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SEISMIC DAMAGE CRITERIA FOR A STEEL LIQUID STORAGE TANK SHELL AND ITS INTERACTION WITH DEMANDED CONSTRUCTION MATERIAL

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ABSTRACT

In this paper, the relation between the steel cylindrical tank geometry and the governing critical damage mode of the tank shell is numerically determined for all practical ranges of liquid storage tanks (aspect ratio *H/D* = 0.2 to 2). In addition, the interaction between the seismic intensity, soil type, acceptable seismic risk and tank geometry along with the extra material demanded by the seismic loads is examined based on the provisions of major codes. The importance of seismic factors on the economics of the design of a liquid tank in zones with high seismic activity is comprehensively discussed. In this regard, an empirical relation to estimate the steel volume required for specific seismic conditions and tank geometries is proposed based on the results of analysis.

Keywords: *Liquid storage tank, Failure mode, Shell buckling, Material consumption*

INTRODUCTION

Thin-walled cylindrical shells for liquid storage tanks having a uniform or stepwise variable thickness are employed in many engineering fields. Vertical aboveground cylindrical steel storage tanks are vital components of lifelines and industrial facilities and are usually constructed with a thin bottom plate, a wall plate and a closed roof or open top. They are used to store water, oil, fuel, petrochemical products and other fluids.

Dynamic loads caused by an earthquake can damage these storage tanks and cause dysfunction, environmental pollution and fire. Damage observed in such tanks in the past earthquakes include overturning, shell buckling, roof damage, anchor bolt and nozzle failure, sliding and uplifting [1]. In most previous earthquakes, shell buckling occurred in atmospheric vertical steel tanks. The seismic design of thin cylindrical tanks to prevent shell buckling is an important challenge for structural engineers.

The effect of the geometrical parameters and anchorage condition of a tank are major factors affecting shell damage which has been confirmed by observation after previous earthquakes. A review of the failure modes of steel tanks during earthquakes in Chile in 1985 (M = 7.8) and 2010 (M = 8.8) showed that some tanks performed well during the 2010 earthquake due to the use of mechanical anchors, but that most of the self-anchored tanks failed in the 1985 earthquake [2]. Damage to stainless steel cylindrical tanks used for wine storage and fermentation caused by the South Napa earthquake ($M_w = 6.0$) included local buckling of tank walls, anchorage failure and damage at the top of the tanks to the catwalk system. It was concluded that more research is required to develop simplified nonlinear models for this type of tank design and to develop retrofitting methods [3].

Comparison of observed tank performance and that predicted by published theoretical methods performed on two tanks with different aspect ratios after the 2008 Silakhor earthquake in Iran ($M_L = 6.1$) produced results that were in good agreement. It was concluded that axial stresses on the tank shells uplifted during the earthquake depend upon the rigidity of the foundation [4]. The effects of past earthquakes indicate that tank geometry in terms of the height-to-diameter ratio, (H/D), is a major factor in shell buckling. Broad tanks ($H/D < 1$) generally experienced elephant-foot buckling (E), whereas slender tanks ($H/D > 1$) demonstrated diamond-shaped (D) buckling [5, 6].

Initial efforts to solve the shell buckling problem under axial compression led to the determination of linear bifurcation stress (or classic elastic critical buckling stress σ_{cl}) for a cylinder with simply supported ends and uniform membrane (in-plan) pre-buckling stress distribution. The boundary conditions, eccentricities and non-uniformities in the applied load, types of support, geometric imperfections and residual stress prompted initial experiments that confirmed that real cylinders buckle at much lower loads than that exhibited under classic buckling stress [7].

Experimental studies by Niwa and Clough on the seismic behaviour of cylindrical tall wine storage tanks on a shaking table showed that D buckling occurred in tall tanks while E buckling was more common in broad tanks. They concluded that the critical buckling stress assumed in American standards (API650 and AWWA-D100) for steel tank design are rather conservative estimations of buckling strength [8]. This conclusion was rejected by Jia and Ketter, but the need for revision of code provisions for E buckling was emphasized [9]. They concluded that E buckling may occur in anchored or unanchored tanks and that anchorage did not prevent the shell from buckling.

Rotter et al. carried out early systematic studies of elephant-foot buckling and derived preliminary results [10-13] that led to the development of a semi-empirical equation for estimation of E buckling load. El-Bkaily and Peek presented an algorithm to predict the location of the E buckling and the extent of bulging using the finite element method (FEM) [14]. The effects of ten seismic codes on different tanks was reviewed and compared by Jaiswal et al. [15]. Their study revealed significant differences among the codes on the design seismic forces for

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various types of tanks. Chen et al. compared resistance to E buckling according to the standards of different countries for oil tanks [16]. Virella et al numerically examined the critical horizontal peak ground acceleration which induced the buckling of a set of anchored cylindrical tanks [17].

Elephant-foot buckling of thick cylindrical shells under transverse loading was studied numerically by Chonghou et al. [18]. Their research showed that moderately thick cylindrical shells under transverse loading exhibited a D buckling mode while thick cylindrical shells exhibited an E buckling mode. This was significantly influenced by the radius to thickness ratio and the material yield strength, rather than by the length to radius ratio and the axial force. Chen et al. examined the effect of openings on E buckling of large non-anchored welded steel oil tanks and utilized a numerical simulation method to analyse the effect of size and position of openings on E buckling critical loading [19].

Yang et al. proposed an analytical formula for elastic-plastic instability of large oil storage tanks with perfect shell walls [20]. They concluded that the interaction of high hydraulic and axial compression causes changes in the material properties of the tank wall which causes the buckling strength of the tank wall to decrease rapidly. Elkashef et al. used nonlinear FEM to investigate the formation of E buckling in anchored tanks [21]. They used the von Mises yield criterion to determine the location and most important parameters in the occurrence of E buckling in vertical tanks. They concluded that the point at which the hoop stress reaches a maximum value was the most likely location for E buckling to occur in steel liquid tanks and estimated that this type of buckling typically occurs at 76.2 to 152.4 cm above the tank base.

Two recent review studies on shell buckling were carried out by Zingoni [22] and Godoy [23]. Zingoni [22] studied the strength, stability and vibration behaviour of liquid-containment shells along with the design of these facilities to withstand loading by liquid and wind pressure, ground movement and thermal effects. Godoy [23] studied the buckling problems of vertical, aboveground tanks under static or quasi-static loads caused by uniform pressure, wind, settlement of the foundation and fire.

Because of the low radius-to-thickness ratio in the cylinders and because shell buckling is the major failure mode of steel cylindrical tanks, rehabilitation methods have also been considered in recent years. The application of fibre-reinforced plastics (FRP) to strengthen the tank shells against E buckling has been considered. An experimental study on the buckling behaviour of unstiffened and stiffened cylindrical shells under quasi-axial loading showed an increase in buckling load as a result of the plastic fibres [24]. The effects of adding FRP composite material to the tank shell has been investigated using numerical and experimental methods. The results indicate the usefulness of these fibres as retrofitting to prevent the occurrence of E buckling and increasing the capacity of a steel cylindrical tank [25].

In addition to analytical, numerical and experimental procedures, probabilistic methods using fragility curves have been recognized as useful alternatives to the deterministic code prescription approach. Researchers believe that uncertainty related to structural performance versus excitation makes probabilistic seismic risk analysis one of the best methods for measuring the seismic performance of a structural system. A fragility curve reveals the probability of reaching or exceeding the limit state for a particular value of ground motion intensity. Consequently, to build a fragility curve, a failure criterion (limit state) for the analysed structure is required. Numerous failure criteria have been proposed in the literature. Increasing the capacity against failure is the desired outcome of most design methods [26-29]. It is clear from the literature mentioned above that the mechanism of shell buckling has been properly determined and agreed upon previously by researchers. The results of this agreement are reflected in the similar attitudes adopted by prominent codes. However, there are important unclear issues for those who want to use the outcomes of such research as it relates to shell buckling. For example, the interaction of various seismic factors with the tank geometry to allow optimization of the material used has not been addressed in previous studies. In other words, the effects of seismic parameters such as seismic zone, level of seismic intensity and soil-shell interaction on shell buckling have not been systematically addressed by researchers.

The present study discusses these issues on two levels. First, the paper tries to provide a comprehensive view of the effects of tank geometry on various types of shell damage. In this regard, a range of tank dimensions and related parameters that cover all practical dimensions of steel liquid tanks have been considered. A comprehensive parametric study also has been performed to determine the geometrical boundaries of each shell damage mode. The resulting classifications can benefit researchers who work in this field to properly select their geometric domain. Designers also are provided with comprehensive insight about the interaction of their preliminary geometries and possible critical seismic damage modes. Second, the paper also focuses on the effects of seismic factors on the selection of the optimum geometry for a tank. The interaction of cost-effective factors such as the land required on which to construct a liquid tank and the material required with which to construct a tank to store a certain volume of liquid are quantitatively discussed. An empirical relation is proposed herein based on a parametric study for preliminary estimation of the amount of steel material required for various seismic conditions and tank geometries.

EVALUATION OF SHELL BUCKLING

One of the most important failure modes of cylindrical storage tanks is shell buckling. Generally, shell buckling modes include shear buckling and bending buckling. Bending buckling comprises diamond-shaped (D) and elephant-foot (E) buckling. These buckling modes are related to the geometric parameters of a tank such as the aspect ratio or height to diameter (*H/D*), and the tank diameter to thickness (*D/t*). Shear buckling occurs for small *H/D* whereas the bending buckling occurs for large *H/D*. Shear buckling is caused by shear force and brings about many large diagonal wrinkles in the centre of a tank side wall. This type of buckling has not been considered in this study. In D buckling, the cross-section in the buckling region bends inward. In E buckling, the buckling cross-section expands outward in a ring and the structural strength decreases [30].

In addition to E and D buckling, secondary buckling (SB) also exists and is caused by external pressure and cavitation. Tanks that have failed and lost their contents sometime display substantial damage to the top half of their height, probably the result of the vacuum created by rapid loss of contents due to damage at the base (Figure 1). The current study focused mainly on E and D buckling modes of shell damage as they have frequently been observed in steel liquid tanks during previous earthquakes. E buckling has been explicitly considered in most major codes, such as Eurocode 8 [31] and the New Zealand guidelines [32, 33] and are implicitly accounted for in the API 650 formulation [34].

Eurocode8 and the New Zealand guidelines suggest the formula proposed by Rotter to compute the buckling capacity with respect to elastic–plastic buckling [35]. These equations are:

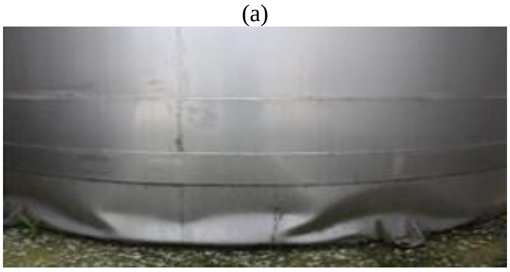
$$F_{mp} = f_{cl}\left[1-\left(\frac{PR}{F_Y t}\right)^2\right]\left(1-\frac{1}{1.12+\rho^{1.5}}\right)\left[\frac{\rho+(F_Y/50)}{\rho+1}\right] \quad (1)$$

$$\rho = R/(400t) \quad (2)$$

$$f_{cl} = E\,t/(R\sqrt{3(1-\nu^2)})(t/R) \cong 0.605E\frac{t}{R} \quad (3)$$

where R is the tank radius, t is the shell thickness, P is the total internal pressure, F_Y is the steel yielding stress, $v = 0.3$ is the Poisson's ratio, E is the Yang's modulus and f_{cl} is the classic elastic critical buckling stress produced by linear bifurcation analysis.

