SEISMIC RESPONSE AND DAMAGE EVALUATION FOR ANCHORED AND UNANCHORED CYLINDRICAL ABOVE GROUND STEEL TANKS

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Abstract - This paper aims to first present a review of studies on numerical modeling and seismic response analysis of above ground steel liquid storage tanks. On that basis, a procedure for estimating dynamic parameters associated with simplified models for anchored and unanchored conditions together with calculation methods of seismic responses and damage states of tanks are presented. In which, the nonlinear behavior of the bottom plate in the case of unanchored conditions caused by sliding and uplift phenomena is properly modeled based on the nonlinear static pushover analysis on a 3D finite element model. Finally, an example of the numerical modeling and seismic response analysis of a water tank is presented. The seismic responses and damage of both anchored and unanchored conditions are compared and evaluated in detail.

Key words - Steel liquid storage tanks; seismic response; spring-mass model; tank-liquid interaction; failure mode

1. Introduction

Above ground steel liquid storage tanks have been commonly constructed in industrial plants, especially petrochemical plants for the storage of chemical substances. Past earthquake damage in industrial zones revealed that storage tanks are often severely damaged resulting in the release of toxic and inflammable substances, which could spread damage to the surrounding area [1, 2].

Studies on the seismic response of storage tanks have been concentrated since the 50s of the 20th century. The earliest study was by Jacobsen [3], who analyzed hydrodynamic pressures on rigid tanks with an anchored support condition subjected to horizontal motion. In his work, the motion of an incompressible fluid is represented by the Laplace equation. Housner [4] used an approximate simplification method in which the total hydrodynamic pressure is decomposed into convective and impulsive parts. Veletsos and Yang [5] used an alternative approach to develop a similar mechanical model for rigid circular tanks. They found that the pressure distribution due to fluid movement for rigid and flexible anchored tanks was similar; however, the magnitude is highly dependent on the wall flexibility. Haroun and Housner [6] developed a reliable method to analyze the dynamic behavior of deformable cylindrical tanks, based on a finite element model of a fluid-tank system. Veletsos [7] improved Housner's mechanical analog to account for the effect of the flexibility of the shell plate. Furthermore, the dynamic response of a cylindrical tank subjected to the base motion was analyzed by Veletsos and Tang [8]. Fische and Rammerstorfer [9] presented an analytical procedure that allows one to unambiguously investigate the effect of wall deformations on both liquid pressure and surface elevation for typical wall deformation shapes. Malhotra et al. [10] simplified Veletsos' flexible tank model; the procedure was later adopted in Eurocode 8 [11].

For practical and economic reasons, many liquid storage tanks have been built directly on compacted soil without anchoring. The behavior of unanchored tanks is significantly different from that of anchored tanks. Malhotra and Veletsos [12, 13] investigated the uplift behavior of the bottom plate of unanchored tanks, where the bottom plate is idealized as semi-infinite prismatic beams on a rigid foundation subjected to a uniform load.

Since the finite element method (FEM) has become a useful tool and widely adopted in many fields of engineering; it can be applied to numerically analyze the tank-liquid system and their interaction. However, due to the complex nonlinear behavior of liquid storage tanks, modeling this system is a very challenging task. Barton and Parker [14] first studied the seismic response of liquid-filled cylindrical tanks using the FEM implemented in ANSYS software. Both the concepts of added mass and fluid finite elements are used to consider hydrodynamic effects. Virella et al. [15] presented buckling analyses of anchored steel tanks subjected to horizontal seismic excitations using nonlinear three-dimensional finite element models. An additional mass is attached to the nodes of the shell element by spring elements. Ozdemir et al. [16] presented a nonlinear fluid-structure interaction method for seismic analysis of anchored and unanchored tanks. In their models, the Arbitrary Lagrangian-Eulerian (ALE) method is adopted to model the fluid-structure interface, and the fluid motion is governed by the Navier-Stokes equations. Recently, a nonlinear static pushover analysis of unanchored steel liquid tanks was proposed by Vathi and Karamanos [17], where the distribution of hydrodynamic pressures on the shell plate is calculated and applied to the steel tank model by a loading subroutine in ABAQUS software. Phan et al [18, 19] proposed full nonlinear finite element models of an unanchored tank using ABAQUS software, using both Arbitrary Lagrangian-Eulerian and Structural Acoustic Simulation methods. The results of their analyses are in good agreement with the experimental data and demonstrated the suitability of both models. The above-mentioned models are basically based on a full finite element model of the tank-liquid system. Although they can provide accurate simulation results but will consume more computational cost, especially in the case of probabilistic and reliability analyses.

