Seismic reliability of continuously supported steel wine storage tanks retrofitted with energy dissipation devices

J.I. Colombo *, J.L. Almazán

Department of Structural and Geotechnical Engineering, Pontificia Universidad Católica de Chile, Vicuña Mackenna 4860, Santiago, Chile

Abstract

Seismic protection of wine storage tanks has become a very important issue due to the booming of the wine industry in seismic countries such as the US, Italy, New Zealand, Chile and Argentina. In order to improve the seismic reliabilities of structures, external energy dissipation may be used beneficially. In this study, the reliabilities of two continuously supported cylindrical steel tanks (one slender and one broad), used for fermentation and wine storage, with and without an external energy dissipation system are calculated by using simulation. A group of non-linear time history analyses is established in order to assess the influence of the energy dissipation system. Each non-linear time history is obtained by means of a mathematical model that considers the fluid–structure–soil interaction. A set of different seismic ground motions is used for the purpose of obtaining robust results in the reliability analysis. Finally, the seismic reliability analysis shows that, for steel wine storage tanks, an external energy dissipation system would reduce the limit state probability in the order of 80%.

Keywords:
Passive energy dissipation
Seismic reliability analysis
Wine storage tanks
Frailty curves

1. Introduction

Continuously supported cylindrical tanks are used in many different civil engineering applications and industrial facilities. Some of these applications are the storage of liquids such as water, wine, oil, nitrogen, high-pressure gas and petroleum. However, almost every major earthquake around the world has affected many of these tanks. For instance, several reports of damage provide evidence of failure and extensive damage in liquid storage tanks such as during the 1960 earthquake in Chile [1], the 1964 earthquake in Alaska [2], the 1977 earthquake in San Juan, Argentina [3], the 1979 Imperial County earthquake [4], the 1980 Livermore earthquake [5], the 1983 Coalinga earthquake [6], and the 1994 Northridge earthquake (all in California) [7], the 2001 Bhuj earthquake in India [8], and the 2010 earthquake again in Chile [9]. Therefore, the poor seismic reliability of these structures is evident.

Substantial economic losses and environmental hazards have been caused by the loss of contents of these tanks [10,11]. The most common types of damage observed in continuously supported liquid storage tanks are: damage to the piping connections caused by large base uplifts, damage to the roof caused by the sloshing of the free liquid surface, buckling of the tank walls caused by the high compressive stress, failure of the anchorage system caused by the high overturning moment transmitted to the base, penetration of the tank wall with anchor bolts caused by the previous failure of the anchorage system and damage to the shell-base connection caused by the plastic rotation of the base plate. However, the failures that are responsible for a large or total loss of the liquids contained in these storage tanks are rupture of the shell-base connection and the penetration of the tank wall with anchor bolts (see Fig. 1).

Continuously supported cylindrical tanks are typically either partially anchored or unanchored at their base. During strong ground motion partially anchored tanks develop a large overturning moment, caused by the hydrodynamic wall pressures, and place high demands on their anchorage system and base foundation. As a consequence of these high demands, anchor bolts may slip from their connections and allow the partial uplift of the base. Subsequently, when the uplifted portion of the base descends, the tank wall may be penetrated or torn by the anchor bolts that have slipped previously.

Similarly, during strong ground motion the unanchored tanks may experience partial uplift of the base due to the overturning moment caused by the hydrodynamic wall pressures. The partial uplift of the base can generate large inelastic rotation demands and possible rupture at the shell-base connection through a low-cycle fatigue failure (see Fig. 2) [12,13]. The failure of this connection can result in the total loss of the liquid contained. In order to avoid this, codes of standard practice such as the Eurocode and...
New Zealand’s Recommendations limit the value of the plastic rotation at the connection to 0.2 radians [14,15]. Koller and Malhotra [16] observed that the limit of 0.2 radians for the plastic rotation was the most significant parameter in the failure of eight different tanks studied. However, Cortes et al. [17], and Prinz and Nussbaum [18] showed that 0.2 radians was excessively conservative and that a limit of 0.4 radians could be justified and was more realistic.

Because of the booming wine industry in some seismic countries such as the US, Italy, New Zealand, Chile and Argentina among others, seismic protection of continuously supported steel wine storage tanks in the face of earthquake hazards is of paramount economic importance. However, as stainless steel wine tanks were not in use when the 1985 earthquake occurred in Chile [10], local evidence of seismic behaviour of these kinds of structures is limited to the recent earthquake in central Chile in 2010. Consequently, there is little information available on seismic hazards in metallic wine storage tanks. It is important to note that at present steel tanks represent 80% of the country's wine storage capacity [11].