(a)



(b)

(c)

Figure 1: (a) E buckling in 2010 Maule earthquake [5]); (b) D buckling; (c) secondary buckling in 2012 Emilia earthquake [6].

INITIAL ESTIMATION OF STRUCTURAL DIMENSIONS

A cylindrical shape with a diameter of approximately 3 to 126 m is the most common form employed for a liquid storage tank [36]. The required capacity (Q) of a reservoir is determined by local requirements. For steel cylindrical tanks, the shell thickness is allowed to increase to 5 cm (2 in) and the maximum permissible height of the tank is determined based on the allowable bearing capacity of the soil upon which the tank is placed. Trial calculations are required to determine the best dimensions for a tank. The following simple equations are generally used for initial estimations [37]:

$$H_{max} = P_{Soil}/\gamma_L \tag{4}$$

$$D = \sqrt{(4 \times Q)/(\pi \times H_{max})} \tag{5}$$

where H_{max} is the maximum permissible height, P_{Soil} is the allowable soil pressure, γ_L is the specific weight of the contained liquid and D is the tank diameter. The shell plate is made from one or more horizontal plates. The tank thickness is calculated at the bottom of each course as:

$$T_h = \gamma_L \times h \times D \tag{6}$$

$$t_h = \gamma_L \times h \times D \;/\; 2fe \tag{7}$$

where t_h is the shell thickness, T_h is the shell tension, h is the depth from the tank top, e is joint efficiency and f is allowable unit stress.

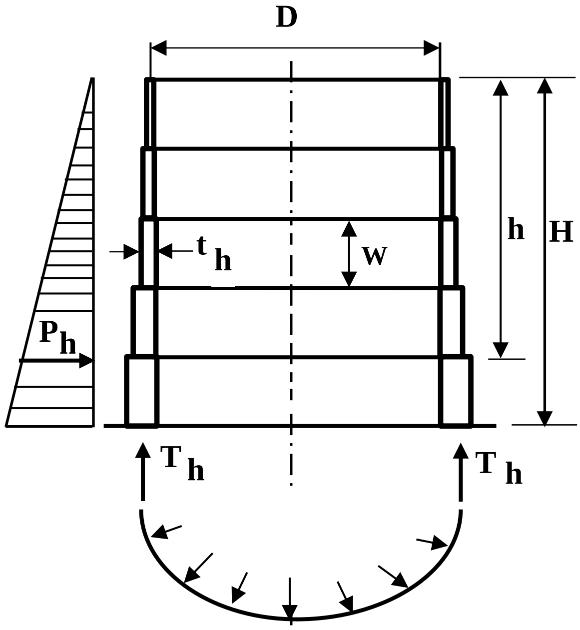


Figure 2: Structural shell design [37].

The American Petroleum Institute (API) has published specifications for welded steel oil storage tanks. Seismic loading was introduced to API 650 in the early 1980s with the addition of Appendix E (Seismic Design of Storage Tanks). Shell thickness in this code is calculated in a manner similar to the aforementioned procedure called the 'one-foot' method, which calculates the thicknesses required at design points 0.3 m (1 ft) above the bottom of each shell course as:

$$t_d = 4.9D(h - 0.3)G \;/\; S_d + CA \tag{8}$$

$$t_t = 4.9D(h - 0.3) \;/\; S_t \tag{9}$$

where t_d and t_t are the design shell thickness and hydrostatic test shell thickness (mm), respectively, G is the design specific gravity of the liquid (which should not be less than 1 because of the water test), CA is the corrosion allowance (mm), S_d and S_t are the allowable stress for the design condition and hydrostatic test condition (MPa), respectively. The required shell thickness should be greater than the design shell thickness (including any corrosion allowance) or hydrostatic test shell thickness, but the shell thickness should not be less than the values presented in Table 1.

Table 1: Minimum nominal shell thickness in API 650.

Tank nominal Diameter (m)	Nominal Plate Thickness (mm)
<15	5
15 to < 36	6
36 to 60	8
> 60	10

SEISMIC REQUIREMENTS OF STORAGE TANKS FROM NZSEE 2009

NZSEE published its recommendations for the seismic design of storage tanks (known as the 'Red Book') in 1986 [32]. This guideline generally does not allow yielding of tank elements, or a reduction of design forces for ductility; thus, it is somewhat more conservative than API 650-E. The revised edition (called the 'Blue Book') was presented in NZSEE 2009 [33]. This document is used in the current study because it provides reasonable procedures for determining seismic design actions in accordance with NZS 1170.5 [38]. In NZSEE 2009, more guidance is recommended to designers on the appropriate ductility factors depending on the tank configuration, anchorage and critical failure mechanism. Prescriptive guidance is given for combining the said modes and resolving the actions into base shear and overturning moment. In current practice, a range of damping levels is used so that, for the convective sloshing mode, damping is assumed to be about 0.5% and, for the impulsive mode, ξ_i=2% is common. The beneficial effects of additional radiation damping from soil-structure interaction can also provide significantly higher damping, This is calculated based on the tank properties and subsoil shear wave velocity from the figures in NZSEE 2009.

Ductility factor μ is applied for reduction of seismic forces and is calculated based on tank type and anchorage condition for the horizontal impulsive mode from the suggested table in NZSEE 2009. The value is 1.0 for the convective and vertical modes. Correction factor $k_f(\mu.\xi_i)$ has been provided for the NZS 1170.5 elastic site hazard spectrum (Figure 3) to account for the ductility and level of damping included in the shell design (Table 2). Return period factor R_u proposed by this code reflects a range of life-safety, property and environmental exposure values, plus the community significance derived based on the consequences of failure. Risk is classified as negligible, slight, moderate, serious or extreme based on the importance level, life safety and hazard level of the tank contents, environmental exposure, national or community significance and adjacent property value at direct risk from tank failure (2007 cost index). The corresponding values for the return period based on the consequences of failure are presented in Table 3.

Site Subsoil Class

The site subsoil class according to NZS 1170.5 shall be determined as being one of classes A to E according to the properties listed in Table 4.

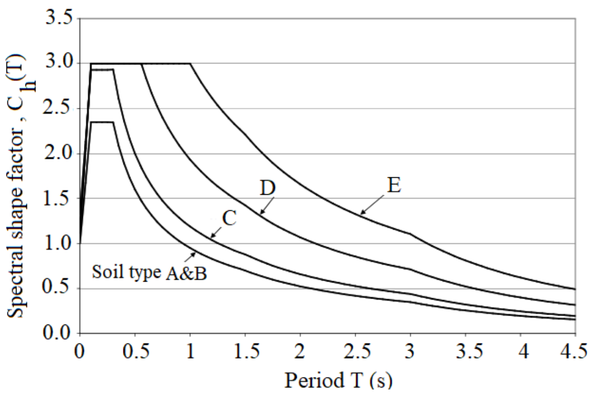


Figure 3: Spectral shape factor [38].

Table 2: Correction factor $k_f(\mu.\xi_i)$ to NZS 1170 elastic site hazard spectrum for damping and ductility [38].

Ductility μ	$k_f(\mu.\xi_i)$							
	ξ_i							
	0.5%	1%	2%	5%	10%	15%	20%	30%
1.00	1.67	1.53	1.32	1.00	0.76	0.64	0.56	0.47
1.25	1.08	1.04	0.96	0.82	0.67	0.58	0.52	0.44
2.00	0.91	0.89	0.84	0.74	0.63	0.55	0.50	0.43

Table 3: Return period factors [38].

Consequence of Failure	Return Period Factor (R_u)
Negligible	0.5
Slight	0.5
Moderate	1.0
Serious	1.3
Extreme	1.8

Table 4: Site subsoil classes [38].

Class	Description	Definition
A	Strong Rock	*UCS > 50 MPa & Vs30 > 1500 m/s & not underlain by < 18MPa or Vs 600m/s materials.
B	Rock	1 < UCS < 50 MPa & Vs30 > 360 m/s & not underlain by < 0.8 MPa or Vs 300 m/s materials, a surface layer no more than 3 m depth (HW-CW rock/soil).
C	Shallow Soil	not class A, B or E, low amplitude natural period ≤ 0.6s
D	Deep or Soft Soil	not class A, B or E, low amplitude natural period > 0.6s underlain by < 10 m soils with undrained shear strength < 12.5 KPa, or < 10 m soils SPT N < 6
E	Very Soft Soil	> 10m soils with undrained shear strength < 12.5 KPa, or > 10m soils with SPT N < 6, or > 10m soils with Vs ≤ 150m/s, or > 10m combined depth of previous properties.

*UCS= Unconfined compressive strength

Seismic Actions Based on NZSEE 2009

In the New Zealand recommendations, the dynamic response of liquid-tank systems is analysed using the Veletsos model (1984) for rigid tanks and Haroun-Housner model (1982) for flexible system. Based on this code, the horizontal seismic shear acting at the base of a tank and the overturning moment can be calculated using the following expressions:

$$V_i = C_d(T_i)m_i g \tag{10}$$

$$C_d(T_i) = C(T_i)k_f(\mu.\xi_i)S_P \tag{11}$$

$$C(T_i) = C_h(T_i)ZR_u N(T_i\ ,D) \tag{12}$$

$$V_f = C_d(\check{T}_f)[m_f + m_w + m_t]g \tag{13}$$

$$V_1 = C_d(T_1)m_1 g \tag{14}$$

$$M_r = C_d(\check{T}_0)[m_0 - m_f]gh_0 \tag{15}$$

$$M_f = C_d(\check{T}_f)[m_f h_f + m_w h_w + m_t h_t]g \tag{16}$$

$$M_1 = C_d(T_1)m_1 g h_1 \tag{17}$$

where m_i is the equivalent mass of a tank and its contents responding to the particular mode of vibration, m_0 is the rigid tank impulsive mass (total impulsive mass for the flexible tank), $m_r = m_0 - m_f$ is the rigid impulsive mass, m_f is the flexible impulsive mass, m_1 is the first convective mass, m_w and m_t are the wall and roof mass, respectively, T_1 is the period of vibration of the first convective mode of vibration and $\check{T}_0$ is the period of the impulsive mode of vibration for the tank-foundation system and for a rigid foundation $(\check{T}_0=0)$. $\check{T}_f$ is the period of vibration of the first horizontal tank-liquid impulsive mode, including the effects of foundation flexibility. For a rigid tank $\check{T}_f = 0$ and $m_f = 0$. T_i is the period of vibration of the appropriate mode of response. $C_d(T_i)$ is the horizontal design action coefficient for mode i, $C(T_i)$ is the ordinate of the elastic site hazard spectrum for horizontal loading for the site subsoil type and the relevant mode obtained from NZS 1170.5 according to tank importance level. $C_h(T_i)$ is the spectral shape factor for the subsoil type site and relevant mode (Figure 3), Z is the seismic zone hazard factor, $N(T_i\ D)$ is the near fault factor, μ is the displacement ductility factor for horizontal impulsive modes and is taken as 1.0 for the convective and vertical modes and 1.25 for the impulsive mode.

