j}} = k R^{3} \pi
\]

The moment – rotation law derived by Couchaux et al (2011) assumes the typical shape reported in the following picture and it can be adjusted for each joint as a function of the actual loading state.
Figure 3-15: Example of moment – rotation curve for a bolted circular joint (Couchaux et al., 2011)

The calculation reported in this report follows the approach developed in Cook et al. (2001) and the previous steps based on the work done by Couchaux et al. (2011) have been presented for reference only.

Details of the component approach for the definition of the stiffness of steel joints is reported in EN 1993-1-8 (2005) but base plate connections are treated only with reference to I sections.

3.4.4 Definition of the engineering demand parameters for the vertical vessels

The definition of the engineering demand parameters start from the following assumptions:

- the skirt has been designed with a resistance bigger than the connection strength;
- the thickness of the vessel wall is governed by the internal pressure and temperature and is not affected by the seismic action;
- both the flexural and the shear failure mechanism of the connections are considered in the vessel model: the shear mechanism is checked “a priori” in the simulated design.

The definition of the engineering demand parameters has taken into account the possible failures induced by external actions in correspondence of the base connection and connected piping. With this aim, the rotation capacity at base, mid-height and top of the vessel have been assumed as EDPs.

Regarding the first EDP (maximum rotation at the base $\theta_{y,\text{base}}$), the choice of this parameter is a consequence of the design assumptions, the base connection has been in fact designed to be the weakest component of the vessel since this reflects the damages observed in previous earthquakes. The anchorage failure represents, besides, a recurrent failure mechanism in vertical vessels subjected to seismic actions.
The selection of the second and third EDPs (maximum rotation at mid-height and top $\theta_{y,0.5H}$ and $\theta_{y,H}$) takes into account the observed damages in piping system. The experience suggests, in fact, a great recurrence of joint connections collapses in case of piping systems connected to vertical vessels.

The rotational demand of all the statistically independent vessels has been derived by the numerical models developed. These models, initially defined for 20 vessels divided in two fragility classes, have been submitted to non-linear dynamic analyses with incremental seismic input. The incremental dynamic analysis (IDA) is a parametric method in which the structural system is submitted to multiple non-linear dynamic analyses by applying a set of ground motion records, each of them scaled to increasing seismic intensities (Vamvatsikos and Cornell, 2002).

In the case of the vertical vessels defined here, a set of 8 real acceleration time-history have been assumed assuring a good compatibility of spectra in the range of periods between 0.1 and 4.0 sec. The corresponding response spectra of acceleration have been scaled with respect to 6 different return periods ($T1 = 30$ years, $T2 = 50$ years, $T3 = 475$ years, $T4 = 975$ years, $T5 = 1400$ years, $T6 = 2475$ years).

As a result of this, a total of 80 analyses have been developed for each class and for each seismic intensity (10 vessels and 8 records). Incremental dynamic analyses have been developed with simplified cantilever models, in SAP2000, in which the vessel has been modeled with beam elements with nonlinear hinge at the base.

3.4.5 Definition of the damage limits for the different Performance Levels, PL

The definition of fragility curves for the vertical vessels under consideration needs the identification of specific performance levels. The selection of adequate performance levels, made in accordance with the recurrent damages observed in past earthquakes for vertical vessels, leads to the following PLs:

- **PL1:** corresponding to minor damages of the vessels’ components that could induce the first leakage of the fluid content;
- **PL2:** corresponding to the global collapse of the vessel and the consequently complete release of the fluid content.

Since the base connection represents the weakest component of the designed model, the two performance levels have been converted into the corresponding limit states of the base connection and referred to its rotational capacity. At the same time, the connection with piping systems has been also taken into account by defining two limit states for the rotation capacity of the vessel at mid-height and top.

