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Influence of geometric imperfections on the seismic performance of unanchored liquid storage tanks

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ABSTRACT

This paper investigates the inelastic buckling of imperfect unanchored cylindrical steel tanks subjected to seismic loads. A pushover-based seismic analysis was conducted considering the impulsive hydrodynamics pressures on the wall and base plate of the tank. Material and geometric nonlinearities were considered in the seismic analysis of the tanks. Three types of imperfections were analyzed: imperfections due to impacts, imperfections located at the bottom of the tank wall, and imperfections with the first buckling mode shape. The buckling analysis was performed for two tank geometries (one slender and one broad), and the effect of the imperfections was correspondingly evaluated. The considered imperfection amplitudes were established according to the New Zealand seismic design code for storage tanks. Additionally, amplitudes that exceed the normative limit were evaluated to further analyze the sensitivity to imperfection. The analysis revealed that geometric imperfections reduce the peak ground acceleration needed to induce buckling failure. Specifically, the critical peak ground acceleration for the slender tank decreased from 0.195 g to 0.180 g for the slender tank and from 0.600 g to 0.535 g for the broad tank. This buckling capacity reduction due to geometric imperfections were about 8 % and 11 % for the slender and the broad tanks, respectively.

1. Introduction

Liquid storage stainless steel and carbon steel tanks are predominantly situated in industrial and food facilities. They find applications in various sectors for the storage of oil, water, milk, wine [1], among others. Thus, tank structures are particularly relevant due to its ability to contain and protect large volumes of liquids; however, the seismic failure of the tanks can be harmful to people, and may cause health problems in communities, environmental damage, and economic losses. For instance, during the Maule earthquake in 2010, losses of around 125 million liters of wine were recorded due to tank failures, which meant a significant economic loss [2]. In New Zealand, after the earthquakes of 2013 and 2016, damages were reported in hundreds of wine tanks [3]. In Italy, during the Emilia earthquake in 2012, significant economic losses were also reported due to tank failures and discontinuity of their operation [4]. Such as consequences have motivated research to understand and improve the seismic performance of liquid storage tank.

During seismic events, hydrodynamic pressures are generated inside the tanks, presenting an impulsive and convective component [5]. The impulsive component is caused by the portion of liquid that moves in synchrony with the tank wall and shares the oscillation period with the tank walls. This component is responsible for generating significant failures in this type of structures due to its significant overturning effect [6]. On the other hand, the convective component is caused by the upper portion of fluid moving independently of the tank and exhibits high oscillation periods compared to the impulsive one. Failure of the piping system due to tank base uplifting, deformation of the roof due to liquid sloshing, and buckling in the mantle resulting from axial compression are commonly reported damages in unanchored cylindrical steel tanks [7]. Despite the several failure types caused by the seismic events, there is a consensus in the literature that one of the most common and most dangerous is the buckling failure of the tank wall [8,9]. This failure is characterized by an outward bulge-shaped plastic deformation in the lower part of the tank wall called elephant foot. This plastic deformation is a state prior to the fracture of the material, at which time the loss of fluid may occur. Elephant foot buckling is produced by axial compression of the tank wall in conjunction with hydrodynamic pressures that act perpendicular to the axial compression [8]. The above results show

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Fig. 1. Drawing of cylindrical steel tanks.

the importance of understanding this phenomenon to provide structural solutions.

Several numerical and experimental investigations have been conducted to understand and evaluate the seismic behavior and failure mechanisms of liquid storage tanks. These studies explore various aspects, including fluid modeling strategy [10], shaking table testing [11], the impact of structural analysis methods [12], and geometrical configurations different from cylindrical tanks [13]. Other areas of analysis consider the uplift of the tank base [14], the possibility of multidirectional excitations [15], the influence on the response of excitation frequency and its amplitude [16], low cycle fatigue [17], the interaction fluid-structure [18], among others. In addition, Zeybek and colleagues have assessed the seismic performance of open-top cylindrical with [19, 20] and without [21,22] auxiliar stiffening rings using detailed finite element analysis. Azzuny and Guzey have numerically explored the failure behavior [23,24] and the fatigue evaluation [24] of tanks with semicircular-topped rectangular cleanout. Godoy presents a review of methods for detecting buckling in tanks under static loads, such as uniform pressure, wind loads or foundation settlement [25].