Shell Buckling Criteria in NZSEE 2009

In NZSEE 2009, the allowable shell buckling stress is calculated from the following equations. Stresses due to hydrodynamic pressure or overturning moment plus the shell weight (f_m) shall not exceed the stress required to induce buckling under membrane compression (F_m) according to the following equations:

$$f_m = (W_s / A) + (M_{OT} / \pi R^2 t) \quad \le \quad F_m \tag{18}$$

Allowable E buckling (F_{mp}) can be calculated using Eq. (1) as suggested by Rotter and the allowable D buckling (F_{me}) can be calculated as:

$$F_{me} = \left[0.19 + 0.81(f_p/f_{cl})\right] f_{cl} \tag{19}$$

$$f_p = f_{cl}\sqrt{\left((1-(1-\bar{p}/5)^2(1-(f_0\,/\,f_{cl})^2)\right)} \le f_{cl} \tag{20}$$

$$f_o = f_y(1-\lambda^2\,/\,4) \;\rightarrow\; \{\,where\ \left(\lambda^2 = f_y\,/\,\bar{\sigma} f_{cl}\right) \le 2\,\} \tag{21}$$

$$f_o = \bar{\sigma}\, f_{cl} \qquad\qquad \rightarrow\; \{\,where\ \left(\lambda^2 = f_y\,/\,\bar{\sigma} f_{cl}\right) \ge 2\,\} \tag{22}$$

$$\bar{\sigma} = 1 - \Psi(\delta\,/\,t)\left[(1+\tfrac{2}{\Psi\,(\delta\,/\,t)})^{1/2} - 1\right] \ ,\ (\Psi = 1.24) \tag{23}$$

$$\delta/\,t = (0.06/a)\sqrt{R/t} \tag{24}$$

$$\bar{P} = P\,R\,/\,t\,f_{cl} \tag{25}$$

where the value of the parameter "a" for normal, quality and very high quality construction is 1.00, 1.5 and 2.5, respectively.

Simultaneous shaking in both a single horizontal direction and the vertical direction shall be considered by combining of the components by the square root of the sum of the squares (SRSS). These equations for calculation of the stress levels should be corrected using the effect of internal hydrostatic (P_h), impulsive (P_i), convective (P_1) and vertical pressures (P_V). For elastic D buckling, the limiting condition will be when the internal pressure is at a minimum. This occurs when the vertical acceleration is at a maximum in the upward direction, thus reducing the hydrostatic pressure. All pressure should be calculated at the base of the tank using selected ductility facto μ as:

$$P = P_{min} = P_h + \sqrt{(P_i)^2 + (P_1)^2} - P_V \tag{26}$$

For E buckling, the limiting condition will be when the internal pressure is at a maximum as:

$$P = P_{max} = P_h + \sqrt{(P_i)^2 + (P_1)^2 + (P_V)^2} \tag{27}$$

Determining material yielding caused by tensile hoop stresses is performed using the following equation and should be less than material yield stress (F_y):

$$f_{Hoop} = f_h + \sqrt{(f_i)^2 + (f_1)^2 + (f_V)^2} \ \le\ F_y \tag{28}$$

PARAMETRIC STUDY OF SHELL BUCKLING PHENOMENON

To provide a comprehensive assessment of the effects of tank geometry on the various shell damage types, a range of tank dimensions and parameters has been considered here to include all practical ranges of liquid tanks. All selected tanks are assumed to be anchored and categorized according to the effective parameters for shell buckling. These parameters include the height of liquid content (*H*), aspect ratio of a tank (*H/D*), soil type, zone seismicity and risk classification. Eight values for tank height H =2.4 to 19.2 have been chosen and, for each specific tank height, the tank aspect ratio ranges from H/D = 0.2 to 2. Four seismic intensity levels as well as three soil categories have been considered as shown in Figure 4. The density of the contained liquid is $\rho = 1000$ (kg/m^3) and the steel type A36M for which $F_Y = 250$ MPa has been considered. Based on these variables, a total of 4224 tanks have been analysed in numerical analysis to cover all practical dimensions. The assumed nominal roof plate thickness and slope are $t_r = 5$ mm and $\theta_r = 5^0$, respectively. The effect of tank wall flexibility has been considered in addition to the impulsive period correction for the soil-structure interaction (foundation flexibility) according to NZSEE 2009 recommendations.

NUMERICAL ANALYSIS

To evaluate the effects of various parameters on the critical buckling mode of the considered tanks, a comprehensive program is provided based on the provisions of NZSEE 2009 for the design process. The cases introduced in the previous section are analysed using the developed program according to the algorithm presented in Figure 5. As the two main outcomes, the required shell thickness and the demanded steel volume to build a tank are extracted from the results of these analyses.

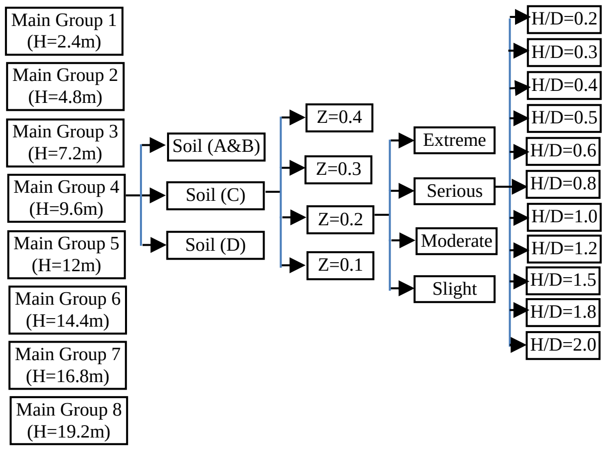


Figure 4: Categories and parameters of all tanks.

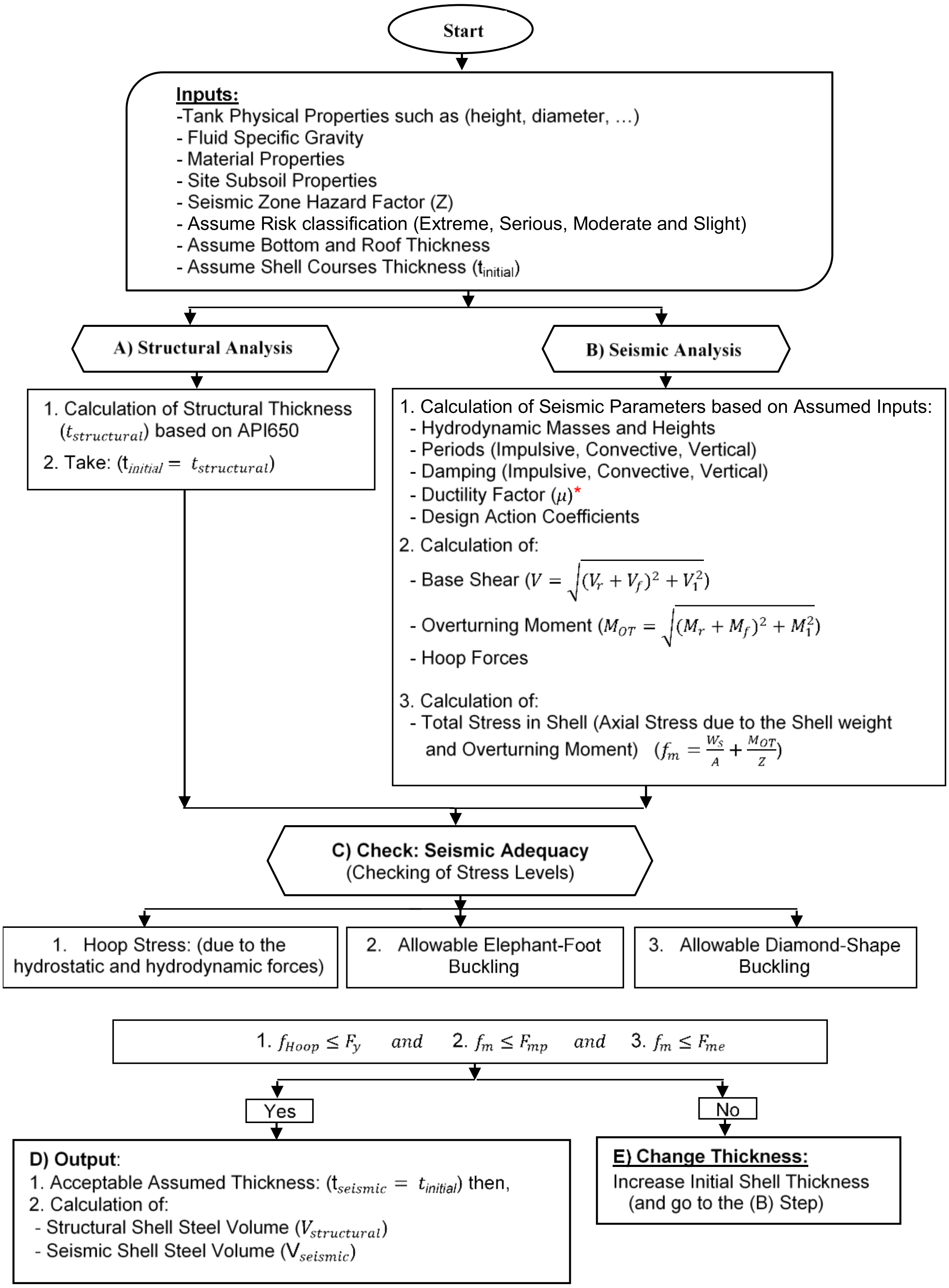
GOVERNING BUCKLING DOMAIN BASED ON A TANK GEOMETRY

Essentially, three types of failure mode may occur in the shell of a liquid tank: (1) Material yielding due to hydrodynamic hoop stress (*H*); (2) Elastic D buckling and; (3) Elasto-plastic E buckling. The shell thickness should be thick enough to prevent these failure modes. For a specific tank, one of these buckling modes may be critical depending upon tank dimensions and the governing seismic conditions. For researchers who study shell buckling, it is important to select the proper range of tank dimensions for their parametric studies. Likewise, tank designers must choose an appropriate primary tank geometry based on the governing seismic conditions. The goal of the analyses presented here is to provide better insight about the relation of tank geometry to dominate type of buckling.

Figures 6 to 8 show the results of analysis for selected tanks having various risk factors (risk factor $= ZR_u$). In these figures, the distribution of buckling stress in the vertical direction of the tank shell are illustrated versus the required thickness of the tank shell. It can be seen that E buckling is generally the governing failure mode near the base level for all cases considered, as has been observed in field observations during past earthquakes. Although the buckling stress is different for the different vertical levels of a tank shell, a critical type of buckling mode can be found for each case.

