PROPOSAL OF SEISMIC COEFFICIENT AND ESTIMATE FOR HORIZONTAL SLIDING FOR STEEL TANKS BY BACKWARD SEISMIC ANALYSIS (BSA) METHOD

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Abstract. Large steel tanks for oil storage, petrochemical industry and process plants have frequently suffered repeated damages during largest earthquakes observed in the world and Chile 1960-1985-2007 and 2010 also in Alaska 1964 and other cities of United States (1933-1995). In most cases when tanks are anchored, these have had a good structural response with repairable damages to return to operation in reasonable periods of time, confirming that effective use of anchors helps prevent buckling "elephant foot" or "horizontal sliding". Based on the observation of real behavior of steel tanks in large earthquakes, the proposed methodology it is aimed at reducing the damage to the tanks and their structural stability, through seismic coefficient for tanks with slenderness ratios for safe design ranges with imperfections in the shell and estimates of horizontal coseismic sliding of unanchored tanks in subduction zones. The proposed methodology is based on the BSA method developed by the authors in previous works (Pineda & Saragoni) considering the Chilean high seismicity, proving that it does not exist direct correlation between the theoretical models with the observed, since the characteristic non-vibratory inertial effect of mega-subduction earthquakes simultaneously with high ground accelerations is not represented in the design codes. The analyses considering the seismic behavior of 382 tanks in operation during the last 80 years, which were mainly designed by different editions of the API650 standard with the appendix E. This proposal is essential to modify the main design codes for steel tanks.

Keywords: Tanks, Anchored, Backward, Subduction, Coefficient.

1 Introduction

This paper presents the seismic coefficient proposed based on Backward Seismic Analysis (BSA) method for steel tanks using information registered from 382 tanks in operation during major subductive earthquakes: Valdivia 1960, Chile Central 1985, Tocopilla 2007, El Maule 2010, in addition to Alaska (1964) and others occurred in the United States between 1933 and 1995 (subductive and cortical), being the only work available with this categorization and results that define the origin of the damage and mitigation measures. Information on tanks located in areas of high seismicity has been incorporated, with their geometric characteristics, design codes and fill levels at the time of the earthquake, being relevant for the evaluation of seismic behavior and determination of the causes of damage, in addition to the safe ranges in their dimensions and measures necessary in the design, especially in anchoring systems, which have been shown to be conditioning factors in the structural stability of tanks. There is strong evidence that during large earthquakes the non-anchored tanks present recurrent failures, which were mostly designed with the API650-E [1] standard, which indicates that its methodology should be reviewed and modified, since it presents limitations by not include relevant seismic aspects. It is necessary to review and modify the design criteria of the main design criteria of API650-E of the AWWA-D100 [2] and NZSEE [3] codes because they contain similar methodologies to estimate the seismic solicitations. Given the special conditions of the tanks with respect to their configuration of service loads with liquid contents, difficult to predict in their seismic response, it requires a special treatment considering different filling levels and analyzing different slenderness's, in addition to imperfections in the shell. It has been observed in recent earthquakes (Chile, Illapel 2015) that the vibrating frequencies of the shell-liquid system are in very distant ranges, with high periods (convective or sloshing mode) that can exceed 10 seconds in large diameter tanks and periods less than 1 second in the confined liquid in the lower area of the tank. At conferences of STESSA 2012 [4], STESSA 2015 [5], 16WCEE [6], 17WCEE [7] and the work of thesis Master in Seismic Engineering of the author [8], the causes of the failures were investigated concluding that mainly because the tanks designed with API 650-E were not anchored.

2 Subductive Earthquakes in BSA Analysis

In this work, the behavior of the steel tanks was analyzed during the main recorded interplate subductive earthquakes ([4], [5], [6], [7] and [8]), generated by the continuous sliding between the Nazca and South American oceanic plates, restricted in the contact areas known as asperities of the plates, which release large amounts of seismic energy when earthquakes occur. The displacement between tectonic plates has generated important horizontal sliding of the non-anchored tanks in the direction perpendicular to the coast in the direction of the convergence of the subducted continental plate.