Most of these investigations consider tanks with perfect geometries, i.e., without any initial deformations, but there are multiple factors that can affect the cylindrical geometry of the tank before entering operation [26]. For example, imperfections may occur during transportation, lifting, positioning, and tank settlement. Rastgar and Showkati [27] established a total of eight imperfection types and how they origin from a field study of welded steel tanks for storing oil. From a state of the art, Razzaghi [28] concluded that the seismic performance of ground cylindrical steel tanks is characterized for geometric and material nonlinearities and depends on issues such as random imperfections. The presence of imperfections in a structure, such as prior deformations in tanks, can effectively reduce the PGA necessary to reach the buckling state. This occurs because imperfections could weaken the structural integrity or alter the stress distribution, making the structure more susceptible to buckling under lower seismic loads. Currently, it is possible to use new technologies for a correct characterization of geometrical imperfections in tanks. In that sense, Lopes et al. [29] developed a laser scan-based technology for determining the structural integrity of tanks from deformation measurements. Then, it is necessary to understand how these deformations affect the seismic performance of the tanks.

There are several studies to assess the influence of imperfections and defects on the buckling of tanks, cylindrical shells and silos under non-seismic forces (i.e., wind pressures [30], hydrostatic pressures [31], settlement [32]). But very few and recent research have been conducted to investigate the seismic performance of liquid storage tanks considering geometric imperfections. Chikhi and Djermane [33] studied the dynamic buckling of aboveground steel tanks considering axisymmetric and asymmetrical imperfections produced in the construction phase, concluding that the critical Peak Ground Acceleration (PGAcr) decreased until 9.11 % respect the perfect configuration. It is important to mention that the critical Peak Ground Acceleration (PGAcr) is defined as the Peak Ground Acceleration (PGA) required to induce a specific critical or limit state in a structure. It represents a seismic intensity measure (IM) that triggers a particular limit state. Spritzer et al. [34] showed that out-of-plumbness and out-of-roundness imperfections do not affect considerably the seismic performance of grounded carbon steel tanks if the API-650 tolerances are respected. Hadj-Djelloul and Djermane [35] analyzed the effect of weld imperfections located in the interface region between the support and storage sections of an elevated conical steel, observing that the PGAcr decreased at least in 45 % as compared with the value obtained for the perfect tank. Thus, further studies are needed for a comprehensively understanding of the effect of geometric imperfections on seismic performance of the different types of liquid storage tanks (e.g., slender and broad tanks, carbon steel and stainless-steel tanks, fixed-roof tanks, legged supported tanks, among others).

This research aims to evaluate the incidence of geometric imperfections in the seismic performance of unanchored cylindrical steel tanks using nonlinear 3D finite element models. Two tanks were considered in this study (one stainless steel tank and one carbon steel tank), which were subjected to hydrodynamic pressures by means of a pushover-based seismic analysis. The 3D finite element models were developed using ANSYS [36]. Three types of imperfections were analyzed: randomly distributed imperfections (AI), which can occur throughout the life of the tank; imperfections located at the bottom of the tank (LI), in the area where elephant foot buckling often occurs (simulating an unfavorable scenario); and imperfections according to a deformation pattern similar to the first elastic buckling mode (MI), assuming an imperfection associated with the limit state of buckling. The capacity spectrum method (CSM) was used to evaluate the seismic performance of the considered tanks [37,38]. The capacity of the tank was obtained by using a pushover-based analysis, and the considered seismic demand was estimated through the Chilean standard NCh 433 of 1996 Mod 2009 [39]. This seismic demand is compatible with high seismicity zones such as New Zealand [40], Argentine [41], Italy [42] and California [43]. Further comparison of some seismic design spectra is presented in Ref. [44].