Recently numerous studies have been carried out in this field in order to improve seismic behaviour and to reduce the risk of damage or failure [19,20]. In these studies two major alternatives are presented: seismic isolation and external energy dissipation. Some examples of seismic protection in liquid storage tanks using isolation systems are given by Shrimali and Jangid [21], Cho et al. [22], Panchal and Jangid [23,24], Abali and Uçkan [25], Shekari et al. [26], Zhang et al. [27], Soni et al. [28] and Almazán [10]. Similarly, examples of seismic protection in liquid storage tanks using external energy dissipation devices are published by Maleki and Ziyaefar [29,30], Pirner and Urushadze [31], Liu and Lin [32], Malhotra [33] and Curadelli [34].

Furthermore, due to uncertainties related to structural performance and, predominantly, to excitation, probabilistic seismic risk analysis is one of the best tools for measuring the seismic performance of a structural system [34–40]. Therefore, probabilistic seismic risk analysis has received increasing attention in the last two decades; however, previous work on probabilistic seismic risk analysis for liquid storage tanks is scarce. Only some recent investigations have presented a probabilistic seismic risk analysis for a few kinds of storage tanks; for instance, such a risk analysis was published by Curadelli in order to assess the effectiveness of a specific retrofit on spherical storage tanks [34]. It would appear earlier seismic reliability analyses for measuring the effect of any

**Nomenclature**

- $a$: height of the gusset plate in the tank-damper connection
- $b$: width of the U-shaped damper
- $C$: random variable that represents the limit state of the structure
- $c_i$: fixed-base damping factor
- $F_b$: maximum value of the axial force of the anchor bolts during a seismic event
- $F_{lim}$: value of the axial force that provoked the failure at the anchor bolts
- $H$: seismic hazard curve distribution
- $h$: height of the tank-damper connection plate
- $h_{ri}$: height of the resultant of the hydrodynamic wall pressures due to the impulsive component
- $k$: shape parameter of the hazard curve distribution
- $k_i$: fixed-base impulsive stiffness
- $l$: height of the U-shaped damper
- $m_i$: impulsive mass
- $M_d$: overturning moment resisted by the steel dampers
- $M_{rb}$: overturning moment resisted by the liquid-loaded base plate
- $PC$: conditional failure probability distribution (probability of event $C=1$ given a peak ground acceleration of $x$)
- $P_{fi}$: limit state probability for an $N$-year period
- $Q$: ground motion intensity level
- $R$: radius of the tank
- $R_b$: radius of the U-shaped damper
- $S$: width of the gusset plate in the tank-damper connection
- $T$: thickness of the U-shaped damper
- $U$: scale parameter of the hazard curve distribution
- $u_{ri}$: overall horizontal displacement of the impulsive mass relative to the moving base
- $w$: width of the tank-damper connection plate
- $\omega_i$: natural frequency of the fixed-base impulsive component
- $x$: specific ground motion intensity in terms of PGA
- $\bar{x}_S$: horizontal ground motion acceleration
- $\beta$: logarithmic standard deviation of the PGA for the limit state $C=1$
- $\phi$: standard normal cumulative distribution function
- $\mu$: median fragility (the 50th percentile of the fragility)
- $\eta$: maximum value of the plastic rotation during a seismic event
- $\eta_{lim}$: value of the plastic rotation that provoked failure at the shell-base connection
- $\psi$: rotation of the tank base
- $\zeta$: damping ratio
Seismic improvement in continuously supported cylindrical tanks have not been reported in the literature, i.e. only deterministic approaches have been shown in previous works. Consequently, with the premise that the most appropriate approach for measuring the effect of energy dissipation systems in structures under seismic excitation is a seismic risk analysis or reliability analysis [41], in this work the seismic reliability of two typical continuously supported steel wine storage tanks – one slender and one broad – with and without energy dissipation devices will be evaluated numerically. More precisely, with the purpose of evaluating the effectiveness of using energy dissipation devices in this structure, the probability of reaching the limit state of two typical continuously supported steel wine storage tanks with and without energy dissipation devices will be calculated and compared. The seismic behaviour of each structure is examined by performing a set of highly non-linear dynamic analyses based on a mathematical model that takes the material and geometrical non-linearity, and the fluid–structure-soil interaction into account [42–44]. The external dissipation system is modelled using the properties of the metallic damper described in a later section. Several artificial ground motions are considered in the simulation study in order to obtain robust results. The artificial ground motions are based on ground motion predictions from earthquakes recorded in Chile. The structures with energy dissipation devices showed a significant increase in structural reliability, measured by means of the reduction of the limit state probability.

