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Mitigation of the Seismic Impact on Storage Gas Tanks by Using Isolation System

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Abstract
The paper presents the studies carried out on a big storage gas tank for the seismic vulnerability assessment and the retrofit through isolation system. This Gasometer, constructed in the early eighties, consists in a steel cylinder of 70 m height and 44 m diameter. The tank bearing structure is composed by a classical steel frame closed with curved steel plates. Inside the tank, a piston-fender and rubber sealing system allow the variation of the container volume and control the gas pressure. The latter represents half of the seismic mass and its position can vary along the tank height. The seismic vulnerability of the steel structure of the Gasometer on its as-it-is state is assessed through the execution of Incremental Dynamic Analyses, placing particular attention on the definition of possible limit states. Subsequently, the seismic retrofit through base isolation system, adopting Double Curved Surface Sliders, is proposed and numerically assessed. The results and comparison between the as-it-is state and the retrofitted state supplies important information about the effectiveness of base isolation for the risk mitigation of gas tanks.

Keywords
seismic protection, seismic retrofitting, storage gas tanks, base isolation, incremental dynamic analysis

1 Introduction

In the past decades several earthquakes, as the Kocaeli (Turkey) earthquake (1999) or more recently the Fukushima (Japan, 2011) and Emilia (Italy, 2012) earthquakes, exposed the seismic vulnerability of industrial plants handling hazardous materials [1–5] affecting not only for the plant itself, but also for the neighborhood population, the economy and the environment. Particularly in Europe, the Emilia earthquake (Italy) highlighted the seismic vulnerability of industrial facilities as reported by [4]. Important economic consequences related to the interruption of production and of the collapse of non-structural elements, such as the racks used to store goods inside the industrial facilities [6, 7], were observed.

The seismic vulnerability of industrial plants is, generally speaking, raised even more due to their complex spatial organization and the high probability of domino effects. Considered the high social and economic impact associated to the shutdown of an industrial plant, the mitigation of the impact of seismic action on such structures has to go beyond the criteria usually adopted for civil buildings.

Recently, several researches focused on the seismic vulnerability and proper retrofit intervention of industrial buildings such as [8–12]. The European research project PROINDUSTRY [13] aimed at developing and setting up innovative seismic protection systems, both for the design of new industrial plants and for the retrofit of existing ones, based on seismic isolation and energy dissipation techniques. Among all the possible geometrical configurations that industrial constructions can assume, the tanks directly connected to the ground, such as the gas tanks, are one of the most complexes to retrofit. Indeed, gas tanks are normally tall cylindrical vessels (Fig. 1) which contain a piston-fender and a rubber sealing system to control the gas pressure. They are very common structures in industrial plants and their seismic vulnerability is usually underestimated due to their low mass density. However, two important aspects make the gas tanks seismic vulnerability assessment very important when compared to other industrial constructions: i) the possibility of storing high toxic and inflammable gas, whose leaking and diffusion

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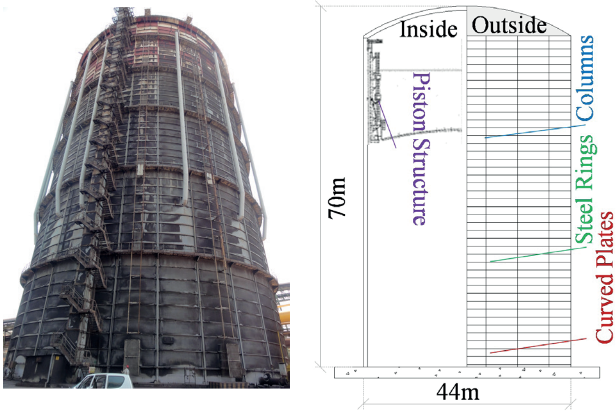

(a) (b)

Fig. 1 Investigated storage furnace gas (AFO) tank [13]; (a) Constructed, (b) Schematic drawing

would cause high danger to the surrounding people and environment; ii) the circumstance that not only the collapse of the structure, but also relative small cracks in the external walls or mechanical elements can cause severe damage and losses. Moreover, failure or significant damage in steel tank may occur in different parts of the structure as identified in [14–19].

The choice of a seismic retrofit or upgrade system shall take into account the peculiar geometry of gas tank structures which makes the application of external energy dissipation devices very difficult and makes the seismic isolation the most suitable solution.