For the connection capacity, the limits states have been expressed in terms of rotational capacity. More specifically, for the PL1 the limit of rotational capacity is set to the value of yielding rotation defined according to Cook et al. (2001). For the PL2 a limit of rotational capacity twice the yielding limit has been assumed in accordance with the acceptance criteria of plastic rotation for structural steel components provided by ASCE 41-06 (2007). Specific indications for the rotational capacity of steel joints to be applied in performance based design are reported in Roldan et al (2014) but circular base plate connections are not included.

Regarding the piping capacity, it has been assumed equivalence between the rotation of the vessel and the one of the connected piping. In this condition, the limits indicated in literature and in some
experimental tests for piping safety have been assumed as reference. The same have been checked in two critical positions along the vessel height, at mid-height and at top. For the first leakage of the fluid content, a limit rotation of 0.003 rad has been assumed, while for the global collapse of the joint connection between vessel and pipes the limit rotation has been fixed to 0.02 rad. Both limits are very conservative since very few indications are provided in the scientific literature and limited experimental tests have been carried out on this kind of non-structural components; these assumptions lead to a dominant failure of the base connection for all the vessels considered.

The following table summarizes the EDPs and the relative limits for all the damage states considered in this study.

Table 3-20: Final damage limits for the Performance Levels

<table>
<thead>
<tr>
<th></th>
<th>Piping</th>
<th>Base Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL1</td>
<td>$\theta = 0.003$ rad</td>
<td>$\theta = \theta_{\text{conn}}$</td>
</tr>
<tr>
<td>PL2</td>
<td>$\theta = 0.02$ rad</td>
<td>$\theta = 2\theta_{\text{conn}}$</td>
</tr>
</tbody>
</table>

3.4.6 Fragility curves fitting

The last step of the present study corresponds to the fragility curves fitting based on the results provided by the IDA carried out on the 20 statistically independent vessels defined above. The data provided by the IDA can be used to compute the probability of reaching or exceeding the specific EDP and then expressed in form of fragility curves. In the present study, once defined the EDP corresponding to each damage failure, the seismic risk index has been defined as follows:

$$\rho_{\text{in},f/\text{PL}/i} = \frac{D_{\text{in},f/i}}{C_{f/\text{PL}}}$$

Where
- $\text{in}$ = index for seismic intensity (return period);
- $f$ = index for failure mechanism (piping or base connection);
- $i$ = index for the $i$-th statistically independent vessel;
- $D$ = maximum rotation demand for a single $i$-th vessel as envelope among the different time steps;
- $C$ = vessel capacity.

Once estimated the seismic risk index $\rho$ expressing the ratio between the seismic demand and the rotational capacity of the vessel, the risk indicator $Y$ has been computed from the envelope of the seismic risk index obtained for all the damage states:

$$Y_{\text{in}/\text{PL}/i} = \max(\rho_{\text{in},f/\text{PL}/i})$$

With the values of the risk indicators, calculated for the two performance levels of all the vessels and for all the seismic intensities, the number of failures has been estimated as the number of cases exceeding the seismic capacity of the structural system ($Y > 1$). The results have been then fitted by following the approach provided by Baker (2013) to obtain the fragility curves corresponding to each performance level and to each class of vessels. As already stated above, the vessels here analyzed have been characterized by a dominant collapse of the base connection; the base connection collapse
represented the bigger likelihood failure mechanisms in accordance with the observed damages in past earthquake.

The results obtained for the fragility curves fitting are affected by strong uncertainties resulting from modelling and analysis assumptions. With the aim to take into account all the possible uncertainties affecting the data, two further types of uncertainties have been added to the curves data. The first uncertainty regards the modelling epistemic uncertainty (\( \beta_m \)) and it is assumed equal to 0.40. The second uncertainty regards the ground motion variability (\( \beta_{gm} \)) and it is assumed equal to 0.10. Both the uncertainties have been defined by following the indications provided in ATC 58 (2012), then the same have been summed as follows:

\[
\beta = \sqrt{\beta_{\text{analyses}}^2 + \beta_m^2 + \beta_{gm}^2}
\]

The procedure described above leads to the fragility curves reported in the following graphs, while the following table summarizes the parameters of the same curves for each performance level and for each fragility class of vessels.