2. Wine-tanks considered

The most important failures in continuously supported tanks, loss of the liquid contained, are the low-cycle failure at the shell–base connection and the penetration or tear of the tank wall with the anchor bolts [11–18]. Therefore, this investigation was focused on the use of energy dissipation anchors to avoid such failures.

Two typical steel tanks – one slender and one broad – were analysed. The slender tank’s characteristics were: capacity 30,000 l, radius $R = 1.46$ m, height of the liquid contained $H = 4.5$ m, and thickness of the wall and base plate 2 mm. Similarly, the broad tank’s characteristics were: capacity 94,000 l, radius $R = 2.40$ m, height of the liquid contained $H = 5.2$ m, and thickness of the wall and base plate 2 mm. The material properties for both tanks were the same. The Young modulus of elasticity and the yielding stress of the tank material were 200 GPa and 248 MPa, respectively. The Poisson ratio was 0.3. The liquid content was wine with a density of 1000 kg/m$^3$. The bases of the tanks were resting on a rigid surface. When the energy dissipation anchors were installed, the tank walls were anchored to a surrounding ring foundation by means of a series of U-shaped strip dampers (Fig. 3). The foundation where the tanks rested was excited by a unidirectional horizontal ground motion $x_g(t)$. When the tanks were subjected to strong shaking, the flexible base plate partially lifted up, which induced a rocking of its wall.

Four different anchorage system options were considered for each tank: (a) unanchored, (b) anchored with 10 bolts with a maximum allowed axial force of 40 kN, (c) anchored with 10 bolts with a maximum allowed axial force of 65 kN, and (d) anchored with 10 U-Shaped metallic dampers. The U-shaped steel dampers dissipated energy when a parallel relative movement between the adjacent surfaces occurred (Fig. 4), in this case a vertical movement of the tank wall. Plastic deformation occurred when the straight part of the strip changed to a curved one and the curved part of the strip became a straight one [45].
3. Wine-tank model

In order to establish the main dynamic behaviour of the wine tanks a simplified mathematical model was used [42–44], which is shown in Fig. 5. The hydrodynamic pressures and forces in the tank can be expressed as the sum of two components. The first component is impulsive, representing the effect of the part of the liquid that moves in synchronism with the tank wall as a rigid body. The second component is convective, representing the effect of the part of the liquid that presents a sloshing motion. However, as the wine storage tanks were completely filled, sloshing was not possible. Therefore, in this model only the impulsive component is considered. The values of the impulsive mode for the slender tank were: $m_i = 18,600 \text{ kg}$, $h_i = 2.04 \text{ m}$, $f_i = \omega_i/2\pi = 11.26 \text{ Hz}$, $\zeta_i = 2\%$; similarly, the values of this mode for the broad tank came to: $m_i = 45,100 \text{ kg}$, $h_i = 1.88 \text{ m}$, $f_i = \omega_i/2\pi = 15.67 \text{ Hz}$, $\zeta_i = 2\%$, where $m_i$ is the impulsive mass, $\omega_i$ is the natural frequency of the fixed-based impulsive component, $\zeta_i$ is its damping ratio, and $h_i$ is the height of the resultant of the hydrodynamic wall pressures due to the impulsive component. Modal parameters were obtained from results published by Veletsos et al. [46] and Veletsos and Tang [47].

The rocking resistance of the liquid-loaded base plate is represented by the rotational base spring. The rotational base spring (i.e. the relationship between the base moment $M_t$ and the spring rotation $\psi$) was obtained by the method of analysing the partially uplifted base plate. In this method, several non-linearities are considered, which are due to (1) the membrane action, (2) the plastic yielding, (3) the varying contact with the foundation, and (4) the varying hydrodynamic base pressures [42–44]. U-shaped strip dampers (arranged in a circular pattern at the base) present a non-linear force–displacement relationship. In addition, it should be noted that the connection between the tank and the damper (in Fig. 5) is represented by the dots located above the dampers, and connect the U-shaped strip damper with the tank base. This tank-damper connection is explained in more detail in the following section.