The present paper shows the studies carried out on the mitigation of the seismic risk on a gas tanks through base isolation. A real case study, consisting of an operating gas tank, was used to perform the referred assessment. The seismic protection is provided by the introduction of Double Concave Surfaces Slider (DCSS), as proposed by [20]. The structural performance in both, as-it-is state and retrofitted state, is assessed through Incremental Dynamic Analyses using a suitable set of ground motion records, consistent with the seismic hazard of an Italian high seismicity zone [21]. Particular attention is given to the definition of the possible limit state for gas tanks. Such structures, indeed, even if are characterized by a lower mass compared to other storage systems infilled with liquid or solid materials, have specific features that make the gas tanks very sensitive to the seismic action. The comparison of the numerical results obtained for the as-it-is and retrofitted states highlights the effectiveness of the seismic isolation in reducing the vulnerability of such structures.

2 Base isolation system

The essential function of the base isolation system is to decouple the superstructure from the soil increasing its fundamental vibration period. Consequently, the superstructure response is moved from a high to a low seismic energy content zone of the design spectra maintaining it in the elastic regime, strongly reducing the probability of interruption of operation activities after the earthquake. On the other hand, the displacement demand on the isolating devices increases inversely. In order to reduce such displacement demand, the isolating device can be provided with a mechanism for the dissipation of the energy transmitted by the earthquake, e.g., through inelastic deformations or friction phenomena, or through the combination of isolation with dampers as proposed in [22].

The superstructure with base isolation will move almost as rigid body above the latter. Though, these large displacements have to avoid "conflict" with the surrounding structures and the connections with other infrastructures, as pipelines, have to accommodate the required deformations. The interface between different infrastructures in earthquake situation is an important issue but it is not in the scope of the present paper. In combination with the horizontal flexibility, the isolation system has to provide sufficient vertical bearing capacity and stiffness to transfer the gravity loads and to anchor the structure in the case of uplifting, e.g., due to the vertical component of the seismic action and overturning moment.

There are a large number of isolation devices available in the market, as it can be found in [23], differing in the type of mechanism, such as: elastomeric and lead-rubber bearings, friction-based isolation bearings, hybrid isolation-damping devices, rocking. The present paper focuses on the use of friction-based isolation bearings, namely the double curved surface slider (DCSS), as illustrated in Fig. 2. Such devices incorporate the important feature of the re-centering capacity of the system, essential for structures that cannot suffer severe damage after strong earthquakes and therefore, requiring minor post-earthquake intervention.

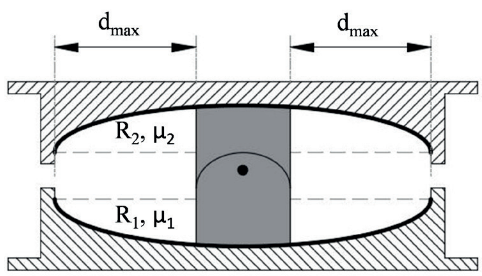

Fig. 2 Double curved surface slider (DCSS)

The oscillation behavior of the DCSS is dependent on different parameters as: i) radii of curvature of the concave surfaces (R_1 and R_2); ii) the friction coefficients of the sliding surfaces (μ_1 and μ_2). Depending on these properties, the response is characterized by a multi-linear hysteric curve as proposed in [20]. In the present study, equal radii of curvature and friction coefficient of the concave surfaces are assumed. Consequently, the hysteric curve presents a bilinear relationship between horizontal force and horizontal displacement (Fig. 3). The dynamic behavior of the isolator is characterized by its oscillation period which is not dependent on the mass but on the length of the pendulum [24]. Also, the period (T) of the isolated structure does not depend on the mass of the structure itself, but mainly depends on the radius R of the curved sliding surface (or the equivalent radius for the DCSS). This can be determined as expressed in Eq. (1), where $d_{\max}$ is the maximum displacement (Fig. 2), g is the gravity acceleration and μ is the friction coefficient. The theoretical bi-linear hysteresis response schematized in Fig. 3 results from a near rigid response ($K_i \approx \infty$) until friction force (μW) is exceeded, where W is the weight of the mass (M) above the isolator. Then, the force increase is proportional to the displacement, according to the dynamic friction stiffness ($K_{IS} = W/R$). Neglecting the influence of friction (μ) [25], the latter can be determined in function of the isolator fundamental vibration period (T) as expressed in Eq. (2).