### Table 3-21: Horizontal vessels fragility curves parameters

<table>
<thead>
<tr>
<th></th>
<th>CL1</th>
<th></th>
<th>CL2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PL1</td>
<td>PL2</td>
<td>PL1</td>
<td>PL2</td>
</tr>
<tr>
<td>PGA(_m)</td>
<td>0.25</td>
<td>0.51</td>
<td>0.27</td>
<td>0.59</td>
</tr>
<tr>
<td>(\beta_{\text{analyses}})</td>
<td>0.09</td>
<td>0.19</td>
<td>0.07</td>
<td>0.04</td>
</tr>
<tr>
<td>(\beta_m)</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>(\beta_{gm})</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>(\beta)</td>
<td>0.42</td>
<td>0.45</td>
<td>0.42</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Figure 3-16: Fragility curves for PL1
The fragility curves obtained show a weaker performance for the vessels of CL1 with respect to the ones of CL2. However, the two fragility classes show a very similar seismic performance since the minimum median PGA assumes very close values between the two classes for both performance levels. A slight greater difference characterizes the performance of the two fragility classes with reference to PL2, in this cases the vessels of CL2 are submitted to a greater seismic input as a consequence of their fundamental period of vibration. At the same time, these vessels are characterized by a lower rotation capacity as a consequence of the design assumptions, so that the combination of these two factors leads to a better seismic performance of class C2.
4 SEISMIC RISK CALCULATION

For all the storage systems under investigation, the seismic risk in a time window of 1 year has been calculated, in order to find out the output for further multi hazard risk analysis.

The damage levels (or performance levels PL) take into account are:

- PL1: corresponds to the first leakage of the fluid content and minor damage in the structure (tank or vessel);
- PL2: complete release of content and global collapse of the tank or vessel.

The seismic risk is defined by the combination of three different factors:

- hazard, i.e. the probability of occurrence of an earthquake exceeding a certain threshold of intensity magnitude (i.e. PGA) in a given area and in a certain interval of time;
- exposure, i.e. the importance of the object exposed to the risk;
- vulnerability, i.e. the level of damage that a structure can suffer when subjected to an earthquake of a certain level of intensity.

In mathematical terms, seismic risk can be described as the unconditional probability of failure ($P_f$) for a system with resistance $R$, under a seismic load $S$, using the following equation:

$$P_f = \int_0^{+\infty} f_S(S) F_R(S) \, dS$$

where $f_S$ is the probability density function of the ground-motion parameter (which can be obtained through the derivation of the seismic hazard curve) and $F_R(S)$ is the probability that the resistance $R$ is less than a given level of severity, $S$ (in this case represented by the fragility curve). Hence, the annual probability of collapse, for example, can be obtained by combining the probability of exceeding the resistance of the structure to collapse for a given level of ground motion [$F_R(S)$], with the annual probability of obtaining that level of ground motion ($f_S$), and summing this product over all possible levels of ground motion.

The hazard is a parameter that depends exclusively on seismicity of the area. Precisely, the hazard represents the estimation of the expected level of seismic intensity in a certain area and for a given observation period. The definition of seismic hazard occurs through the hazard curve. In particular, the curve relates the severity of shaking, defined in this case by the peak ground acceleration $PGA$, with the Annual Frequency of exceedance $AFE$, given by the inverse of the return period $T_r$.

The logarithm of a ground-motion parameter and the logarithm of the corresponding annual frequency of exceedance can be assumed to be linearly-related, at least for return periods of engineering interest. The negative gradient of the log–log hazard curve is referred to as $k$ in this deliverable, following the definition in Part 1 of Eurocode 8 (CEN, 2003). Thanks to the approximation of linear trend, to define the hazard curve it is sufficient to determine the $PGA$ value, corresponding to a return period $T_r$, and the negative gradient of the log–log hazard curve $k$ that passes through the reference point. Conventionally, it is assumed as a point of passage of the curve that corresponds to the return period $T_r$ of 475 years. The hazard curve is then defined by the following relation:
\[ AFE = AFE_{475} \left( \frac{S \cdot a_{g475}}{a_g} \right)^k \]

where \( AFE_{475} \) and \( PGA_{475} \) are the annual frequency of exceedance and the \( PGA \) corresponding to the return period \( T_r \) of 475 years while \( S \) is the soil factor. Once defined the hazard curve, it is possible to calculate the seismic demand related to \( PGA \) for any return period \( T_r \).