3 Backward Seismic Analysis Method

The BSA method ([7] and [8]) has been applied in this work to evaluate the seismic behavior of steel tanks, showing that the real conditions at the time of the earthquake must be considered, the recommendations of the design codes being insufficient. For the application of this method, the following should be considered: geometry, plate thicknesses, fill height during the earthquake, types of soil foundations, design codes used, seismic records, seismic directivity, observed damage, buckling shell and collapses. The theoretical models of the main design codes are based on the Housner method, together with the experimental models (shaking tables) do not reflect the actual behavior of tanks during earthquakes because they do not meet the following hypotheses: thin shell effect, behavior of the liquid (laws of similarity), imperfections in the shell plates reducing the admissible stresses in the shell, real conditions of the foundation soil, soil-structure-liquid effect, and seismic directivity.

4 Results with Backward Seismic Analysis

The results of the main important earthquakes recorded in process plants in the world have been analyzed, with the objective of evaluating the structural behavior of the tanks in relation to their seismic coefficients. Figure 1 corresponds to the tanks located in the Con Con refinery during the Chile subduction earthquake in 1985. In this case, all the tanks were designed with the API650-E standard, eight tanks presented buckling shell type “elephant foot”, three tanks showed slight buckling shell and only one was undamaged. The seismic coefficients based on BSA method are scattered and in some cases conservative with respect to specified by the design standards and since none of the tanks was anchored, it is concluded that this is a relevant factor in the poor seismic response. Since the tanks were designed by API650-E, it is necessary to review the recommendations of this design standard. In other works, presented by the authors ([7] and [8]) it is confirmed that the API 650 standard underestimates the seismic design demand with apparently efficient theoretical values, this in addition to not using anchors resulted in failure in all tanks.

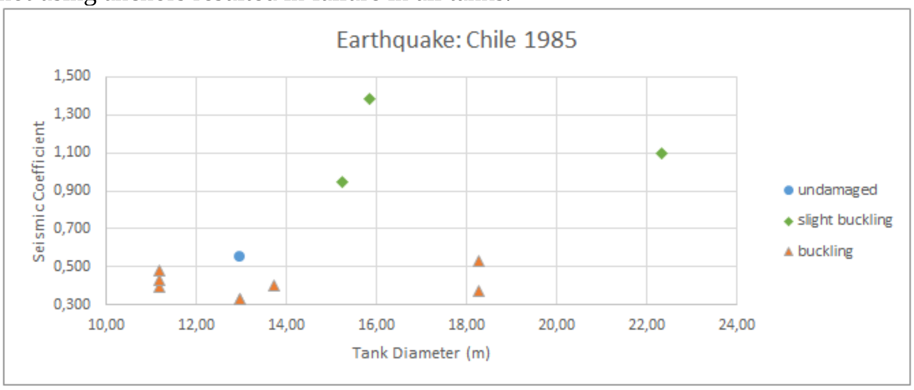

Fig. 1. Tank shell performance in the Chile Central 1985 earthquake, central zone.

Figure 2 shows the tanks located in different process plants (Interacid, CMPC Santa Fe Project, Alto Norte, Codelco-Salvador/Caletones/San Antonio, Arauco II Cellulose) during the Chilean subduction earthquake in 2007. In this case, all the tanks had good seismic behavior, since they were anchored at the time of the earthquake, except for one that had no anchors and presented buckling in the upper part of the shell.