The re-centering capacity of the friction pendulum, a fundamental requirement for base isolated structure [26, 27], is dependent on the restoring force ($K_{ef}d$) which represents the gravity action towards the equilibrium position and the friction force (μW) that can act away from the origin [28]. Where k_{ef} is effective stiffness and d the isolator motion, as represented in Fig. 3.

$$T = 2\pi\left(1/\left(g\left(1/R + \mu/d_{\max}\right)\right)\right)^{1/2} \quad (1)$$

$$K_{IS} = M\left(2\pi/T\right)^2 \quad (2)$$

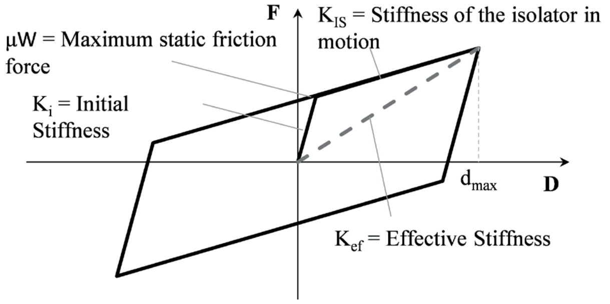

Fig. 3 Hysteresis of the DCSS

3 Case study

3.1 General description

The case study is a Gasometer constructed in the eighties (see Fig. 1) in Italy. The original design of the Gasometer was governed by the wind action, and therefore, the design rules given in the [29] were not applied. The gas tank is used to store gas released from the furnace in a steel making mill (AFO gas). The tank structure has a cylindric form with 70 m height and 44 m of diameter, and consists of a main steel frame composed by columns, steel rings, and roof steel grid. The cladding is realized using curved steel plates. Two main types of structural connections can be identified: i) connections between structural members, including between steel plates and steel profiles, consisting of riveted and welded connections; ii) column bases, anchored to the RC foundation by means of steel anchors. To seal the tank container volume, a mechanical system consisting of a piston-fender and a rubber membrane is used. The latter moves vertically inside the tank allowing the volume variation and consequently, the control of the gas pressure. This system is supported by the gas tank main structure, namely the columns (see Fig. 1(b)). Further details on this case study may be found in [13].

In order to perform the seismic analyses, the position of the piston can strongly influence the global seismic performance of the tank, as it represents half of the total mass. Thus, within this work, the piston was assumed located at the maximum operational height, equal to 2/3 of the total height of the gas tank (≈47 m). The gas tank contains gas at low pressure (400 mm $H_2O \approx 0.039$ atm ≈ 3.92 kPa) and, therefore, its mass, and its action in general, was neglected.

3.2 Numerical modelling

The finite element (FE) numerical model, developed in Abaqus [30], takes into account the stiffness of the main structural members of the tank and in particular the frame (columns, wall rings, roof grid) and cladding plates (wall and roof), as illustrated in Fig. 4. Two type of FE were used to model the tank structure. All linear members, as columns, stiffening rings and roof steel grid were modelled with beam elements (B31). The curved steel plates used in the walls and roof were modelled with shell elements (S4R). Detail on these two FEs may be found in [30].

To include the total mass of gas tank in the model, the mass of the non-explicitly modelled parts were added as non-structural masses and distributed as described in [31]. The connections between the structure members and the steel plates have been modelled as continuous. The connec-

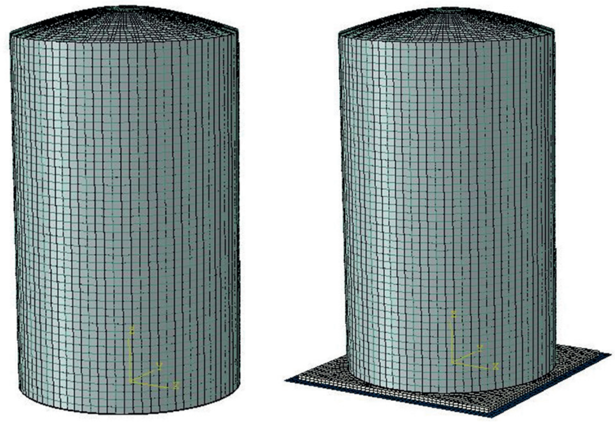

Fig. 4 FE model developed for the investigated gas tank structure; a) Non-isolated Structure, b) Isolated structure