4. Tank-damper connection model

In order to confirm that the connection of the tank with the dissipation device remained without failure, a simplified model for stress verification was used. This verification was developed in ANSYS with the aid of non-linear static pushover analysis. The stress–strain relation was determined by means of a bilinear isotropic hardening model, where the material parameters of the stress–strain relation were the same as those indicated in Section 2 and the tangent modulus was 1.8 GPa. The connection was discretized using a 3-D 20-node solid element exhibiting a quadratic displacement behaviour (SOLID186). This element has three degrees of freedom per node (i.e. translation in the $x$, $y$ and $z$ directions of the nodal) and makes it possible to perform a non-linear analysis. Large displacement and deformation effects, such as large deflection, large rotation and large strain, were accounted for by using the non-linear geometry option in ANSYS.

Vertical rollers located above the shell-base connection were used to simulate the surrounding tank structure. These rollers kept
the tank shell vertical during uplift and during the application of hydrostatic pressure. The rigid foundation was simulated with compression-only springs, which provided no resistance during uplift. The non-linear damper force was placed in the hole of the tank-damper connection (Fig. 6). The scheme for the structure simulated in ANSYS is shown in Fig. 7.

5. External energy dissipation system

One of the most efficient mechanisms available for dissipating energy in vibrating structures is the plastic deformation of metallic elements with large hysteretic cycles. The first applications of metallic dampers to dissipate part of the seismic energy in structures are attributed to Kelly et al. [48] and Skinner et al. [49]. For this investigation, the external energy dissipation system consisted of 10 metallic dampers arranged in a circular form at the base. Due to the simplicity of the construction and the enormous plastic deformation capacity reported in past earthquakes [50], U-shaped strip dampers were used. The dimensions of the U-shaped strip dampers were \( b = 10 \text{ cm}, \; l = 20 \text{ cm}, \; r = 3 \text{ cm}, \; \text{and} \; t = 1 \text{ cm} \). These dampers were used to anchor the tank wall to the foundation. Each damper had the bilinear approximation for the force–displacement relationship shown in Fig. 8. This relationship was obtained by the method described by Skinner et al. [45], and also reported by Skinner et al. [51]. This method takes into account the dimensions and material of the U-shaped strip damper in order to establish the corresponding bilinear approximation for the force–displacement relationship. The dimensions of the plate connection of each damper with the tank were \( w = 10 \text{ cm}, \; h = 15 \text{ cm}, \; \text{and} \; 0.4 \text{ cm} \). The dimensions of the gusset plate were \( a = 9 \text{ cm}, \; q = 5 \text{ cm}, \; s = 5 \text{ cm}, \; \text{and} \; 1 \text{ cm} \). The diameter of the hole in the gusset plate measured 2.6 cm (Fig. 4). In this context, it is worth mentioning that these types of dampers are widely used in Japan for energy dissipation of base-isolated structures [52,53].

5.1. Stress verification of tank-damper connection

Non-linear static pushover analysis for the tank-damper connection was carried out (displacement control analysis) using the tank-damper connection model described in Section 4. It is noteworthy that similar analyses have been presented in previous works for the shell-base connection of tanks [13,17,18]. However, in this study the shell-base connection was analysed in conjunction with the tank-damper connection and the damper force. Vertical displacement history was applied to the shell following a ramp-shaped function, i.e. linear and monotonic increasing. The displacement step was variable from a minimum of 0.5 to a maximum of 2 mm. The maximum vertical displacement applied was 5 cm. This value was the maximum displacement, at which the plastic rotation failure occurs, observed in the non-linear dynamic analysis performed in a later section. It should be note that the main purpose of the pushover analysis was to check the stresses in the tank-damper connection at the maximum displacement demand at which the plastic rotation failure occurs.

The Von Mises stress distribution of the structure subjected to vertical displacement was calculated for each displacement step. The maximum Von Mises stress for each displacement step is shown in Fig. 9. The maximum Von Mises stress values in the shell tank and the tank-damper connection were 227 MPa and 462 MPa, respectively. In both cases, the structure is shown in the deformed configuration (Fig. 10). As expected, the stresses were highly...
concentrated at the weld toe, where the maximum effective stress reached 462 MPa. Hence, comparing these maximum stress values with the ultimate tensile strengths reported by González et al. [11] for stainless steel used in their tanks (700 MPa), it can be concluded that the tank-damper connection will remain without failure.