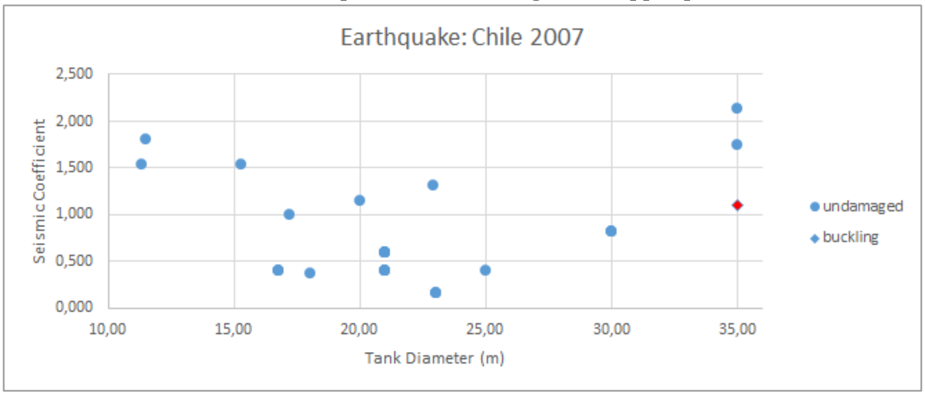

Fig. 2. Tank shell performance in the Tocopilla 2007 Earthquake.

Figure 3 contains a series of tanks located in different process plants (Los Lirios Plant, Santiago International Airport, San Vicente International Terminal, ENAP, CAP, Angamos Power Plant, Ventanas Power Plant, ENAMI, Codelco-Gaby/Molybdenum/San Antonio, Alto Norte, Terquim1-Reception, Terquim2-Storage, Arauco II Cellulose) during the Chilean subduction earthquake in 2010. Cases are presented with anchored and non-anchored tanks, all with good seismic behavior, but it is important to note that the non-anchored ones have diameters that exceed the anchoring recommendations according to API650-E since they are seismically stable. The case of the only tank (Santiago International Airport) that presented collapse is since it was not anchored and had insufficient shell thickness according to the seismic design recommendations of the usual design standards.

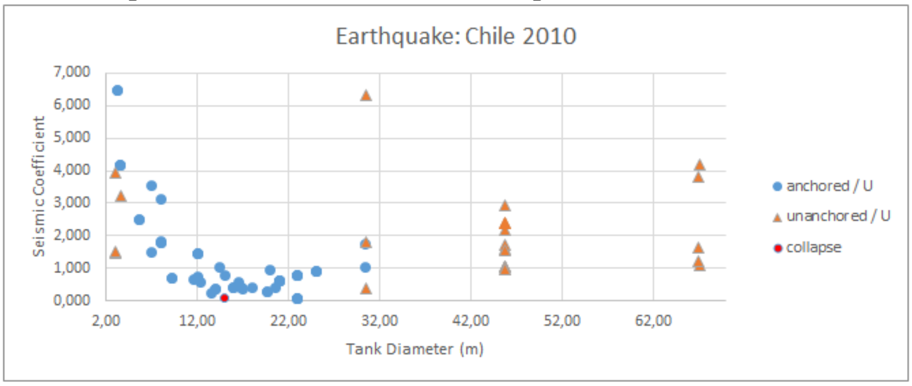

Fig. 3. Tank shell performance in the El Maule 2010 Earthquake.

Figure 4 shows the tanks located in the Lumina-Caserones mine during the 2015 Illapel subduction earthquake. Tanks of different diameters are observed, and all were anchored, of which none had damage in shell, which confirms the need for incorporate anchors systems.

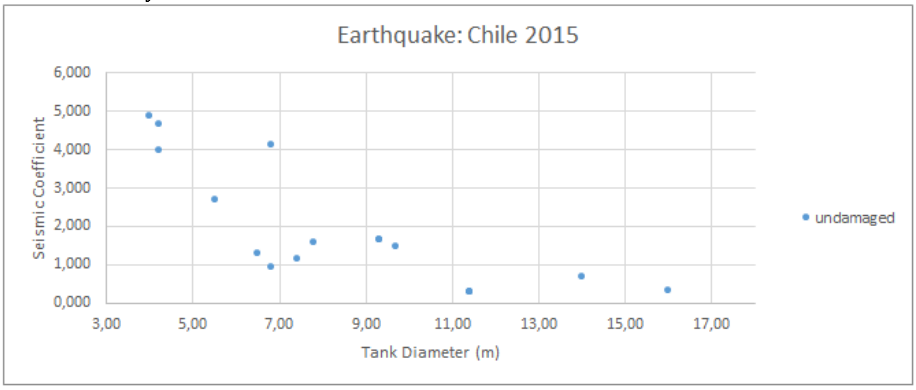

Fig. 4. Tank shell performance in the Illapel 2015 Earthquake.