tions to the foundation were considered as pinned supports. For the mechanical properties of the steel, not having the possibility of assessing the real one through experimental tests, the nominal values, according to the [32], were used. All steel grades (S235 for profiles, S275 for shell/plates) were modelled by means of an isotropic elastic-plastic material constitutive model with an yield plateau (15 ε_y) and strain hardening (up to f_u). The geometrical nonlinearities were explicitly taken into account. Finally, in the absence testing results to validate the developed model, the validation of the used modelling techniques was accomplished by partial benchmarking studies realized within the scope of [13] and with particular attention for the modelling of the isolation system described hereafter in Section 4.5.

3.3 Failure criteria

With the purpose of identifying possible failures during the numerical simulations, failure criteria were established, namely: i) ultimate plastic strain ($\varepsilon_{pl} \leq \varepsilon_u = 0.2$); ii) structural instability (global or local); iii) Anchorage of the column base ($R_{uplif} \leq N_{Rd,anchorage}$). To avoid the complexities related to the modelling of the column bases connection, the latter criterion was controlled in a post-processing stage, assuming the failure of the column base as a brittle mechanism. In particular, the uplift reaction forces (R_{uplif}) on the column bases was compared to the anchorage capacity ($N_{Rd,anchorage}$). The latter were computed according to the design rules given in [33, 34].

4 Seismic assessment of the case study

4.1 Preliminary analysis

The seismic behavior of the case study was assessed through dynamic linear, static and dynamic nonlinear analyses. First, in a modal analysis, using the Lanczos method, the fundamental (global) mode is identified for a frequency of 2.64 Hz, with mass participation of 88.65%, and is characterized by a local deformation at the position of the piston mass (2/3 of the height of the gas tank structure).

Considering the high influence of the first vibration mode and in order to have a preliminary assessment of the nonlinear behavior of the tank, a static pushover analysis was carried out. The horizontal loads were applied directly to the columns of the structure at the position of the piston mass and they were incremented up to the fulfilment of the two first failure criteria given in §3.3.

Fig. 5 shows the base shear versus relative gas tank roof displacement (d_{Top}) curve produced with the results of the push-over analysis. It can be seen that the failure of the tank is obtained for very low base shear forces due to the low resistance of the columns base anchoring. This result is consistent with the original design of the case study, in which the governing horizontal load was the wind one and the associated resultant uplift forces, due to the tank overturning, do not exceed the column base resistance. The figure reports also the pushover curve in the hypothetical case of neglecting the failure of the column base, showing that after achieving the maximum resistance, a sudden drop of the base shear is observed due to local loss of stability of the gas tank structure. The ultimate strain is then attained in tank structure members for a substantial lateral displacement measured at the top of the tank.

4.2 Ground motion recordings selection for the Nonlinear Response History Analysis

The seismic assessment of the gas tank structure was based on Nonlinear Response History Analyses (NL RHA) using in particular the Incremental Dynamic Analysis (IDA) method. To this end a set of 11 Ground Motions (GM) was selected for an Italian high seismicity zone, the site of Reggio Calabria, and taken from several strong motion databases available in [35–37]. Accordingly to the results

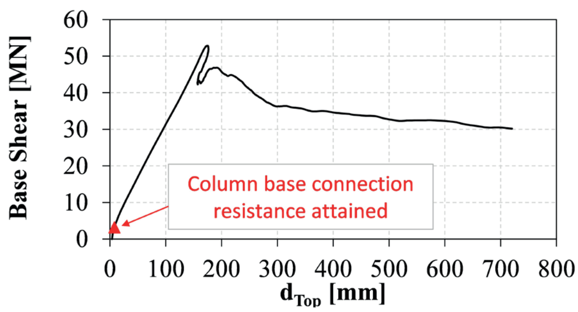

Fig. 5 Push-over base shear-relative roof displacement curve

obtained in [12], the selected ground motion set is coherent with the Uniform Hazard Spectrum spectral shape and the reference spectrum was matched with the geometric mean (GeoMean) of the two horizontal ground motions components, ensuring that the mean spectral ordinates of each set are never lower than 90% of the reference spectrum in the period range between 0 s and 2 s, consistent with the spectrum-matching requirements of the [26] (§3.2.3.1). In this way it was possible to use the same seismic action for both the current state and retrofitted state of the building, limiting thus the influence of the seismic action on the comparison of results. To perform the IDA analyses, the selected ground motions were scaled using 9 scale factors for consistency with the spectrum requirements in [26]. The selected ground-motions consist of two horizontal and one vertical components. In the performed calculations, the three ground-motions components were applied simultaneously. The spectral accelerations and the corresponding scale factors considered in IDA analyses are given in [13].