Finally, the main contributions of this research are related to:

- An analysis on the seismic performance for unanchored slender steel tanks with small thickness with imperfections is reported for first time in the literature. Such type of tank is typically used for wine storage in seismic countries such as Chile, Argentine, New Zealand, Italy and USA (California) [2,45–47].
- Three types of geometrical imperfections that could occur on the life of the tanks are considered. Such imperfections complement to those studied in Chikhi and Djermane [33] and Spritzer [34] for unanchored steel tanks.
- The effect of imperfections whose size is both accepted or not for the normative New Zealand design code [48] is approached. This analysis would allow understanding if the threshold defined in the code is adequate.

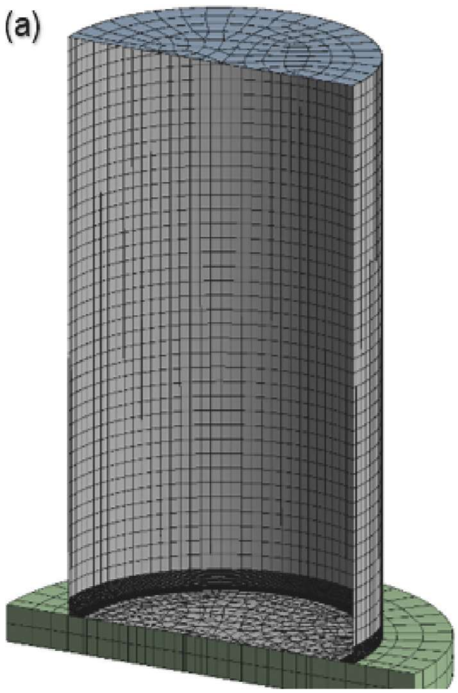

Fig. 2. Finite element model of the cylindrical steel tanks.

2. Methods: Seismic analysis of unanchored cylindrical steel tanks with imperfections

This section describes the process followed in this research to assess the non-linear seismic response of unanchored cylindrical steel tanks with imperfections. The main step of the procedure corresponds to i) develop a finite element (FE) model for the tanks that considers the interaction fluid-structure, ii) define the numerical strategy for including the analyzed imperfections into the FE model, iii) assess the seismic performance of the tanks with and without imperfections trough a pushover analysis and a given seismic hazard [49–51]. Fig. 1 presents a schematic drawing of cylindrical steel tanks on a planar terrain. This is only a schematic representation of a set of slender tanks which are typically located in group over the same foundation [2].

2.1. Finite element modeling

This section presents the FE modeling procedure followed in this study to assess the nonlinear response of unanchored cylindrical steel tanks under seismic loads in ANSYS software [36]. The geometry of an unanchored cylindrical steel tank is very simple, understanding that they are composed by three parts: i) the walls, ii) the base and iii) the top. The steel types used in this study for the tank fabrication correspond to 304L stainless steel and S235 carbon steel. Both materials were modeled using Misses yield criterion in combination with kinematic hardening rule for the post-yield steel material behavior. The wall, bottom and roof of the tank were modeled using SHELL181 elements.