For the area in which Plant A and Plant B are located, in south-eastern Sicily (Italy), the hazard curve reported in Figure 4-1 has been calculated using the data described in Deliverable D.A.2.

Therefore, for the seismic risk calculation the hazard curve on soil is shown in the figure above and the fragility curves for tanks, horizontal vessels and vertical vessels are those described respectively in §2.3, 3.3 and 3.4.

The outputs of this deliverable are:

1) the probability of reaching the performance point PL1 and PL2 in a time window of 1 year for tanks, horizontal vessels and vertical vessels;

2) the mean annual frequency of occurrence of PL1 and PL2 for tanks, horizontal vessels and vertical vessels.

The first output is the seismic risk in 1 year calculated as described in the beginning of this chapter. The mean annual frequency of occurrence, instead, has been calculated using the Jalayer and Cornell (2003) formulation reported below:

\[ \lambda_{DS} = \lambda(S_{a_{r,50\%}}) \exp \left[ \frac{1}{2} (k \beta_{tot})^2 \right] \]
where:

- $\lambda(S_a,50\%DS)$ = mean annual frequency of occurrence of the median value of the fragility curve (obtained from the hazard curve)
- $k$ = exponential of the linear fitting of the hazard curve in log scale
- $\beta_{TOT}$ = dispersion of the fragility curve

### 4.1 SEISMIC RISK AND FREQUENCY OF OCCURRENCE FOR TANKS

In the fragility curves evaluation for tanks, the damage levels considered were four: light damage, moderate damage, severe damage and collapse. Therefore, in order to obtain the seismic risk for the performance point PL1 and PL2, these correspondences have been identified:

- moderate damage corresponds with the first leakage of the connected piping;
- collapse corresponds with the complete release of the content.

In Table 4-1 the probabilities of reaching PL1 and PL2 in a time window of 1 year for each class of tank are reported. Furthermore, in Table 4-2 the mean annual frequencies of occurrence of PL1 and PL2 for each class of tank are shown.

#### Table 4-1: Probability of reaching PL1 and PL2 in a time window of 1 year for tanks

<table>
<thead>
<tr>
<th>Class</th>
<th>Prob PL1 (%)</th>
<th>Prob PL2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>0.448529</td>
<td>0.080212</td>
</tr>
<tr>
<td>Class 2</td>
<td>0.281069</td>
<td>0.050061</td>
</tr>
<tr>
<td>Class 3</td>
<td>0.344832</td>
<td>0.06219</td>
</tr>
<tr>
<td>Class 4</td>
<td>0.422875</td>
<td>0.079481</td>
</tr>
</tbody>
</table>

#### Table 4-2: Mean annual frequency of occurrence of PL1 and PL2 for tanks

<table>
<thead>
<tr>
<th>Class</th>
<th>$\lambda_{PL1}$</th>
<th>$\lambda_{PL2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>0.003791624</td>
<td>0.000675703</td>
</tr>
<tr>
<td>Class 2</td>
<td>0.002307472</td>
<td>0.000411213</td>
</tr>
<tr>
<td>Class 3</td>
<td>0.003199041</td>
<td>0.000570099</td>
</tr>
<tr>
<td>Class 4</td>
<td>0.011055932</td>
<td>0.001970272</td>
</tr>
</tbody>
</table>

Please note that, as described in §2.3, the classes of storage tanks considered in this project are:

- class 1: $0.7 \leq D/H \leq 1$;
- class 2: $1 < D/H \leq 1.5$;
- class 3: $1.5 < D/H \leq 2$;
• class 4: D/H > 2.