4.3 Incremental Dynamic Analysis

The IDA showed that, in all cases, the failure criteria related to limit plastic strain and the local or global stability are not violated, even for the highest values of scale factors. On the contrary, in almost all cases and even for lower scale factors, the resistance of the column anchorage to the uplift forces (3rd failure criterion) was surpassed, as shown in Fig. 6(a)). The uplift forces, generated by the tank overturning and vertical component of the seismic action, represent the critical point of the structure which for the strongest ground motions can exceed 10 times the column base resistance. These observations are consistent with the push-over analysis.

In the hypothesis of sufficiently resistant column base connection, the IDA showed the occurrence of plastic deformations in the structure, especially for the highest scale factors. Fig. 6(b)) shows the maximum plastic deformations (PEEQ) observed during the different simulations. These are observed in the structural members, namely columns and ring stiffeners, in the zone where the piston mass was applied (2/3 of the gas tank height). With the increase of the peak ground acceleration is possible to notice damping effects which are due to the increase of plastic deformations in the tank structure. Though the plastic deformations did not exceed the limit strain (ε_u), permanent deformations may be a serious problem for the operation capability of the gas tank after the earthquake and consequently, lead to important economic losses.

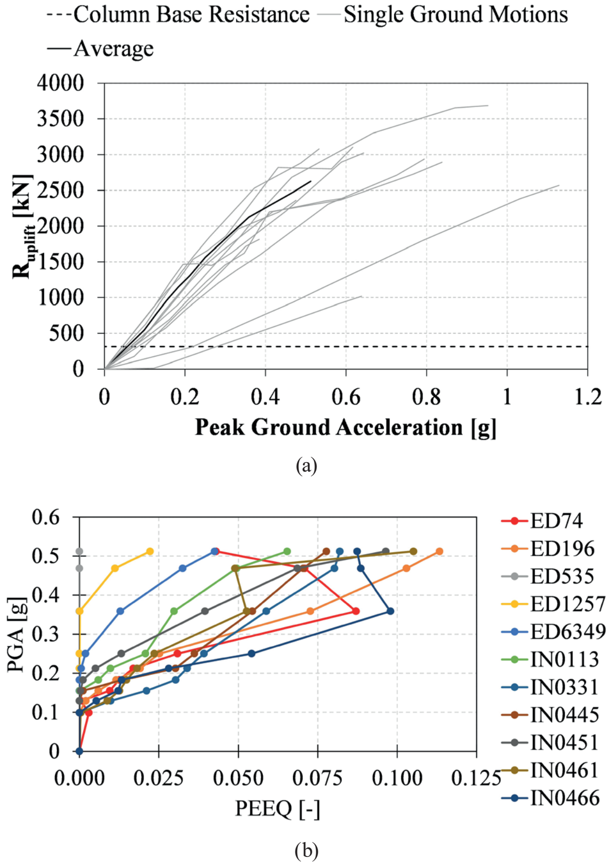

Fig. 6 Incremental dynamic analysis results; (a) IDA curves, (b) Maximum plastic deformations on the non-isolated structure

4.4 Seismic isolation of the case study

To seismically isolate the gas tank at the base level (with base isolation), a new ring foundation, under the existing foundation slab, was assumed (see Fig. 4(b)). Subsequently, the seismic isolators (DCSS) were placed in between these (foundation elements) and in the same position of columns. A total of 28 devices (Isolators) were considered, one for each steel column. The incorporation of the isolators in the numerical model followed the strategy described here after.