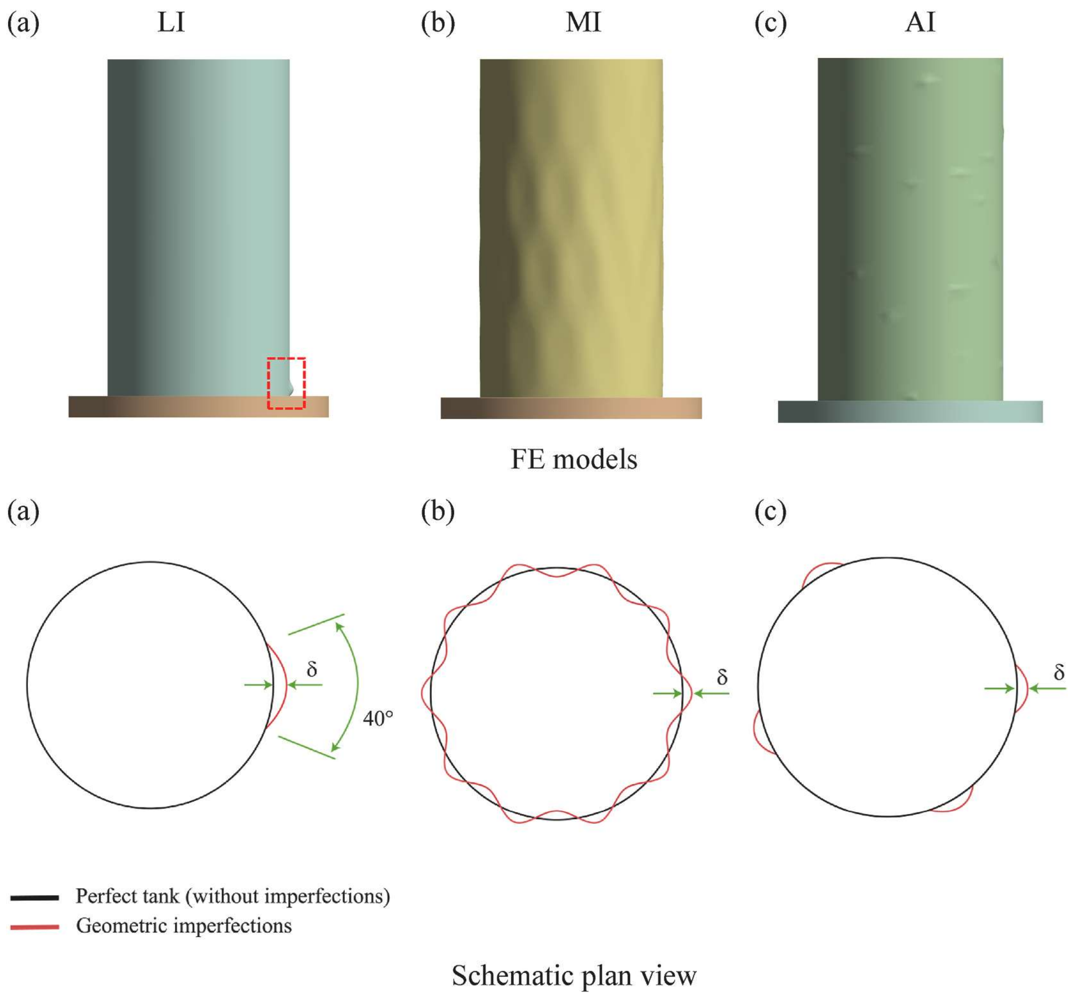

Fig. 3. Types of imperfections for a steel tank: (a) localized, (b) first mode-shaped, and (c) arbitrary distributed.

This element is a four-node element with six degrees of freedom at each node: translations in the *x-y-z* directions, and rotations about the *x-y-z* axes [36]. Five integration points were used for the SHELL181 elements. It is considered that there is a fixed connection between the walls and the base and top plates.

It is considered that the tanks lie on a concrete slab foundation with no anchors. The foundation was modeled with elements type SOLID186, which corresponds to a high-order 3D 20-node solid element that exhibits quadratic displacement behavior. This element has three degrees of freedom per node: translations in the x-y-z directions [36]. For the interaction between the tank base and the foundation, CONTA175 elements were used [36]. This element type considers possible contact, separation and sliding between the tank base and the foundation. A friction coefficient of 0.4 was considered [52]. The foundation slab is completely constrained in the bottom face. A simple refinement mesh was used for the foundation since it does not substantially influence the analysis required for this research.

For all FE models, the mesh size for the base and wall was established by numerical convergence tests, keeping the size as lower as possible to avoid higher computational cost. Fig. 2 shows an example of a constructed 3D nonlinear finite element models for a cylindrical steel tank in Ansys. The application of the hydrostatic loads is presented in Section 2.3. In addition, the definition of the FE models considers several nonlinear mechanisms in the seismic analysis of unanchored tanks, such as: (i) yielding of the base plate and the wall of the tank, (ii) varying contact area between the base plate and the foundation during partial base uplifting, and (iii) second-order effects. The inclusion of the imperfections on the FE models is discussed in Section 2.2.