This paper focuses on the numerical modeling approach, seismic response, and damage analyses of cylindrical above ground steel liquid storage tanks. In this regard, possible numerical modeling approaches for anchored and unanchored steel liquid storage tanks are first presented. Attention is paid to the simplified model of the tank-liquid system, which is suitable for probabilistic and reliability analyses. While the model for anchored tanks is based on the proposal of Malhotra et al. [10] and Eurocode 8 [11], an enhanced model is proposed for unanchored tanks. This model is improved based on the model of Malhotra and Velesos [13], in which the overturning moment-rotation relationship of the bottom plate is determined precisely from the nonlinear static analysis of the 3D finite element model. Based on the analysis for a specific cylindrical steel tank, different seismic responses of the tank with and without anchorage are presented. Accordingly, limit states for failure modes are also calculated and evaluated with the obtained seismic responses.

2. Numerical model of above ground tanks

2.1. Anchored tank model

A possible numerical model for the anchored tank represented by two viscoelastic oscillators is shown in Figure 1, where the impulsive and convective masses (m_i and m_c) are lumped on cantilever tips with stiffness (k_i and k_c) and damping coefficients (c_i and c_c). For each cantilever, the calculations of mass, length, and natural period can be obtained by the simplified method of Malhotra et al. [10]. Considering a ground motion, the impulsive and convective responses are calculated independently and can be combined using the absolute-sum rule. This procedure has also been adopted in Eurocode 8 [11].

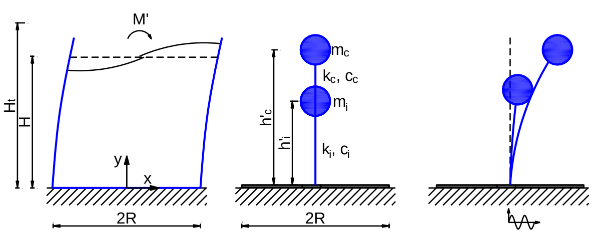


Figure 1. *Spring-mass model for the anchored tank*

The natural periods of impulsive and convective vibrations (T_i and T_c) are calculated as

$$T_i = C_i \frac{H\sqrt{\rho}}{\sqrt{t_{eq}/R} \times \sqrt{E}}, \quad (1)$$

$$T_c = C_c \sqrt{R}, \quad (2)$$

where H is the height of the liquid, ρ is the liquid density, t_{eq} is the equivalent thickness of the shell plate, E is the modulus of elasticity of the steel tank, and C_i and C_c are the coefficients which can be obtained from Malhotra et al. [10].

The corresponding stiffness and damping coefficient of each response are:

$$k_i = \omega_i^2 m_c \text{ and } c_i = 2\xi_i m_i \omega_i \quad \text{with } \omega_i = 2\pi/T_i \quad (3)$$

$$k_c = \omega_c^2 m_c \text{ and } c_c = 2\xi_c m_c \omega_c \quad \text{with } \omega_c = 2\pi/T_c \quad (4)$$

where ω_i and ω_c are the angular frequencies of the impulsive and convective vibrations, respectively.

Since the acceleration responses of the impulsive and convective components are obtained, they can be combined by taking the numerical sum, and the total base shear, the moments above and below the bottom plate are given as

$$Q = (m_i + m_w + m_r) \times A_i + m_c A_c \quad (5)$$

$$M = (m_i h_i + m_w h_w + m_r h_r) \times A_i + m_c h_c A_c \quad (6)$$

$$M' = (m_i h_i' + m_w h_w + m_r h_r) \times A_i + m_c h_c' A_c, \quad (7)$$

where m_w, m_r are the shell plate and roof masses, $h_i(h_i')$ and $h_c(h_c')$ are the heights of the impulsive and convective hydrodynamic pressure centroids, h_w and h_r are the heights of the shell plate and roof gravity centers, A_i and are A_c are the impulsive and convective acceleration responses.

2.2. Unanchored tank model

In many cases, tanks can be constructed without anchorages, namely unanchored or self-anchored tanks. when these tanks are subjected to strong seismic excitations, the partial uplift and sliding of the bottom plate occur. Hence, the seismic response of the tanks is highly influenced by these phenomena.

A simplified model of unanchored tanks was proposed by Malhotra and Veletsos [13]. The uplift mechanism of the tanks is simulated by a rotation spring that represents the rocking resistance of the base, as shown in Figure 2. In this model, the masses of the shell plate, m_w, and tank roof, m_r, are lumped with the impulsive mass. The total impulsive mass, $m = m_i + m_w + m_r$, is lumped on the cantilever tip with the equivalent length, $h' = (m_i h_i' + m_w h_w + m_r h_r)/m$, the stiffness, $k = \omega_i^2 m$, and the damping coefficient, $c = 2\xi_i m \omega_i$.