6. Solution method for the non-linear time history analysis

The dynamic equilibrium forces on the mass $m_i$ in Fig. 5 implies that

$$m_i \dot{u}_i + c_i(\dot{u}_i - h_1 \dot{\psi}) + k_i(u_i - h_1 \psi) = -m_i \ddot{x}_g(t)$$

(1)

and the equilibrium of base moments implies that

$$-m_i(\ddot{x}_g + \ddot{u}_i)h_1 = M_T(\psi) + M_D(\psi)$$

(2)

where $u_i$ is the overall horizontal displacement of the mass relative to the moving base (Fig. 5), $\psi$ is the rotation of the tank base, an overdot denotes differentiation with respect to time, $\ddot{x}_g(t)$ is the ground acceleration, $k_i = \omega_i^2 m_i$ is the fixed-base impulsive stiffness, $c_i = 2\zeta \omega_i m_i$ is the fixed-base damping factor, $M_T(\psi)$ is the overturning moment resisted by the liquid-loaded base plate, and $M_D(\psi)$ is the moment resisted by the steel dampers. Eqs. (1) and (2) are solved incrementally by means of the linear acceleration method [54].

7. Structural reliability

The procedure for evaluating the effectiveness of the energy dissipation system through a reliability analysis of continuously supported tanks is essentially as follows: (1) a set of seismic ground motions is obtained and normalized by the peak ground acceleration (PGA), (2) fragility curves are constructed by simulation, counting the relative number of times that the limit state of the structure is achieved for each PGA level, and (3) the limit state probability is estimated with the aid of fragility curves in conjunction with the seismic hazard at the structure site. A schematic diagram of this procedure is shown in Fig. 11. The limit state probability for an N-year period in seismic risk assessment is estimated using the expression

- **Failure criteria**

  $(\theta_{\text{lim}}$ and $F_{\text{lim}})$

- **Seismic hazard**

  (Eq. (7) and Fig. 14)

- **Estimated of the limit state probability**

  Calculated with Eq. (9) and the values of $k$ and $\beta$ obtained from the fragility curve and the seismic hazard model

Fig. 9. Maximum Von Mises stress of the tank-damper connection for each displacement step.

Fig. 10. Von Mises stress distribution (in MPa) of the model subjected to vertical displacement applied to the shell at maximum uplifting for (a) the shell-base tank and (b) the tank-damper connection.
where $C$ is the random variable that represents the limit state of the structure and $Q$ is related to the ground motion intensity level, $P[C = 1|Q = x]$ is the conditional probability of reaching the limit state given the occurrence of a seismic ground motion with a specific value of intensity $x$, and $P[Q = x]$ is the probability that this seismic ground motion intensity exceeds the specific level $x$ during a given time period.

This procedure was carried out on the tanks and analysed for each of the four different anchorage systems mentioned in Section 2 and the results were compared in order to assess the effectiveness of the energy dissipation system.

7.1. Seismic ground excitation

A fundamental aspect of developing a reliability analysis is that a set of ground motions is required (step (1) of the procedure above), for which artificial acceleration time histories or actual earthquake records may be used [35]. In the present investigation, a set of forty-two artificial seismic ground motions in accordance with the Chilean code spectrum was assumed [55]. More precisely, an elastic design spectrum for 5% damping and a soil classified as type II of the Chilean code was considered. Near-fault effects were not taken into account because the fault rupture process at the structure site was not related to a near-source fault. Each record was normalized to fifteen specified levels of peak ground accelerations. The total set of forty-two artificial records, scaled to the fifteen levels of PGA, was the seismic ground excitation ensemble considered for the probabilistic analysis. It is worth noting that different scaling procedures are possible [56], such as peak ground velocity or response-spectrum intensity among others but it is not expected that the differences in seismic reliability will be affected by this choice.

7.2. The failure criterion and fragility model

The fragility curve is defined as the probability of reaching or exceeding the limit state for a particular value of the ground motion intensity. Consequently, to build the fragility curves (step (2) of the procedure above) a failure criterion for the analysed structure is required. Although numerous failure criteria have been proposed in the technical literature [57–61], for the current investigation, it was decided that the limit state of damage in the tank occurred when the shell-base connection was broken or when any anchor bolt exceeded the maximum allowed axial force. Therefore, the limit state or failure was reached when

(a) in the unanchored tank, the plastic rotation at the connection became equal to or greater than 0.4 radians;

(b) in the anchored tank, the axial force at any anchor bolt became equal to or greater than the maximum allowed axial force (i.e. 40 kN for a concrete anchor foundation of normal compressive strength and 65 kN for a concrete anchor foundation of high compressive strength);

(c) in the retrofitted tank, the plastic rotation at the connection became equal to or greater than 0.4 radians.