2.2. Considered geometric imperfections

The considered geometric imperfection refers to the deviation of the tank from a perfect cylinder. In simpler terms, the radius of the tank does not exactly match the nominal radius. Consequently, the tank radius may be greater or smaller in any specific zone. The amplitude of the imperfection, δ, is defined as the maximum value of the difference between the nominal radius and the actual radius at any given zone of the tank shell (see Fig. 3). From the several type of geometrics imperfections that can be occur in a cylindrical steel tank, three of the most common ones were considered in this study:

- Imperfections located in critical zones of the tank (denoted as LI): this case simulate an unfavorable scenario where the imperfection occurs in a critical zone of failure. In this research this of imperfection is assumed to be in the bottom part of the tank walls where elephant foot buckling often occurs (Fig. 3a). In particular, the imperfection was imposed at a height of 20 cm within an approximately 40° arc, corresponding to the compression zone reported in previous studies for slender tanks [14,53]. This location and configuration are considered the most critical and probable imperfections resulting from previous seismic impacts on such tank.
- First mode-shaped imperfections (denoted as MI): this case considers the presence of imperfections that follow the first mode pattern with the aim of assessing the worst condition of the dominant mode (Fig. 3b). This imperfection affects the entire wall tank.
- Arbitrary distributed imperfections (denoted as AI): which can occur throughout the life of the tank in random locations (Fig. 3c). This type of imperfection can be generated during the manufacture, transport and/or installation of the tank, hence there is not a single deformation pattern. These imperfections are defined as encompassing 10 % of the total tank wall area. Specifically, 30 points on the tank wall were selected randomly with a uniform probability distribution. At these points, localized imperfections of specific amplitude were assigned (see Fig. 3c). It is important to clarify that this case serves as an illustrative comparison to the more common imperfection modeling scenarios mentioned above (i.e., LI and MI). While our sample is limited, it is intended to supplement, rather than independently validate, the conclusions drawn from the typical cases. For a statistical analysis covering a broader range of scenarios with random imperfections, the readers are referred to Miladi et al. [54].

Fig. 3 presents the finite element models for the tanks including the three types of imperfections listed above. In Ansys, the inclusion of the different imperfections required a generation of a CAD model developed in FUSION 360 [55]. It is worth mentioning that the imperfections are introduced in the FE models previous to the hydrostatic and seismic loads.

In this research, it will be explored several amplitudes for the imperfections to determine the feasibility of the maximum allowable values given in a structural design code. Eq (1), originally proposed by Rotter [48], was used for the threshold definition:

$$\frac{\delta}{t}=\frac{0.06}{a}\sqrt{\frac{R}{t}} \tag{1}$$

where δ is the maximum amplitude of the imperfection, t is the thickness of the tank wall, R is the radius of the tank, and a is the construction quality, in which $a = 1.0$ is associated with a "normal" construction quality, $a = 1.5$ is associated with a "high" construction quality, and $a = 2.0$ is associated with a "very high" construction quality [48]. The scope of this research considers tanks with a normal construction quality,