Note that although the soil type increases the required steel shell volume, it does not considerably affect the failure mode; thus, the figures are presented only for tanks placed on soil D. This critical failure mode has been extracted and presented in Table

5 for all considered cases where H, E and D represent the required shell thickness for critical failure modes of material yielding due to hoop stress, elephant-foot buckling and diamond shape buckling, respectively. Based on the results in Table 5, the failure modes in low-height tanks (H < 8 m) are limited to a high *H/D* value and high risk factors. In the other ranges, the structural thickness is usually adequate and there is no failure mode.



** The investigated tanks in current study are assumed to be anchored with non-ductile holding down bolts and according to the table 3.1 in NZSEE2009, ductility factor in theses tanks can be considered as 1.25. Therefore, this value was chosen for ductility factor in this study. Steel volume predicted here may be reduced if ductile behaviour is expected.*

Figure 5: Algorithm developed to evaluate the critical buckling mode.

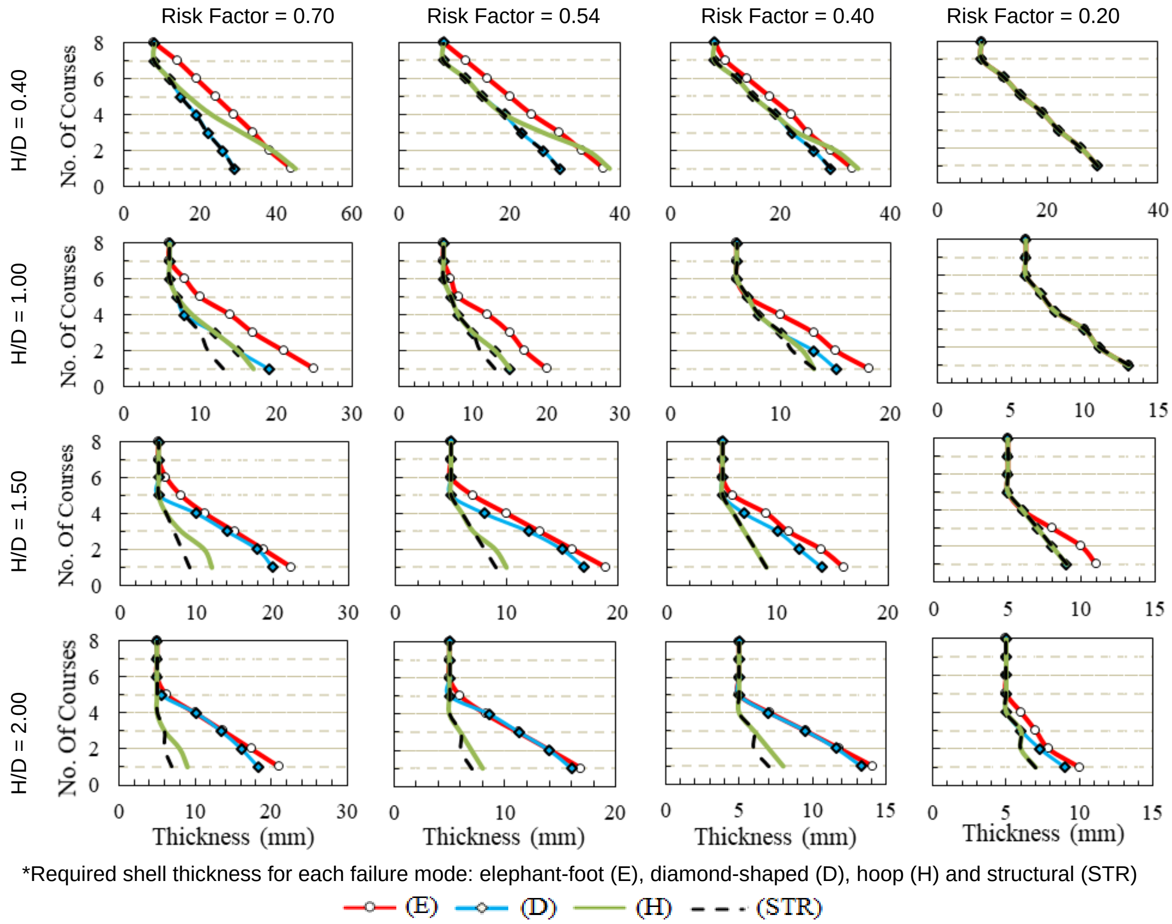


Figure 6: Required shell thickness for each failure mode (H=19.2 m, Soil D).

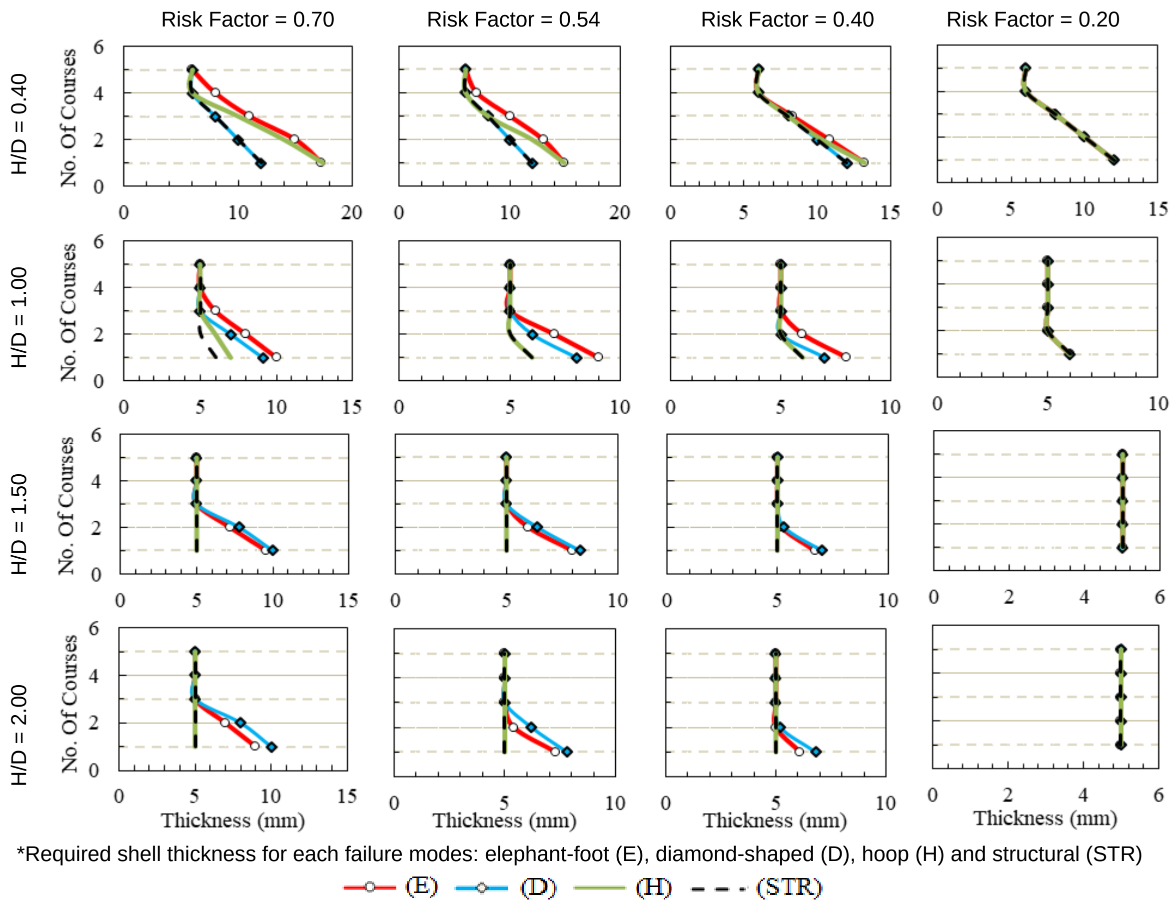


Figure 7: Required shell thickness for each failure mode (H = 12 m, Soil D).

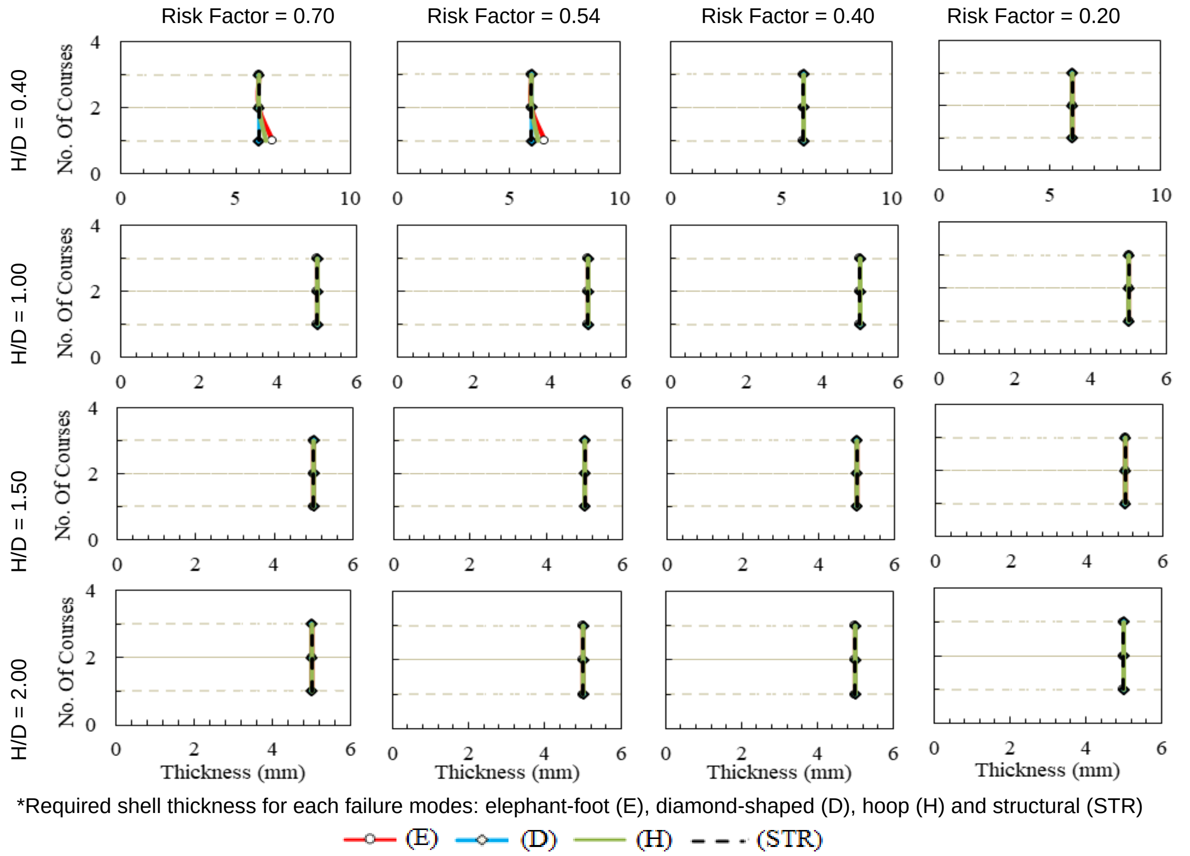


Figure 8: Required shell thickness for each failure mode (H = 7.2 m, Soil D).