Figure 5 shows the tanks located in different cities of Alaska during the 1964 subduction earthquake. For decades, the equation proposed by Rinne [11] has been used considering only the dimensions of the tanks to classify them as "safe", but the present analyzes conclude that was insufficient, since it does not ensure the stability of the tanks by itself, being necessary to incorporate anchors systems.

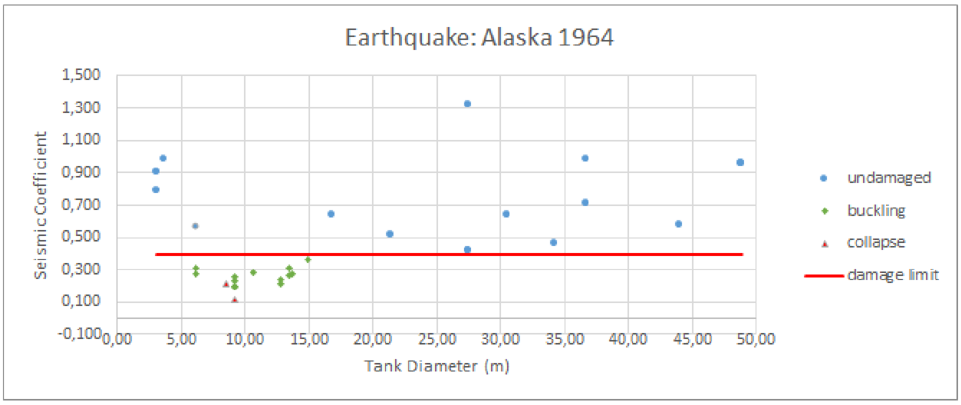

Fig. 5. Tank shell performance in the Alaska 1964 Earthquake.

Figure 6 shows a series of tanks registered in the main earthquakes that occurred in the USA (1933-1995). Similar diameters are observed with different seismic coefficients without faults and with buckling shell. Some cases with anchors failed because they had insufficient shell thicknesses, the rest of the cases presented failures and were not anchored in addition to thicknesses under what is established by current design standards. It concludes that the use of anchors with thicknesses according to the design codes are necessary to avoid structural damage.

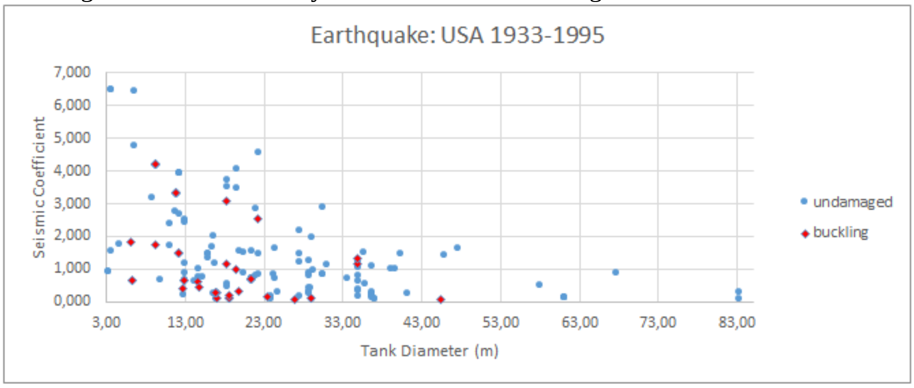

Fig. 6. Tank shell performance in the United States of America between 1933-1995 Earthquake.