4.2 SEISMIC RISK AND FREQUENCY OF OCCURRENCE FOR HORIZONTAL VESSELS

The evaluation of the seismic risk for the horizontal vessels has been carried out by assuming the following two performance levels:

• PL1: corresponds to the first leakage of the fluid content and minor damage to the vessels structure;
• PL2: global collapse of the vessels and consequent complete release of the fluid content.

The horizontal vessels assumed in the present study have been defined by randomly vary the H/R ratio, by assuming three types of connections and three different moment/rotation curves with lower, medium and best estimate soil parameters. The 90 statistically independent vessels defined are not divided in classes but they have been analyzed by means of equivalent linear static analyses in both the principal direction. As a consequence of these assumptions, the values obtained for the probabilities of reaching one of the two performance levels in a time window of 1 year and the corresponding mean annual frequencies of occurrence of PL1 and PL2 have been defined for each loading direction. The mentioned results are reported in Table 4-3 and Table 4-4.

Table 4-3: Probability of reaching PL1 and PL2 in a time window of 1 year for horizontal vessels

<table>
<thead>
<tr>
<th></th>
<th>Prob PL1 (%)</th>
<th>Prob PL2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Dir.</td>
<td>0.179904</td>
<td>0.095502</td>
</tr>
<tr>
<td>Longitudinal Dir.</td>
<td>0.093867</td>
<td>0.057020</td>
</tr>
</tbody>
</table>

Table 4-4: Mean annual frequency of occurrence of PL1 and PL2 for horizontal vessels

<table>
<thead>
<tr>
<th></th>
<th>( \lambda ) PL1</th>
<th>( \lambda ) PL2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Dir.</td>
<td>0.00180066</td>
<td>0.00095548</td>
</tr>
<tr>
<td>Longitudinal Dir.</td>
<td>0.00093911</td>
<td>0.00057036</td>
</tr>
</tbody>
</table>

4.3 SEISMIC RISK AND FREQUENCY OF OCCURRENCE FOR VERTICAL VESSELS

Also in the case of the vertical vessels, the damage levels (or performance levels PL) taken into account for the evaluation of the seismic risk are the following:

• PL1: corresponds to the first leakage of the fluid content and minor damage to the vessels structure;
• PL2: global collapse of the vessels and consequent complete release of the fluid content.
The set of vertical vessels assumed in the present study are divided into two classes as a function of the H/R ratio. More specifically the following classes have been assumed:

- Class 1: vertical vessels with $4 < \frac{H}{R} \leq 7$;
- Class 2: vertical vessels with $7 < \frac{H}{R} \leq 11$.

By applying the procedure described at the beginning of the chapter, the following values have been obtained for the probabilities of reaching one of the two performance levels in a time window of 1 year and the corresponding mean annual frequencies of occurrence of PL1 and PL2. The mentioned results are reported in Table 4-5 and Table 4-6 for both the classes of vessels considered, respectively.

Table 4-5: Probability of reaching PL1 and PL2 in a time window of 1 year for vertical vessels

<table>
<thead>
<tr>
<th>Class</th>
<th>Prob PL1 (%)</th>
<th>Prob PL2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>0.331030</td>
<td>0.100850</td>
</tr>
<tr>
<td>Class 2</td>
<td>0.288505</td>
<td>0.074485</td>
</tr>
</tbody>
</table>

Table 4-6: Mean annual frequency of occurrence of PL1 and PL2 for vertical vessels

<table>
<thead>
<tr>
<th>Class</th>
<th>$\lambda$ PL1</th>
<th>$\lambda$ PL2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>0.00331580</td>
<td>0.00100901</td>
</tr>
<tr>
<td>Class 2</td>
<td>0.00288922</td>
<td>0.00074513</td>
</tr>
</tbody>
</table>
5 CONCLUSIONS

The present study regards the analysis of risk induced by seismic input on typical structural components present in petro-chemical plants and usually devoted to storage of contaminant fluids. The attention is therefore focused on three types of storage systems: tanks, horizontal and vertical vessels.