Table 5: Prominent failure modes in tank shell for all tanks.

		Risk Factor																																		
		0.70							0.54							0.4							0.3							0.2						
		Height (m)							Height (m)							Height (m)							Height (m)							Height (m)						
	H/D	19.2	16.8	14.4	12.0	9.6	7.2	4.8	19.2	16.8	14.4	12.0	9.6	7.2	4.8	19.2	16.8	14.4	12.0	9.6	7.2	4.8	19.2	16.8	14.4	12.0	9.6	7.2	4.8	19.2	16.8	14.4	12.0	9.6	7.2	4.8
Soil A & B	0.2	H	H	H	H	H	H	H	H	H	H	H	H	H	-	H	H	H	H	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.3	H	H	H	H	H	H	E	H	H	H	H	H	H	-	H	H	H	H	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.4	H	H	H	H	E	E	E	H	H	H	H	E	E	-	H	H	H	H	E	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.5	E	E	E	E	E	E	E	E	E	E	E	E	E	-	H	E	E	E	E	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.6	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	E	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.8	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	E	-	-	E	E	E	E	-	-	-	-	-	-	-	-	-	-
	1.0	E	E	E	E	E	D	D	E	E	E	E	E	E	-	E	E	E	E	E	E	-	E	E	E	E	E	-	-	-	-	-	-	-	-	-
	1.2	E	E	E	E	D	D	D	E	E	E	E	E	D	D	E	E	E	E	E	E	-	E	E	E	E	E	-	-	-	-	-	-	-	-	-
	1.5	E	E	D	D	D	D	D	E	E	E	D	D	D	D	E	E	E	E	D	D	-	E	E	E	E	E	D	-	E	E	E	-	-	-	-
	1.8	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	E	D	D	D	-	E	E	E	E	-	-	-
	2.0	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	E	D	D	D	-	E	E	E	E	-	-	-
Soil C	0.2	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.3	H	H	H	H	H	H	E	H	H	H	H	H	H	E	H	H	H	H	H	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.4	H	H	H	H	E	E	E	H	H	H	H	E	E	E	H	H	H	H	E	E	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.5	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	-	-	-	-	-	-	-	-	-	-	-	-
	0.6	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	-	-	-	-	-	-	-	-	-	-	-	-
	0.8	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	-	-	-	-	-	-	-	-	-	-
	1.0	E	E	E	E	E	D	D	E	E	E	E	E	E	D	E	E	E	E	E	E	-	E	E	E	E	E	-	-	E	E	-	-	-	-	-
	1.2	E	E	E	E	D	D	D	E	E	E	E	D	D	D	E	E	E	E	E	D	-	E	E	E	E	E	-	-	E	E	-	-	-	-	-
	1.5	E	E	E	D	D	D	D	E	E	E	D	D	D	D	E	E	E	D	D	D	D	E	E	E	E	D	D	-	E	E	E	E	E	-	-
	1.8	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	E	D	D	D	-	E	E	E	E	E	-	-
	2.0	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	-	E	E	E	E	D	-	-
Soil D	0.2	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.3	H	H	H	H	H	H	E	H	H	H	H	H	H	E	H	H	H	H	H	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.4	H	H	H	H	E	E	E	H	H	H	H	E	E	E	H	H	H	H	E	E	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
	0.5	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	-	-	-	-	-	-	-	-	-	-
	0.6	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	-	-	-	-	-	-	-	-	-	-
	0.8	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	E	-	E	E	E	E	-	-	-	-	-	-	-	-	-	-
	1.0	E	E	E	E	E	D	D	E	E	E	E	E	D	D	E	E	E	E	E	E	-	E	E	E	E	E	-	-	E	E	-	-	-	-	-
	1.2	E	E	E	E	E	D	D	E	E	E	E	E	D	D	E	E	E	E	E	D	-	E	E	E	E	E	-	-	E	E	-	-	-	-	-
	1.5	E	E	E	D	D	D	D	E	E	E	D	D	D	D	E	E	E	D	D	D	D	E	E	E	E	D	D	-	E	E	E	E	E	-	-
	1.8	E	E	E	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	E	D	D	D	-	E	E	E	E	E	-	-
	2.0	E	E	E	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	D	E	E	D	D	D	D	-	E	E	E	E	D	-	-

(-) : Structural thickness provides satisfactory seismic safety (H): Hydrodynamic Hoop Stress
(E) : Elephant-Foot Buckling (D): Diamond Shape Buckling

Considering the dominant failure mode for each case, it appears that the critical buckling mode mainly depends on the tank height (*H*) and aspect ratio (*H/D*). It can also be observed that E buckling is rarely the dominant mode in broad tanks ($H/D \leq 0.4$) and, thus, this type of failure mode can be ignored for them. In fact, for broad tanks, the dominant failure mode of a tank shell is essentially caused by the dynamic H stress. Practically speaking, E buckling can be expected in medium or slender tall tanks. Elastic D buckling was observed to be critical in slender tanks ($H/D \geq 1.2$) of medium height ($H \leq 14$). This type of buckling is not likely to happen for broad or medium tanks ($H/D \leq 1.2$) as well as slender tanks with large tank height ($H \geq 14$).

Field observations of past earthquakes have demonstrated a similar trend in which E buckling is critical in medium tanks ($H/D = 1$) and D buckling in slender tanks ($H/D = 2$) [5, 6]. Past observations are in agreement with the general trend of the current study results. Note that, although looser soil reduces the critical buckling stress, the type of critical buckling mode does not change considerably in response to a change in soil type. Therefore, the effects of soil type on the dominant failure mode can be ignored.

PARAMETRIC STUDY ON DEMANDED STEEL FOR TANKS

Parameters such as welding, transportation and site conditions can affect the total fabrication cost of a tank. The demanded cost of each factor mainly depends on the regional conditions where the tank will be placed. Most of these factors can be estimated based on the material required for construction of the tank. In the current study, the total quantity of demanded steel is considered as the main component of the total cost. This assumption may not always be the case, but does cover most practical situations.

To acquire initial insight into the interaction of seismic effects and the cost of a liquid tank, the volume of required material should be considered. The required shell thickness of the tanks at various vertical levels were extracted in the previous section and can be used to calculate the total volume of demanded steel for each tank. Five important parameters should be considered to determine the extra material necessary to counter the effect of seismic activity: (1) height of the tank; (2) tank aspect ratio; (3) soil type; (4) seismic intensity; and (5) level of accepted seismic risk.

Note that the steel volume required to resist hydrostatic loads (structural steel) may be sufficient to resist seismic loads as well. However, the presence of seismic loads demands more steel than the required structural steel for most loads (seismic steel) and can be determined by subtracting the structural steel from total required steel. Knowing the seismic steel volume for a comprehensive set of practical circumstances can give valuable feedback about construction costs which have not been discussed in previous studies. Moreover, a designer can select the optimum dimension for the primary design mainly by comparing the cost of the land and material required to build a tank in high seismic zones. Evaluation of the demanded steel is necessary to determine if employing a particular seismic reduction method is economically valuable.

By way of example, the extra seismic steel for all considered tanks and for the high and low levels of seismic intensity ($Z = 0.4$ and 0.1) are presented in Figures 9 and 10. It can be seen that for broad tanks under low seismic intensity ($Z = 0.1$), an extra volume of seismic steel is not required and the structural steel is sufficient to resist seismic loads. Even for slender tanks under low seismic intensity, the extra required seismic steel volume is not more than 15% for the high level of accepted seismic risk. For a slight level of accepted seismic risk, the extra required steel volume is not considerable (less than 10%) in high seismic intensity zones. However, for moderate, serious and extreme levels of accepted seismic risk, the extra steel volume demanded by seismic loads can reach 15%, 25% and 45% for broad tanks, respectively. For slender tanks, these values are even higher, at up to 40%, 60% and 100%, respectively. It is clear that in places with high seismicity, using a smaller height and lower aspect ratio can significantly reduce the required material and related construction cost. Therefore, a proper economic balance should be struck between the cost of a higher tank diameter (more occupied land) and lower amount of material required to store a certain volume of liquid.

When considering soil type effects, it should be mentioned that, regardless of tank geometry, the demanded steel volume for tanks built on soft soil (type D) can be up to 40% more than for those placed on rocky soil (types A and B). This value is smaller for lower levels of accepted seismic damage. Considering this major difference, the use of different methods of enhancing soil strength or employing different types of tank foundation is an economically reasonable alternative for tanks placed on soft soil. In Figures 9 and 10, the extra steel volume demanded by the seismic loads are shown for extreme (EX Steel Diff%), serious (SE Steel Diff%), moderate (MO Steel Diff%) and slight (SL Steel Diff%) levels of seismic risk. As an example, the list of input parameters used for specific tank geometries and seismic conditions is presented in Table 6.

Table 6: Input parameters for a typical example tank.

Tank Geometric Parameters	
Diameter	$D = 15\ m$
Height of Liquid	$H = 12\ m$
Height to Diameter ratio	$H/D = 0.8$
Shell initial thick. (mm)	"Assumed"
Minimum shell thickness	$6\ mm$
Bottom plate thick. (mm)	$8\ mm$
Roof plate thick. (mm)	$6\ mm$
Roof slope	5%
Corrosion allowance (mm)	$1\ mm$
Fluid Property	
Specific gravity of fluid	Water ($SG = 1.0$)
Fluid density	$\rho_w = 1000\ kg/m^3$
Material	
A36	$F_Y = 250\ MPa$
Density	$\rho_s = 7850\ kg/m^3$
Poisson's 's ratio	$\nu = 0.3$
Young's modulus	$E = 200\ GPa$
Seismic Input Parameters	
Product Design Stress	$S_d = 160\ MPa$
Hydrostatic test Stress	$S_t = 171\ MPa$
Site subsoil class	"C"
Seismic zone hazard factor	$Z = 0.4$
Consequence of Failure	"Extreme"
Distance to near fault	$25\ km$