4.5 Numerical modelling of DCSS

In order to incorporate the base isolation system in the structural model, a simplified modelling approach was adopted as proposed in [38] and further used in [39]. The considered approach consists in combining a linear elastic spring with a friction element working in parallel. The mechanical behavior of the linear elastic spring is characterized by the isolator stiffness (K_{IS}) to horizontal motion. The mechanical behavior of the friction element is modelled by means of a rigid-plastic law where the static friction (μW) defines the onset of pure plastic deformation. The two described elements are unidirectional and work only on the horizontal (ground) plane. The movement in the gravity direction between the superstructure and the foundation is considered restrained and the possibility of uplifting is checked on a post-processing phase controlling that no vertical tension force is induced in the isolators. The re-centering forces developed with the potential energy due to the structure moving in the vertical direction are taken into account through the elastic spring.

The properties of isolator (DCSS) to determine the mechanical behavior of the referred friction and spring element have been obtained after a calibration process to optimize the isolation performance. The following parameters were subject of variation: i) the friction coefficient (0.03 to 0.04) and ii) the fundamental oscillation period (0.5 s to 3.5 s). The radii of curvature of the concave surfaces ($R_1 = R_2$) was assumed equal. Details of the calibration study may be found in [13]. In Table 1 are given the properties of the DCSS considered for the subsequent analysis.

4.6 Analysis of the efficiency of the base isolation system

In Fig. 7 the global deformation of the gas tank structure with the increase of the peak ground acceleration is shown and compares the response of the original and retrofitted structure. The deformation represents the average value of the relative tank roof displacements. A reduction of approximately 70% is obtained when the base isolation system is used. The maximum roof relative displacement is about 40 mm which is a significantly small value giving the height of the gas tank structure. Furthermore, given the dimensions of these types of facilities, no risk of impact with other infrastructures is expected.

The comparison of the uplift forces evaluated in the current state and in the isolated situation (Fig. 8) shows a reduction up to approximately 9% of the forces obtained in the original structure. This means that the structure overturning is considerably reduced. The remaining uplift forces at the column bases are due to the vertical component of the seismic action. However, isolation systems are not capable of reducing such forces and in the case the structure has not sufficient capacity to bear them, a different type of intervention should be thought.

Table 1 Isolator properties considered for the IDA simulations

M [ton]	W [kN]	μ [-]	T [s]
1581.1	15495.3	0.04	3.0
K_{IS} [kN/m]		Static friction force [kN]	
Total	Per Isolator	Total	Per Isolator
6935	248	620	22.14

Concerning the serviceability conditions (defined as no significant damage that can interrupt the operability of the infrastructure) Fig. 9 shows that plastic deformations are barely existent in the isolated structure. Such plastic deformations in the isolated structure are mainly due, again, to the vertical component of seismic action. Finally, Fig. 10 compares the fragility curves associated to the uplift resistance of the anchorage. It is clearly perceptible that the

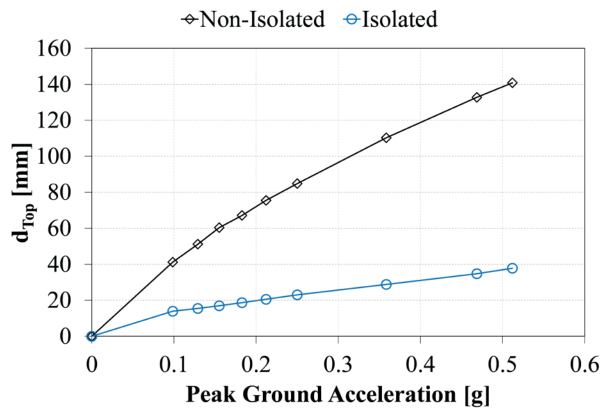

Fig. 7 Comparison between the non-isolated and isolated structure global deformation (Average relative roof displacement)

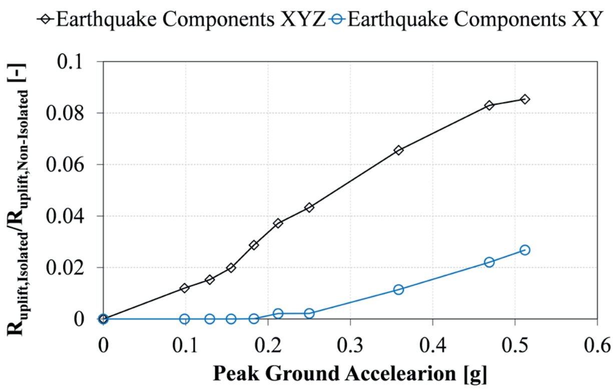

Fig. 8 Comparison between non-isolated and isolated structure – Column base uplift forces