The seismic analysis of these structural components has been carried out with the goal of assessing the seismic vulnerability of existing structures. For this reason, a great attention has been devoted to the inventory of recurrent damage mechanisms that have been observed on such structural elements in past earthquakes. The observed failure mechanisms, in addition to the indications provided by the international standards available, have represented a useful guide in the definition of the limit damage states. The final goal has been the identification of those damages that could lead to the spilling of the contaminant fluids contained in the structures under consideration.

The procedures described here have been applied to mechanical models defined in order to be representative of existing storage systems in petro-chemical plants. With this aim, the following modelling assumptions have been assumed for the structures under consideration:

- **Tanks** characterized by two possible soil - structure configuration (anchored and not-anchored tanks) and distinguished in a varying number of classes as a function of the D/H ratio; in the case of anchored tanks two case studies have been considered representative of squat (0.5 < H/R < 1.5) and slender (1.5 < H/R < 6) tanks, respectively;

- 90 statistically independent **horizontal vessels** defined with 10 different radii, 3 connection typologies (2+2 bolts, 4+4 bolts and 6+6 bolts) and 3 foundation moment/rotation curves with lower, medium and best estimate soil parameters;

- 20 statistically independent **vertical vessels** defined with 10 different, randomly determined radii, grouped in 2 classes characterized by two different H/R ratios (4<H/R≤7 and 7<H/R≤11).

For all the statistically independent structures mentioned above, a simulated design has been performed by following the design prescriptions provided by national codes available in the period of construction. Then simplified numerical models have been simulated using FEM procedures. These models have been submitted to different structural analyses, i.e. equivalent linear static analysis and non-linear dynamic, in order to evaluate the seismic capacity of each model.

The evaluation of the seismic vulnerability of existing storage structures stems from the processing of the output data derived by the numerical models and the consequent fitting of fragility curves. Specific procedures for the definition of fragility curves have been described here by deriving indications from the literature, from experimental tests carried out on the structures mentioned above but also from the international standards available. In all studied cases, fragility has been computed as the probability of exceeding a certain damage limit state for effect of a given seismic intensity.

The definition of fragility curves has required a preliminary identification of a set of reference parameters as a defining parameter for the seismic intensity and the engineering demand parameters that allow the definition of the seismic capacity of the structural systems under consideration. The seismic intensity has been defined, for all types of structures mentioned above, in terms of Peak Ground Acceleration (PGA).
For the definition of the engineering demand parameters, particular attention has been devoted to the recurrent damages in past earthquake but also including some indications provided by the international codes. In the case of horizontal and vertical vessels, these parameters have been expressed in terms of rotational capacity and rotational stiffness in critical sections, as the base connection or the sections of hypothetical connection with piping systems. In the case of tanks, the damage mechanisms assumed as more critical have been the buckling of the wall as a consequence of elastic or elastoplastic mechanism (the so-called Diamonds shaped and Elephant foot buckling) and the toppling of the storage tank.

Finally, a careful validation of the fragility curves has been carried out by comparison with curves published in the scientific literature. More specifically, in the case of tanks the fragility curves proposed in Hazus (FEMA 1999) for the two limit states of moderate and severe damage have been assumed as reference for the validation of the results obtained. For all three types of storage structures under consideration, the assessment procedure adopted can be considered reliable even if applied to simplified models, since they have been based on a careful calibration of geometrical and mechanical data and compared with indications provided by the scientific literature. The simplifications adopted in the analyses of the vessels have been developed with the goal of the identification of a tool applicable to a large portfolio of industrial equipments without the aid of time consuming analyses.

For all the storage systems seismic risk, by means of mean annual frequency of exceeding a damage state, and consequences have been derived in order to find out the output for further multi hazard risk analysis.
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