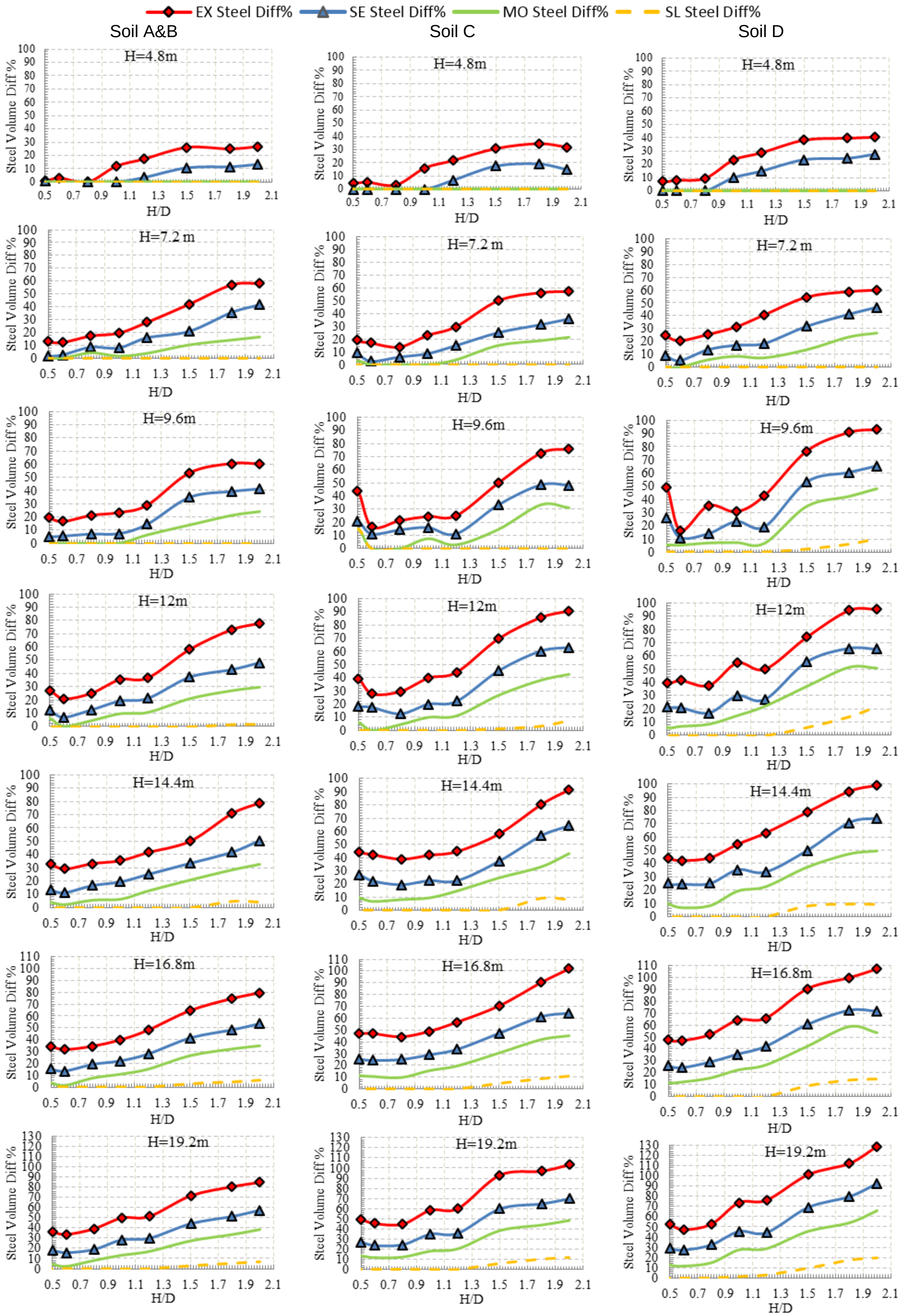


Figure 9: Extra steel volume demanded by seismic loads in high seismic zones (Z = 0.4).

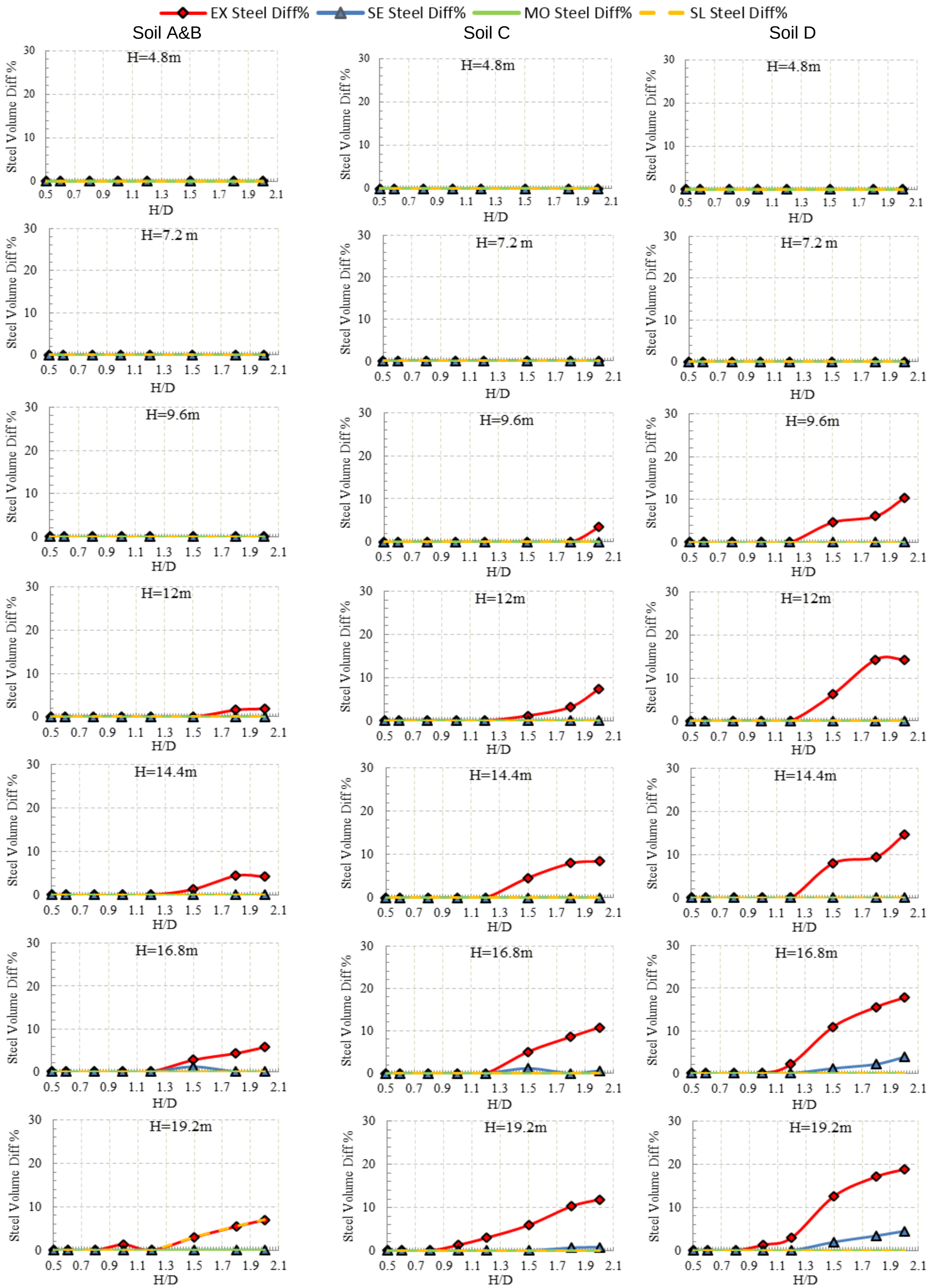


Figure 10: Extra steel volume demanded by seismic loads in high seismic zones (Z = 0.1).

ESTIMATION OF DEMANDED MATERIAL FOR TANK CONSTRUCTION

From the results of numerical analysis, it can be observed that the variation in the total demanded steel for the tanks versus the capacity of the tank can be properly approximated as a linear function. Figure 11 shows examples of such graphs for high intensity seismic zones. As can be seen, for a specific tank capacity and aspect ratio using a higher tank height can substantially increase the demanded material. In all circumstances, choosing the lower tank height for a specific volume of contained liquid is economically preferable in high seismic zones.

The variation in required material volume versus tank capacity is generally more useful for a specific aspect ratio (*H/D*). This variation can be adequately approximated using a second-order function for all tanks as can be seen in Figure 12 for extreme risk, a high seismic zone hazard factor ($Z = 0.4$) and soil types A and B. The results were similar for other cases; thus, they have not been presented here. The second-order polynomial equations on the plots are the trend lines obtained by the Excel curve fitting function. As can be seen in Figure 12, the second-order equation appears to properly predict the results of analysis; therefore, this function has been used to estimate the total quantity of demanded steel.

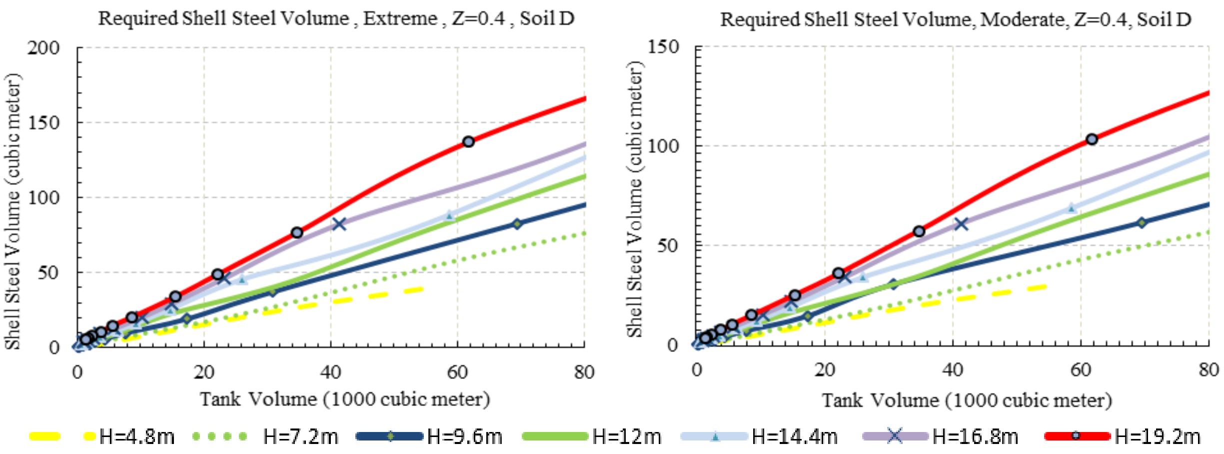
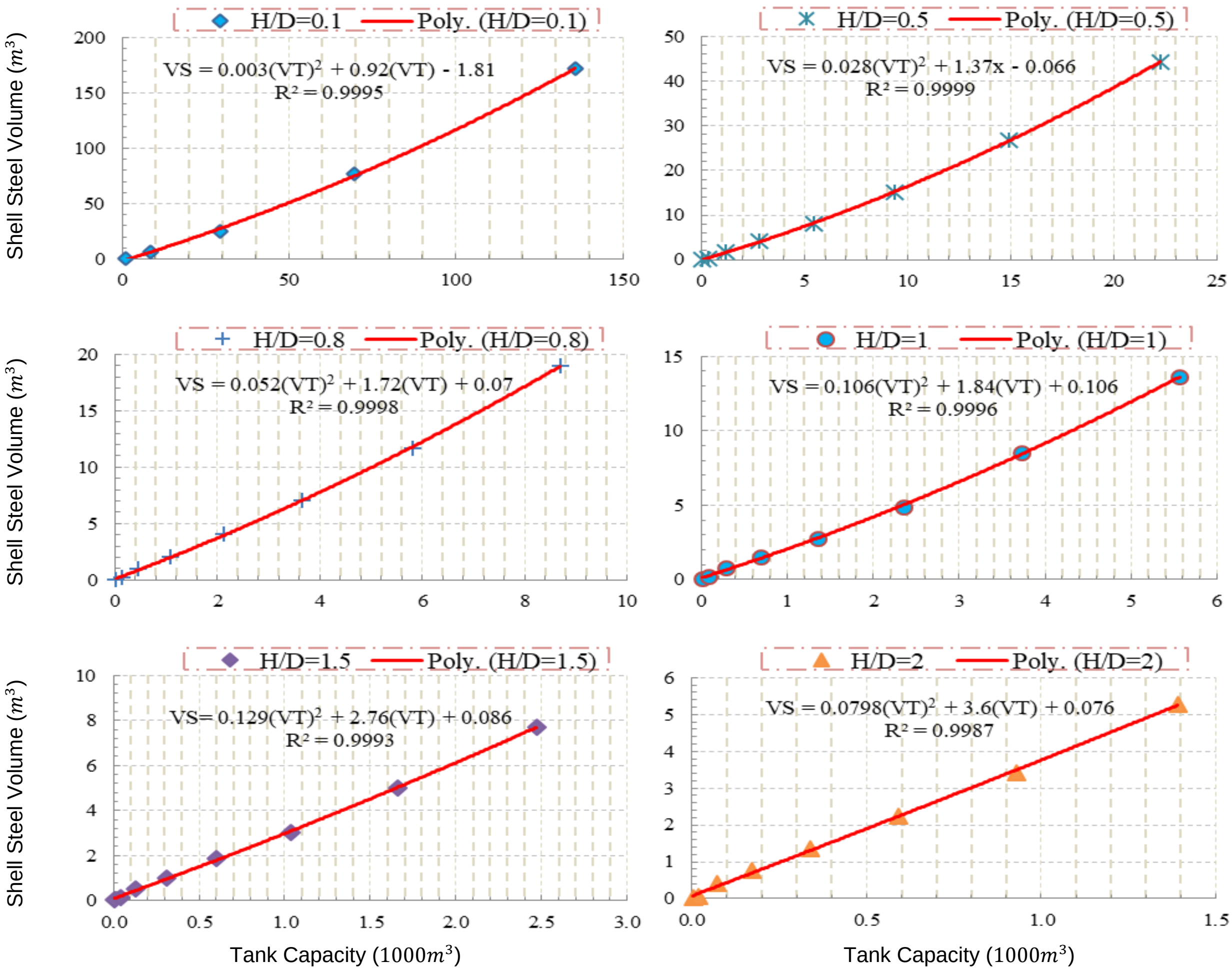