It should be noted that, in order to achieve the aim of the present work, it is not necessary to define several failure criteria that rigorously quantify the damage level of the structure under seismic excitation. One damage state (failure or no failure) was considered to be sufficient as the objective of the present work was to compare the performances prior to and after the structural retrofit.

Therefore, it is possible to define a failure random variable $C$ as

$$P_C = \sum_x P[C = 1|Q = x]$$

(3)

where $C$ is the random variable that represents the limit state of the structure and $Q$ is related to the ground motion intensity level, $P[C = 1|Q = x]$ is the conditional probability of reaching the limit state given the occurrence of a seismic ground motion with a specific value of intensity $x$, and $P[Q = x]$ is the probability that this seismic ground motion intensity exceeds the specific level $x$ during a given time period.

After obtaining the set of seismic ground motions, the structural model and the failure criterion were defined, and the structural responses of the tanks for each anchorage system were calculated for each excitation sample by means of non-linear time history analysis. The conditional failure probability distribution was assessed counting the relative number of times the response reached the limit value for the plastic rotation (for the cases defined in (a) and (c)) or the limit value for the axial force (for the case defined in (b)), which can be expressed as

$$P_C = P[C = 1|Q = x]$$

(5)

where $C$ is the random variable that represents the limit state of the structure and $Q$ is related to the ground motion intensity level, expressed in terms of the peak ground acceleration. Therefore, $P_C$ is the probability of event $C = 1$ given a peak ground acceleration of $x$.

The fragility curves related to seismic analysis, and as functions of the peak ground acceleration, have a lognormal functional form given by (see e.g. [62–65])

$$P_C(x) = \phi \left[ \frac{1}{\beta} \ln \left( \frac{x}{\mu} \right) \right]$$

(6)

where $\phi[]$ is the standard normal cumulative distribution function, $\mu$ is the median value of peak ground acceleration for which the structure reaches the 50th percentile of fragility, and $\beta$ is the lognormal standard deviation of the PGA for the limit state $C = 1$. These parameters are determined by fitting a lognormal function to the conditional failure probability distribution obtained from Eq. (5).

7.3. The seismic hazard model

In Eq. (3), the term $P[Q = x]$ represents the distribution of possible ground motion intensity levels determined as the derivative of the seismic hazard curve $H(x)$. This derivative of the seismic hazard is usually modelled using a complementary cumulative distribution function obtained from a seismic hazard analysis (SHA) of the site over a period of time. The hazard curve, in recent seismic risk analyses, has been described by a Type II distribution of largest values as (see e.g. [36])

$$H(x) = 1 - \exp \left[ -\left( \frac{x}{\eta} \right)^k \right]$$

(7)

where $\eta$ and $k$ are the scale and shape parameters of the distribution, respectively. More details on probabilistic seismic hazard analysis can be found in Lee et al. [66], Field [67], SSHAC [68] and McGuire [69].

7.4. Estimation of the limit state probability

The limit state probability, Eq. (3), can be estimated by convolving the fragility $P_C(x)$ with the derivative of the seismic hazard curve $H(x)$ as follows

$$P_s = \int_0^\infty P_C(x) \frac{dH(x)}{dx} dx$$

(8)

Assuming that only a relatively narrow range of $x$ values in the integrand of Eq. (8) contributes significantly to $P_s$, the limit state probability can be approximated by (see e.g. [65,70])
Therefore, the limit state probability was estimated with the seismic hazard $H(x)$ and evaluated at the median fragility $\mu$, and multiplied by a correction factor that takes into account the parameters related to the uncertainties associated with the ground motion intensity (measured by $k$) and the structural capacity (measured by $b$). It should be mentioned that the latter two parameters, $k$ and $b$, were obtained by means of evaluation of the seismic hazard model and the fragility model, respectively. All the structural reliability calculations were carried out with MATLAB routines developed by the authors. Typical values of $k$ are in the range of approximately 1.5–2.5 in regions of moderate seismicity and 3–4 in highly seismic zones [71]. Similarly, the values of $b$ vary between 0.15 and 0.25 depending on the structural performance level [72].