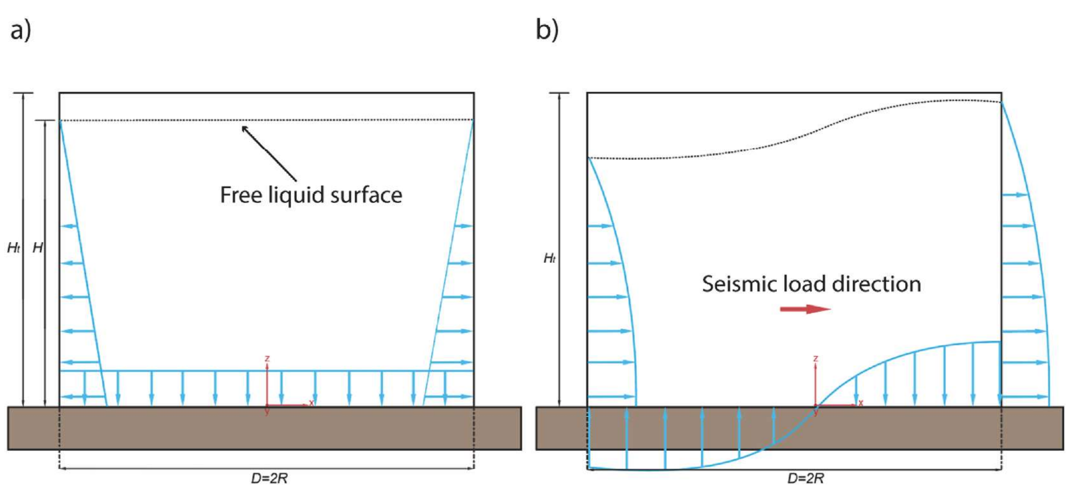

Fig. 4. Schematic representation of the pressure distributions in a tank: (a) hydrostatic pressures and (b) hydrodynamic pressures.

Table 1
Geometric properties of the considered tanks.

Dimensions	Tank A	Tank B
Height, H (mm)	7760	10000
Radius, R (mm)	2100	5000
Base plate thickness, t_b (mm)	2	4.5
Wall thickness, t (mm)	2	9
Capacity (m^3)	100	785

hence $a = 1.0$ is used for the analysis. This equation is also used in the New Zealand [48] and European design codes [56]. It is important to mention that the analyzed imperfections are coherent with possible previous elephant foot buckling, or typical imperfections suggested by the standards and technical literature (first elastic buckling mode).

2.3. Seismic performance of a steel tank from a pushover analysis

Due to the high computational cost associated with the detailed modeling of liquid storage tanks for seismic analyses, several studies assess the seismic performance of these structures by applying a pushover-based seismic analysis with the impulsive hydrodynamic pressure distribution [12,37,49–51,53,57,58]. In particular, the seismic performance of liquid storage tanks has been effectively evaluated using the Capacity Spectrum Method (CSM) [37,38,51,59]. Therefore, a pushover-based seismic analysis was carried out in this research. This methodology allows to evaluate the nonlinear response of the structure subjected to large seismic demands, optimizing the computational costs. In the case of tanks, the fundamental mode of vibration is the impulsive mode of the tank-liquid system.

The structural capacity of each tank was obtained in three steps refereed to the loads type. First, in step 1, gravity was applied to the base and walls of the tanks. Then, in step 2, hydrostatic pressures were applied to the base and walls of the tank to determine the initial state of the tanks. Subsequently, in step 3, the hydrodynamic pressures were applied, which were gradually increased until the critical buckling condition of the tank was obtained. The critical buckling condition is one in which excessive radial deformation induces the initiation of the entire structure instability [12].

Hydrodynamic pressures were modeled using equations (2) and (3), which distribute the pressure at the wall and base of the tank, respectively [60].

$$p_{(z,\phi,t)} = \alpha(z)\rho_l\, R\, A(t)\, \cos\phi \tag{2}$$

$$p_{(r,\phi,t)} = \alpha(r)\rho_l\, R\, A(t)\, \cos\phi \tag{3}$$

where α is a dimensionless function that radially distributes pressure, both in the wall $\alpha(z)$, as in the base $\alpha(r)$, ρ_l is the density of the stored fluid, and $A(t)$ is the impulsive pseudo-acceleration. Fig. 4 shows a scheme of the distribution of induced hydrodynamic pressures in a Tank, for the hydrostatic and the seismic condition.

Finally, the critical buckling condition is determined from the application of the Budiansky-Roth criterion [61], which has been extensively used in the buckling analysis of storage tanks and silos [12, 37,62,63]. As pseudo-acceleration increases, the increase in radial deformation becomes more significant, until reaching a point where radial deformations increase considerably given small increases in pseudo-acceleration. This high increase in the radial deformation represents a critical point from which the tank wall entered in a condition of structural instability, exhibiting the elephant foot buckling pattern [12, 37].