Figure 11: Demanded steel volume versus tank capacity for various tank heights and seismic parameters (H/D = 0.25 - 2)



* VS=Required steel volume in tank shell (excluded the roof and floor steel) & VT=Tank Capacity

Figure 12: Required shell steel volume versus tank capacity for different H/D ratios (Z = 0.4; extreme; soil A&B.)

The general form of this relation in which the demanded steel volume for building a tank is expressed as a second-order function of tank capacity can be written as:

$$V_S = \alpha V_T^2 + \beta V_T + \gamma \tag{29}$$

In this relation, V_T and V_S are the total volume of stored liquid and the total demanded steel volume for construction of a certain tank (excluded the roof and floor steel), respectively. The units used for V_S is in m^3 and for V_T is in $1000m^3$. Coefficients α, β and γ depend on the active seismic variables. By considering six risk factors, the numerical value of α, β and γ can be extracted from the results of analysis and are presented in Table 7 for all considered cases. Using this table, the total demanded steel volume can be properly estimated in the initial design stage. The optimum dimensions of a tank for storing of a specific volume of liquid then can be approximated based on the governing economic variables.

Table 7: Coefficients for calculating required shell steel volume

		Risk Factor																	
		0.70			0.54			0.40			0.30			0.20			0.13 (Structural)		
	H/D	α	β	γ	α	β	γ	α	β	γ	α	β	γ	α	β	γ	α	β	γ
Soil A & B	0.1	0.003	0.92	-1.82	0.003	0.76	-1.25	0.002	0.70	-0.83	0.002	0.67	-0.66	0.002	0.69	-0.79	0.002	0.69	-0.79
	0.2	0.004	1.15	-1.29	0.004	1.03	-0.96	0.003	0.98	-0.94	0.003	0.94	-0.66	0.003	0.94	-0.66	0.003	0.94	-0.66
	0.3	0.011	1.28	-0.53	0.009	1.16	-0.46	0.008	1.06	-0.31	0.007	0.99	-0.12	0.007	0.99	-0.10	0.007	0.99	-0.99
	0.4	0.018	1.36	-0.31	0.015	1.21	-0.14	0.014	1.08	0.01	0.010	1.10	-0.03	0.010	1.10	-0.03	0.010	1.10	-0.03
	0.5	0.028	1.37	-0.07	0.025	1.19	0.10	0.020	1.12	0.17	0.018	1.10	0.17	0.018	1.10	0.17	0.018	1.10	0.17
	0.6	0.037	1.43	0.07	0.030	1.27	0.18	0.020	1.24	0.18	0.017	1.25	0.17	0.017	1.25	0.17	0.017	1.25	0.17
	0.8	0.052	1.72	0.07	0.030	1.63	0.09	0.024	1.50	0.12	0.020	1.45	0.15	0.015	1.47	0.14	0.015	1.47	0.14
	1.0	0.106	1.84	0.11	0.052	1.83	0.08	0.045	1.68	0.13	0.031	1.60	0.14	0.018	1.62	0.14	0.018	1.62	0.14
	1.2	0.140	2.03	1.14	0.088	1.96	0.13	0.069	1.81	0.16	0.050	1.72	0.18	0.028	1.70	0.18	0.028	1.70	0.18
	1.5	0.129	2.76	0.09	0.054	2.50	0.10	-0.063	2.45	0.10	-0.028	2.22	0.13	-0.136	2.26	0.13	-0.127	2.19	0.14
	1.8	0.023	3.36	0.07	-0.020	2.95	0.09	-0.075	2.82	0.10	-0.124	2.65	0.11	-0.273	2.62	0.11	-0.350	2.66	0.12
	2.0	0.080	3.61	0.08	0.090	3.17	0.09	-0.012	3.03	0.10	-0.182	2.85	0.11	-0.502	3.00	0.11	-0.567	2.97	0.11
Soil C	0.1	0.004	0.91	-1.47	0.003	0.81	-1.30	0.003	0.72	-0.94	0.002	0.67	-0.75	0.002	0.69	-0.79	0.002	0.69	-0.79
	0.2	0.005	1.23	-1.77	0.004	1.08	-1.27	0.003	0.98	-0.94	0.003	0.94	-0.66	0.003	0.94	-0.66	0.003	0.94	-0.66
	0.3	0.013	1.39	-0.69	0.010	1.23	-0.58	0.010	1.08	-0.32	0.008	0.99	-0.12	0.007	0.99	-0.10	0.007	0.99	-0.99
	0.4	0.022	1.44	-0.35	0.017	1.28	-0.24	0.015	1.15	-0.11	0.011	1.10	-0.03	0.010	1.10	-0.03	0.010	1.10	-0.03
	0.5	0.033	1.46	-0.86	0.027	1.27	-0.01	0.023	1.17	0.08	0.018	1.10	0.16	0.018	1.10	0.17	0.018	1.10	0.17
	0.6	0.038	1.59	-0.06	0.028	1.42	0.07	0.026	1.27	0.16	0.017	1.25	0.17	0.017	1.35	0.17	0.017	1.25	0.17
	0.8	0.054	1.80	0.03	0.041	1.63	0.08	0.032	1.51	0.12	0.013	1.51	0.13	0.015	1.47	0.14	0.015	1.47	0.14
	1.0	0.110	1.95	0.10	0.079	1.79	0.10	0.046	1.73	0.11	0.048	1.58	0.15	0.018	1.62	0.14	0.018	1.62	0.14
	1.2	0.148	2.14	0.12	0.110	1.94	0.13	0.081	1.85	0.15	0.072	1.69	0.19	0.028	1.70	0.17	0.028	1.70	0.18
	1.5	0.170	2.81	0.09	0.092	2.57	0.10	-0.110	2.48	0.10	-0.038	2.33	0.12	-0.102	2.21	0.13	-0.127	2.19	0.14
	1.8	0.120	3.48	0.07	0.014	3.13	0.09	0.148	3.00	0.08	-0.176	2.80	0.10	-0.257	2.63	0.12	-0.350	2.66	0.12
	2.0	0.110	3.93	0.05	-0.102	3.50	0.08	-0.096	3.17	0.09	-0.228	3.04	0.10	-0.403	2.95	0.11	-0.567	2.97	0.11
Soil D	0.1	0.004	0.91	-1.40	0.003	0.79	-1.35	0.003	0.71	-1.21	0.004	0.55	0.07	0.002	0.63	-0.76	0.002	0.69	-0.79
	0.2	0.005	1.27	-1.73	0.004	1.10	-1.30	0.003	1.01	-1.09	0.003	0.94	-0.66	0.003	0.94	-0.66	0.003	0.94	-0.66
	0.3	0.013	1.45	-0.86	0.011	1.25	-0.69	0.010	1.11	-0.46	0.008	0.98	-0.11	0.007	0.99	-0.10	0.007	0.99	-0.10
	0.4	0.021	1.51	-0.40	0.017	1.30	-0.23	0.015	1.16	-0.09	0.013	1.06	0.04	0.010	1.10	-0.03	0.010	1.10	-0.03
	0.5	0.035	1.44	-0.06	0.028	1.26	0.01	0.022	1.17	0.08	0.019	1.09	0.19	0.018	1.10	0.17	0.018	1.10	0.17
	0.6	0.039	1.6	-0.05	0.034	1.37	0.12	0.034	1.37	0.12	0.017	1.25	0.17	0.017	1.25	0.17	0.017	1.25	0.17
	0.8	0.063	1.84	0.04	0.048	1.66	0.07	0.048	1.66	0.07	0.028	1.43	0.16	0.015	1.47	0.14	0.015	1.47	0.14
	1.0	0.120	2.12	0.04	0.082	1.90	0.08	0.082	1.90	0.08	0.041	1.65	0.14	0.018	1.62	0.14	0.018	1.62	0.14
	1.2	0.180	2.24	0.11	0.120	2.05	1.30	0.121	2.05	0.13	0.080	1.76	0.16	0.028	1.70	0.18	0.028	1.70	0.18
	1.5	0.220	3.04	0.07	0.130	2.76	0.09	0.130	2.76	0.09	0.032	2.32	0.12	-0.073	2.26	0.13	0.127	2.19	0.14
	1.8	0.240	3.65	0.05	0.045	3.37	0.07	0.045	3.37	0.07	-0.019	2.76	0.10	-0.196	2.69	0.11	0.350	2.66	0.12
	2.0	0.170	4.23	0.04	0.080	3.67	0.08	0.081	3.67	0.08	-0.107	3.07	0.10	-0.260	2.91	0.11	0.567	2.96	0.11

CONCLUSIONS

The present study qualitatively and quantitatively provides comprehensive insight into the interaction of seismic parameters and shell buckling modes of a cylindrical steel liquid storage tank. For this purpose, a range of tank dimensions and related parameters covering all practical cases was considered in a comprehensive parametric study. The range of tank geometries for each shell damage type was determined based on the available provisions. The interaction between the governing shell buckling mode, tank geometry and various seismic parameters was investigated and the following conclusions were drawn:

1) The critical buckling mode mainly depends on both the tank height (H) and the tank aspect ratio (H/D).
2) For broad tanks ($H/D \leq 0.4$), the dominant failure mode of a tank shell is mainly caused by the dynamic hoop stress. For medium tanks ($0.4 < H/D \leq 1.2$), the governing buckling mode of failure is mainly elephant-foot buckling.
3) For slender tanks ($H/D \geq 1.2$), the diamond-shaped buckling mode is the governing mode, except for tall tanks. For tall slender tanks ($H \geq 14$), elephant-foot buckling should be typically expected.
4) Although looser or softer soil reduces the buckling stress, the critical buckling mode in the shell of a tank does not alter in response to a change in soil type; therefore, the effects of soil type on the dominant failure mode can be ignored.