8. Discussion of results

The seismic fragility relations and the limit state probabilities for both storage tanks (slender and broad) with the four different anchorage systems were estimated using the procedure described in Section 7. The fragility points obtained from the simulation (Eq. (5)) were fitted with a lognormal function with 95% confidence for each anchorage type in both tanks (Fig. 12).

A considerable increase in the capacity against failure of the structure was observed with the external energy dissipation system (Fig. 12). For instance, in order for the slender tank (Fig. 12(a)) with the anchorage with bolts of $F_{\text{lim}} = 40 \text{kN}$ to reach the fifty percent probability of failure, i.e. the median fragility, a peak ground acceleration of 1.62 m/s² was necessary. Similarly, in order to reach the median fragility for the anchorage condition with bolts $F_{\text{lim}} = 65 \text{kN}$ a PGA of 2.35 m/s² was required. At the same time, in the unanchored tank and in the one with the energy dissipation devices the PGA at the median fragility amounted to 2.48 m/s² and 5.12 m/s², respectively.

The broad tank presented a similar behaviour (Fig. 12(b)), where the PGA at the median fragility was as follows: 1.76 m/s² for the anchorage condition with bolts of $F_{\text{lim}} = 45 \text{kN}$, 2.15 m/s² for the anchorage condition with bolts of $F_{\text{lim}} = 60 \text{kN}$, 3.03 m/s² for the unanchored condition, and 5.98 m/s² for the condition with dampers installed. As can be seen, the rise in the PGA that was required to reach the median fragility by using external energy dissipation devices represented an increase greater than 106% for the slender tank and greater than 97% for the broad tank (Tables 1 and 2).

The increase in the capacity against the failure of the tanks with the external energy dissipation system was achieved due to 1) the increase in the energy dissipated as a result of the hysteretic behaviour of the dampers and 2) the forces of the dampers, which were opposing the uplifting at the base; this in turn reduced the maximum uplifting at the tank base and thereby reduced the maximum plastic rotation at the shell-base connection. For instance, in order to compare the unanchored tanks and the tanks with dampers subjected to an earthquake with a mean recurrence interval of 0.12

![Fig. 12. Seismic fragility relations for the structure with and without external dissipation devices: (a) the slender tank and (b) the broad tank.](image-url)
475 years (approximately with a PGA of 3.7 m/s²) in the region the effective damping was estimated from the time history results of one ground motion record with the above PGA of the seismic ground motion ensemble. The effective damping was computed from the size of the biggest loop [73]. The dissipated energy due to the hysteresis of the dampers was reflected in an increment of the effective damping, which rose from 5% to 15% for the slender tank, and from 6% to 10% for the broad tank, when the dampers were installed. Similarly, the effect of the damper forces was reflected in an increment in the total resisted overturning moment, which was two or three times higher at the same base rotation when compared with the tank without dampers (Fig. 13).

The seismic hazard curve \( H(x) \), over a period of 100 years and obtained from the SHA based on the historical and instrumental data of the region, is presented in Fig. 14. On a 100-year basis, the seismic hazard of the slender tank for each anchorage configuration, evaluated at the median fragilities (1.62 m/s², 2.35 m/s², 2.48 m/s² and 5.12 m/s², respectively), came to the values 0.8663, 0.5292, 0.4792 and 0.0932, respectively. Multiplying these values by the exponential factor (see Eq. (9)) the limit state probabilities amounted to 0.9171, 0.5817, 0.5932 and 0.0932, respectively (Table 1).

Similarly, the seismic hazard of the broad tank for each anchorage configuration, evaluated at the median fragilities (1.76 m/s², 2.15 m/s², 3.03 m/s² and 5.98 m/s², respectively), resulted in the values 0.8507, 0.6763, 0.6763 and 0.0636 (Table 2). Thus, the probability of reaching the limit state of the structure was reduced significantly in both tanks, i.e. by at least 81%, using external metallic dampers.

The above results can also be interpreted as follows: An earthquake with a mean recurrence interval of 970 years (10% of being exceeded in 100 years) might be described approximately by a PGA of 4.7 m/s² in the region (Fig. 14). At this ground motion intensity and for tank configurations in which the dampers are not

### Table 1
Limit state probability and PGA at median fragility of the slender tank for the four anchorage options: Unanchored, Anchored with bolts of \( F_{\text{lim}} \) equal to 40 kN and 65 kN, and anchored with dampers (over a 100-year period).