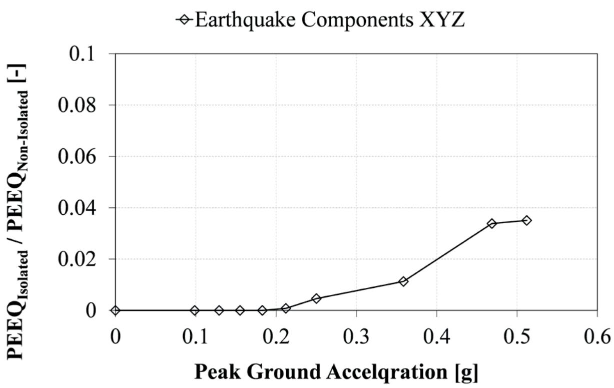

Fig. 9 Comparison between non-isolated and isolated structure – Average maximum plastic deformations

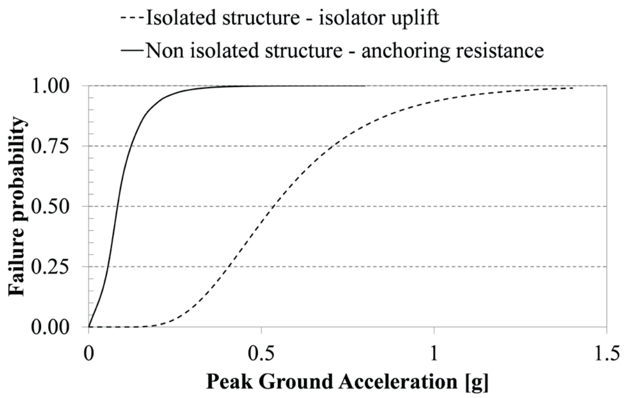

Fig. 10 Comparison of the fragility curves between non-isolated and isolated structure

introduction of base isolation improves significantly the seismic performance of the structure therefore, reducing the risk of severe damage and of reparation works.

5 Conclusions

In the present paper is reported a numerical study aimed at assessing the efficiency of the seismic protection of a real gas tank through a base isolation system. Several types of FEM simulations were performed (modal, push-over and incremental dynamic analysis) to assess the seismic behavior of the case study. These analyses highlighted the typical critical aspects of gas tanks and in particular: i) the brittle failure of column bases, generally not designed to resist to the seismic uplift loads (in this case study for $a_g > 0.06$ g the column base resistance was exceeded), and ii) the induced plastic deformations (up to $\varepsilon_p = 0.1$) on the structural elements bearing the piston-fender and rubber sealing system, representing 50% of the total mass. The former aspect represents an important vulnerability source for the whole structure, especially when considering the collapse prevention. The latter is strictly related to the possibility of interruption of the operation due to gas leakage.

Subsequently, in order to mitigate the impact of the seismic action, the study considered the retrofit of the existing structure by means of a base isolation system, in particular adopting the double curved surface slider. The response of the isolated structure, considering both the collapse prevention and the serviceability limit states, attested the suitability of the foreseen seismic protection: i) seismic uplift loads reduced up to 90% and ii) plastic deformations reduce up to 96%. Important effects of the seismic action on the isolated structure are however present and due to the vertical component to which the currently existing isolation systems are practically ineffective. However, the isolation system shows to be the most convenient solution for such structures with low anchoring resistance. Moreover, the facility being located in Southern Italy, the reference value of the PGA for normal industrial facility (i.e., 10% exceedance in 50 years) is about 0.4 g. Considering this level, implementing the isolation let the probability of failure decrease from 100% to 25%, which can be considered as satisfactory. At the contrary, if assuming that the facility is associated to a hazardous product for which a longer reference return period would have to be considered, leading typically to a reference PGA of 1.0 g, the isolation is showed to reduce the probability of failure from 100% to 90%. In this case, it is clear that another solution should be found to ensure an acceptable safety level of the tank.

Finally, it should be stated that the present study did not took into account the interaction of the gas tank with other infrastructures, as pipelines. For this matter, the compatibility of deformation during the earthquake should be guaranteed by anti-seismic devices at the interface.

Acknowledgement

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