Taking the Rinne [11] equation as a reference, in this work Eq. (1) is proposed for the calculation of the seismic coefficient, considering the allowable compression with imperfections of the shell plates proposed by Saragoni [12] and dense gravel-type soil, for lower quality soils this coefficient should be increased according to the project site studies:

$$C_{ps} := \frac{\frac{\pi}{4} \cdot t_s \cdot (D_e)^2 \cdot F_c}{M_t} \cdot \frac{V_s}{M_s} \quad (1)$$

$$F_c = 0.908\ \alpha\ E\ t_s/D_e < 0.375\ F_y \quad (2)$$

$$\alpha = 0.83/\sqrt{(1+0.005\ D_e/t)} \quad , \quad \text{if} \quad D_e/t < 424 \quad (3)$$

$$\alpha = 0.70/\sqrt{(0.1+0.005\ D_e/t)} \quad , \quad \text{if} \quad D_e/t \geq 424 \quad (4)$$

When D_e/t is less than $2.422 \propto E/F_y$, it is assumed that the failure corresponds to an inelastic buckling. To prevent this occurrence, the allowable stress F_c will be evaluated assuming a linear variation of said stress between $0.375F_y$ and 0.60Fy for D_e/t within the range 0 to $2.422{\propto}E/F_y$. For the range of D_e/t between 400 and 2000, the following approximate formula may be used:

$$F_c = 0.20\ E\ t_s/D_e < 0.375\ F_y \quad (5)$$

A linear variation of the allowable work rate F_c is assumed, between 0.6Fy and $0.375F_y$ for the inelastic range. D_e/t varies between 0 and $0.534E/F_y$.

$M_t = m_i + m_s + m_r + m_c$
m_i : impulsive mass (kg)

m_s : shell mass (kg)
m_r : roof mass (kg)
m_c : convective mass (kg)
M_t : total mass (kg)
t_s : shell thickness (m)
D_e : diameter tank (m)
V_s : total design base shear (kg)
M_s : overturning moment (kg-m)
E : elastic modulus of tank material (kg/m^2)
F_y : minimum specified yield strength of tank material (kg/m^2)
F_c : allowable longitudinal shell-membrane compression stress (kg/m^2)

5 Horizontal Sliding for Unanchored Tanks

Non-anchored tanks present a high risk of having horizontal sliding during subduction earthquakes, because of the inertial forces of the masses of the tank-liquid system. In other works, the authors ([7] and [8]) it has been shown that the sliding is also due to coseismic tectonic displacements measured in meters using GPS. The instrumental spectral effects refer to vibratory movements, while the values obtained from Eq. (6) refer to the inertial behavior in the coseismic direction. Table 1 shows the horizontal slides of the tanks observed in some earthquakes ([5], [6], [7] and [8]), which have been used to propose Eq. (6) for estimated values of coseismic slip only for earthquakes of subduction.

Table 1 – Horizontal Sliding Observed in Tanks ([5], [6]).

Earthquake	Magnitude	Tectonic	S_h (mm)	D (mm)	H (mm)
Alaska 1964	9.2	Subduction	1524	3200	9144
Tocopilla 2007	7.7	Subduction	10	35000	14500
Landers 1992	7.3	Cortical	80	16500	7300

$$S[m] = -7.76 + 1.01M \quad ; \quad M \geq 7.7 \qquad (6)$$

S[m]: coseismic sliding in meters ; M: moment magnitude.

The proposed equation must be updated with new records of tanks that have presented horizontal sliding, for which it is necessary to have instrumentation at the base of the tanks.

6 Recommendations and Conclusions

To avoid the horizontal sliding of the tanks, the use of anchoring systems in tanks is recommended, together with having thicknesses of the shell plates according to the design standards of each country. This, together with the use of the seismic coefficient proposed in this work, reduces the risk of failures due to buckling shell and horizontal sliding. The tanks analyzed show similar seismic coefficients and different seismic behaviors, without damage (anchored) and with buckling shell (not anchored), this confirms that the use of a conservative seismic coefficient is not enough to ensure the structural stability of a tank if it is unanchored. The use of anchors has shown that in most cases it is mandatory in the design, so all design codes should incorporate it as a recommendation.

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