<table>
<thead>
<tr>
<th>Anchorage Configuration</th>
<th>Limit state probability</th>
<th>PGA (m/s²) at median fragility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unanchored</td>
<td>0.4985</td>
<td>2.48</td>
</tr>
<tr>
<td>Anchored with bolts (( F_{\text{lim}} = 40 ) kN)</td>
<td>0.9171</td>
<td>1.62</td>
</tr>
<tr>
<td>Anchored with bolts (( F_{\text{lim}} = 65 ) kN)</td>
<td>0.5817</td>
<td>2.35</td>
</tr>
<tr>
<td>Anchored with dampers</td>
<td>0.0932</td>
<td>5.1</td>
</tr>
</tbody>
</table>

### Table 2
Limit state probability and PGA at median fragility of the broad tank for the four anchorage options: Unanchored, Anchored with bolts of \( F_{\text{lim}} \) equal to 40 kN and 65 kN, and anchored with dampers (over a 100-year period).

<table>
<thead>
<tr>
<th>Anchorage Configuration</th>
<th>Limit state probability</th>
<th>PGA (m/s²) at median fragility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unanchored</td>
<td>0.3314</td>
<td>3.03</td>
</tr>
<tr>
<td>Anchored with bolts (( F_{\text{lim}} = 40 ) kN)</td>
<td>0.8507</td>
<td>1.76</td>
</tr>
<tr>
<td>Anchored with bolts (( F_{\text{lim}} = 65 ) kN)</td>
<td>0.6763</td>
<td>2.15</td>
</tr>
<tr>
<td>Anchored with dampers</td>
<td>0.0636</td>
<td>5.98</td>
</tr>
</tbody>
</table>
installed, both the slender and broad tanks have a 100% probability of reaching their limit states (Fig. 12) whereas at the same ground motion intensity, the retrofitted tanks with the dampers have approximately a 10% and 2% probability of reaching their limit states in the case of the slender and the broad tank, respectively. As explained above, this significant reduction in the failure probability is a direct consequence of the increment in the amount of the dissipated energy (the increment in effective damping) and the strengthening of the uplifting base behaviour (the increment in the resisted overturning moment).

It should be noted that the measuring of the effectiveness of the dissipation devices installed on continuously supported tanks presented here is different from the deterministic approaches shown in previous works, in which acceleration or displacement reductions are evaluated. The results presented in this investigation were obtained by a probabilistic method that offers a more reliable measure of the expected seismic performance because it considers the uncertainties related to structural behaviour and excitation. Finally, the robustness of the results presented in this study is one of the main advantages as the results are expressed in terms of risk reduction and not in terms of a particular acceleration or of displacement reductions.

9. Conclusions

The effectiveness of a particular energy dissipation system in two typical wine storage tanks (one slender and one broad) was assessed by means of the seismic reliability concept. The seismic reliability analysis of the storage tanks with and without the energy dissipation system was developed by simulation. For the tanks without the energy dissipation system, three different anchorage options were evaluated: (1) unanchored, (2) anchored with bolts that had a maximum allowed axial force of 40 kN, and (3) anchored with bolts that had a maximum allowed axial force of 65 kN. The seismic reliability analysis showed that the energy dissipation devices were very effective in reducing the probability of reaching the limit state of the structures by at least 81% in both the slender and the broad tank.

The following conclusions are based on the results presented in this work:

- For continuously supported wine storage tanks, anchorage with steel dampers can significantly reduce the probability of reaching the limit state, i.e. by a reduction of at least 81%.
- For slender and broad tanks, the lowest seismic reliability condition is the tank anchored with bolts that have a maximum allowed axial force of 45 kN.
- For slender tanks, there is no significant difference between the seismic reliability of tanks anchored with bolts that have a maximum allowed axial force of 60 kN and the unanchored tanks.
- Unanchored broad tanks demonstrate a greater seismic reliability than broad tanks anchored with bolts.
- Using external energy dissipation systems may represent a significant improvement in the seismic reliability of Continuously supported wine storage tanks.

Acknowledgments

The authors wish to acknowledge the financial support provided by CONICYT-PCHA/Doctorado Nacional/2013-63130131 and the FONDECYT project number 1120937. In addition, a particular thank-you is extended to Dr. Praveen Malhotra for his valuable comments on the non-linear dynamic analysis of liquid storage tanks.