In the second part of the study, the effects of seismic parameters on the economics of a project as well as the selection of the optimum tank geometry for a specific seismic area have been discussed and the following conclusions were drawn:

5) In low seismic intensity zones ($Z = 0.1$ and 0.2), there is no need for the extra material demanded by seismic loads for broad tanks; the structural steel will be sufficient to resist seismic loads. Also, for slender tanks, the extra material required by seismic loads is not considerable (less than 13%).

6) In high seismic intensity zones ($Z = 0.4$), the extra steel material demanded by seismic loads is not considerable (less than 10%) if the level of seismic risk is low or slight. However, if moderate, serious or extreme levels of seismic risk exist, the extra steel volume demanded by seismic loads can be up to 1%, 25% and 45% for broad tanks, respectively. For slender tanks, these values will increase further up to 40%, 60% and 100%, respectively. Therefore, in high seismic intensity zones, choosing a smaller height and lower aspect ratio for specific seismic circumstances can significantly reduce the required material and related construction cost. An economical balance should be struck between the higher cost of the land required and the lower seismically demanded material for storing a specific volume of liquid.
7) Regardless of tank geometry, the steel volume solely demanded by seismic loads for a tank built on soft soil (type D) is up to 43% more than for one placed on rocky soil (types A and B). Therefore, employing various methods for enhancing the soil strength or using the different types of tank foundations to tackle the softness of soil can be well compensated for by the cost of extra material demanded by earthquake loads in high seismic intensity zones.

A useful empirical relation that covers all practical situations is proposed herein based on the results of analysis for preliminary estimation of the extra material demanded by earthquake loading under different various seismic conditions and for different tank geometries. The demanded steel volume provided in this study is based on the assumption of ductility factor of 1.25 for anchored tanks with non-ductile holding down bolts. It should be noticed that the steel volume predicted here may be slightly reduced if ductile behaviour is somehow expected.

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REFERENCES

1 Cooper TW (1997). "*A study of the performance of petroleum storage tanks during earthquakes, 1933-1995*". Report NIST GCR 97-720, US National Institute of Standards and Technology, 102 pp.

2 Pineda P, Saragoni GR and Arze E (2012). "Performance of steel tanks in Chile 2010 and 1985 earthquakes". *Proceedings of the 7th International Conference on Behavior of Steel Structures in Seismic Areas* (Stessa'12), 9-11 January, Santiago, Chile, S09b-3, 337-342.

3 Erica F, Liu C and Varma AH (2016). "Investigation of Cylindrical Steel Tank Damage at Wineries during Earthquakes: Lessons Learned and Mitigation Opportunities". *Practice Periodical on Structural Design and Construction*, **21**(3): 04016004-1-11.

4 Eshghi S and Razzaghi MS (2007). "Performance of cylindrical liquid storage tanks in Silakhor, Iran earthquake of March 31, 2006". *Bulletin of the New Zealand Society for Earthquake Engineering*, **40**(4): 173-182.

5 Erick G, Almazán J, Beltrán J, Herrera R and Sandoval V (2013). "Performance of stainless steel winery tanks during the 02/27/2010 Maule Earthquake". *Engineering Structures* **56**: 1402-1418.

6 Brunesi E, Nascimbene R, Pagani M and Beilic D (2014). "Seismic performance of storage steel tanks during the May 2012 Emilia, Italy, earthquakes". *Journal of Performance of Constructed Facilities*, **29**(5): 04014137-1-9.

7 Teng JG and Rotter JM (2004). "*Buckling of thin Metal Shells*". CRC Press, ISBN-13: 978- 0419241904, 520 pp.

8 Niwa A and Clough RW (1982). "Buckling of cylindrical liquid-storage tanks under earthquake loading". *Earthquake Engineering & Structural Dynamics*, **10**(1): 107-122.

9 Jia Z and Ketter RL (1989). "*Experimental Study of elephant Foot Bulge Instability of Thin-walled Metal Tanks*". Technical Report NCEER-89-0004, National Center for Earthquake Engineering, State university of New York at Buffalo.

10 Rotter JM (1985). "*Local Inelastic Collapse of Pressurized Thin Cylindrical Steel Shells under Axial Compression*". Research Report No. R502, School of Civil and Mining Engineering, University of Sydney, Australia, 26pp.

11 Rotter JM (1987). "Bending theory of shells for bins and silos". *Transactions of Mechanical Engineering, Institution of Engineers*, Australia, **12**(3): 147-159.

12 Rotter JM and Seide P (1987). "On the design of unstiffened cylindrical shells subject to axial load and internal pressure". *Stability of Plate and Shell Structures, Proceedings of An International Colloquium,* Ghent, Belgium, 6-8 April 1987.

13 Rotter JM (1990). "Local collapse of axially compressed pressurized thin steel cylinders". *Journal of Structural Engineering, ASCE*, **116**(7): 1955-1970.

14 El-Bkaily M and Peek R (1998). "Plastic buckling of unanchored roofed tanks under dynamic loads". *Journal of engineering mechanics,* **124**(6): 648-657.

15 Jaiswal OR, Durgesh CR and Sudhir KJ (2007). "Review of seismic codes on liquid containing tanks". *Earthquake Spectra*, **23**(1): 239–260.

16 Chen Z, Sun B, Yu C, Fang Z and Zeng M (2009). "Comparison of the strength design and prevention method of elephant foot buckling among countries' standards of oil tanks". *Proceedings of the ASME 2009 Pressure Vessels and Piping Division Conference* (PVP2009), Prague, Czech Republic, July 26-30, 2009, Vol. 1, 461–466.

17 Virella C, Godoy LA and Suárez LE (2006). "Dynamic buckling of anchored steel tanks subjected to horizontal earthquake excitation". *Journal of Constructional Steel Research*, **62**(6): 521-53 pp.

18 Chonghou Z, Yansheng L and Goto Y (2008). "Plastic buckling of cylindrical shells under transverse loading". *Tsinghua Science & Technology*, **13**(2), 202–210.

19 Chen Z, Yan S, Yu C, Cao G, Wang L and Yang L (2011). "Effect of opening on elephant's foot buckling of large oil tanks". *Advanced Materials Research*, **163**(2011): 169-172.

20 Yang L, Chen Z, Cao G, Yu C and Guo W (2013). "An analytical formula for elastic-plastic instability of large oil storage tanks." *International Journal of Pressure Vessels and Piping*, **101**(2013): 72-80.

21 Elkashef M, Rashed A and Haroun M (2014). "A study on the elephant-foot bulge formation of anchored steel tanks". *Australian Journal of Structural Engineering,* **15**(3): 321-335.

22 Zingoni A (2015). "Liquid-containment shells of revolution: a review of recent studies on strength, stability and dynamics". *Thin-Walled Structures,* **87**(2015): 102-114.

23 Godoy LA (2016). "Buckling of vertical oil storage steel tanks: Review of static buckling studies". *Thin-Walled Structures*, **103**(2016): 1-21.

24 Yazdani M, Rahimi H, Khatibi AA and Hamzeh S (2009). "An experimental investigation into the buckling of GFRP stiffened shells under axial loading". *Scientific Research and Essays*, **4**(9): 914-920.

25 Vakili M and Showkati H (2015). "Experimental and Numerical Investigation of Elephant Foot Buckling and Retrofitting of Cylindrical Shells by FRP". *The Journal of Composites for Construction*, **20**(4): 04015087-1-9.

26 Colombo I and Almazán JL (2015). "Seismic reliability of continuously supported steel wine storage tanks retrofitted with energy dissipation devices". *Engineering Structures*, **98**(2015): 201-211.

27 Vathi M, Karamanos SA, Kapogiannis IA and Spiliopoulos KV (2015). "Performance Criteria for Liquid Storage Tanks and piping systems subjected to seismic loading". *Journal of Pressure Vessel Technology*, **139**(5): 051801.

28 Paolacci F, Phan HN, Corritore D, Alessandri S, Bursi OS and Reza MS (2015). "Seismic fragility analysis of steel storage tanks". *Proceedings of the 5th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering* (COMPDYN'15), Crete Island, Greece, 25–27 May 2015.

29 Bakalis K, Vamvatsikos D and Fragiadakis M (2015). "Seismic Fragility Assessment of Steel Liquid Storage Tanks". *Proceedings of ASME 2015 Pressure Vessels and Piping Conference* (PVP), Boston, Massachusetts, USA. July 19-23, 2015, PVP2015-45370.

30 Maekawa A (2012). "*Recent Advances in Seismic Response Analysis of Cylindrical Liquid Storage Tanks*". *in Earthquake-Resistant Structures-Design, Assessment and Rehabilitation, Chapter 12*, InTech, 307-336.

31 Standard, British (1998). "*EN 1998-4, Eurocode 8: Design Provisions of Earthquake Resistance of Structures, Part 4: Silos, Tanks and Pipelines*". European Committee for Standardization, Brussels, 83 pp.

32 Standards New Zealand (1986). "*Seismic design of storage tanks, Recommendations of a NZSEE Study Group on Seismic Design of Storage Tanks*". (Red Book), Standards New Zealand, Wellington, 180 pp.

33 Standards New Zealand (2009). "*Seismic design of storage tanks, Recommendations of a NZSEE Study Group on Seismic Design of Storage Tanks*". (Blue Book), Standards New Zealand, Wellington, 177 pp.

34 American Petroleum Institute (2013). "*Welded Steel Tanks for Oil Storage".* API 650 Twelfth Edition, USA, 514pp.

35 Rotter JM (2006). "Elephant's foot buckling in pressurised cylindrical shells". © *Ernst & Sohn Verlag für Architektur und technische Wissenschaften GmbH & Co. KG, Berlin · Stahlbau*, **75**: 742-747.

36 Petrowiki (2016), http://petrowiki.org/Oil_storage, (Accessed 17/9/2016).

37 Wozniak RS (1990). *"Steel Tanks in Structural Engineering Handbook"*. Gaylord, EH and Gaylord, Eds., 3rd Edition, McGraw-Hill, NY, USA, 27-1:27-29.

38 Standards New Zealand (2004). "*NZS1170.5: Structural Design Actions. Part 5: Earthquake Actions-New Zealand*". Standards New Zealand, Wellington, New Zealand, 76 pp.