3. Numerical examples

This study analyzes the seismic performance of two different cylindrical tank geometries, whose dimensions correspond to typical values following the application. These two tanks presented damage on the 2010 Maule and the 2015 Illapel earthquakes. These two geometries are the slenderest geometries reported in Jaramillo et al. [64] and Colombo et al. [14]. Slender tanks have been reported as the most vulnerable tank geometries. Table 1 presents the geometric characteristics of the analyzed tanks. Tank A is a commonly used in the wine industry, presenting uniform wall thickness [2]; and Tank B is commonly used for water and oil storage [64]. The dimensions considered correspond to real tanks designed following the NZSEE recommendations [48]. The design procedure involved an iterative process. More precisely, for a given specific volume of liquid to be stored, the tank's radius and height were determined. Subsequently, a tank wall thickness was chosen, and conditions such as allowable values for axial compressive stress and base uplift were verified. For more details about the design process, see Ref. [65]. In this case, it was assumed that the wall thickness is uniform as approached by other previously studies [58,66,67]. Due to the height-to-radius ratio, Tanks A and B were considered as a slender tank and a broad tank, respectively. It is worth mentioning that both tank geometries have been analyzed in previous studies at both anchored and unanchored conditions [14,64]. In addition, to ensure that the elephant foot buckling occurred consistently in the lowest shell course, the tanks were analyzed with a uniform shell thickness [58]. Both tanks were assumed to be at full capacity, a common condition for wine storage and firefighting water storage tanks. Moreover, recent studies have demonstrated that the maximum seismic demand on shell stresses, induced by impulsive hydrodynamic pressures, occurs at the maximum fill height [68]. Finally, the conclusions obtained in this research are referred to the type of tank analyzed, requiring a further quantity of examples to approach all the possibilities of tanks.

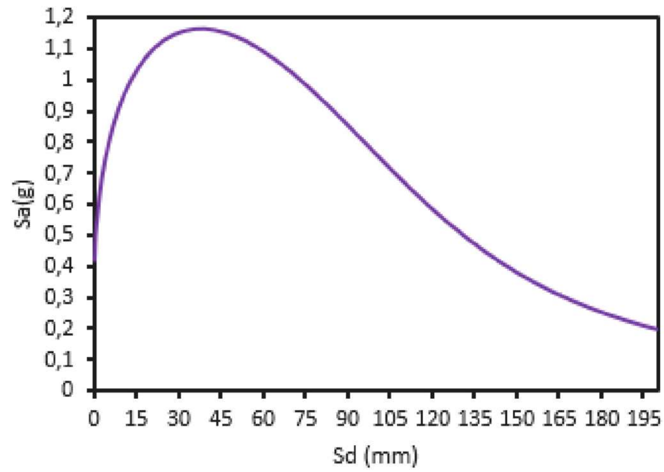

Fig. 5. Design spectrum according to NCh433 [28] and DS61 [46].

The mechanical properties of the materials used for the tanks are defined as follows: Tank A is made of 304L stainless steel with Young's modulus of 193 GPa, yield stress of 210 MPa, tangent modulus of 1.8 GPa, Poisson's ratio of 0.31 and a density of 7750 kg/m^3. Tank B is made of S235 carbon steel with a Young's modulus of 210 GPa, yield stress of 235 MPa, tangent modulus of 1.8 GPa, Poisson's ratio of 0.3 and a density of 7850 kg/m^3 [1]. The liquid stored in both tanks is water, with a density of 1000 kg/m^3. A foundation slab 50 cm thick composed of concrete was considered for Tank A, and 100 cm thick for Tank B. The concrete material properties were Young's modulus of 30 GPa, Poisson's ratio of 0.18, and a density of 2300 kg/m^3.

The considered demand curve corresponds to the design spectrum provided by Supreme Decrete 61 based on NCh 433 Of 1996 "Seismic Design of Buildings" [39,69]. A seismic zone II, soil type C, and equivalent damping of 5 % were considered. This damping value was considered constant to represent a conservative condition since it is the lowest value reported for energy dissipation of storage tanks through the