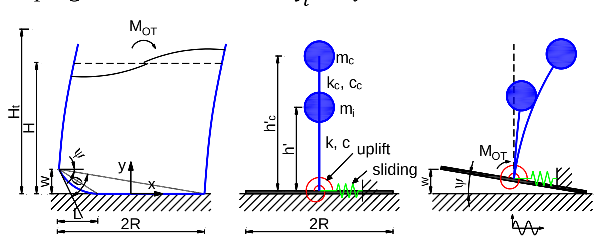


Figure 2. *Spring-mass model for the unanchored tank*

To accurately obtain the rotation spring behavior (M_{OT}- ψ nonlinear relationship) for the uplift model and the friction behavior for the sliding model, a static pushover analysis procedure for the tank system is presented in this study. The analysis is based on a three-dimensional finite element model of the steel tank using the ABAQUS software, where both geometric and material nonlinearities are considered [19]. For example, Figure 3(a) shows the finite element modeling of an unanchored tank, where the shell and bottom plates are modeled using shell elements, while solid elements are used to model the base slab. Due to the geometric symmetry, only half of the tank is modeled.

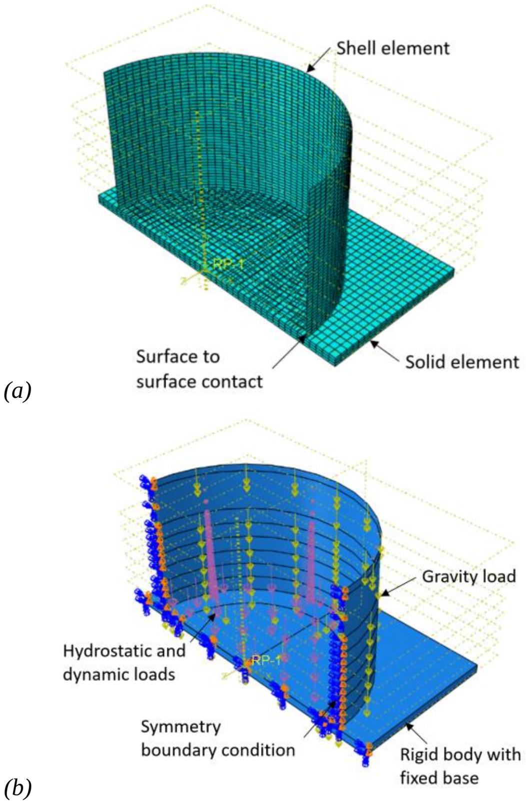


Figure 3. *An example of the finite element modeling of an unanchored tank: (a) finite element meshes and (b) boundary conditions and load cases*

The steel tank is subjected to a static pushover loading that includes the gravity, hydrostatic and hydrodynamic pressures acting on the shell and bottom plates. The hydrodynamic load is calculated using the formula in Eurocode 8 [11] and applied as a distributed surface load (i.e., pressure) to the shell and bottom plates, as shown in Figure 3(b), using the DLOAD subroutine.

3. Seismic response and limit state calculations

3.1. Seismic response calculations

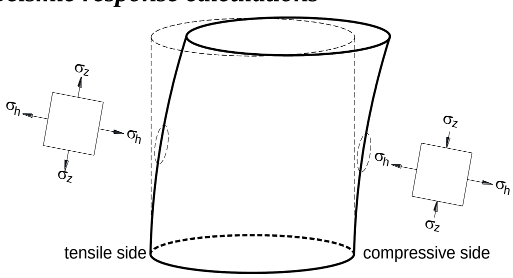


Figure 4. *Tensile hoop and meridional stresses in the shell plate*

The critical responses of above ground tanks under the seismic load are the maximum hoop tensile and meridional stresses in the shell plate, the maximum sloshing of the free surface, and the rotation demand of the shell-to-bottom connection in the case of unanchored tanks.

The hoop hydrodynamic stresses, as described in Figure 4, are caused by impulsive and convective motions (denoted as σ_{hi} and σ_{hc}, respectively) and can be calculated based on explicit equations stated in API 650 [20]. The total hoop stress in the shell plate is the sum of the hydrostatic hoop stress (σ_{hs}) and the hoop hydrodynamic stresses, given as

$$\sigma_h = \sigma_{hs} + \sigma_{hi} + \sigma_{hc}. \tag{8}$$

For anchored tanks, the meridional stress, i.e., σ_z in Figure 4, is associated with the axial force, N, per unit circumferential length, given as

$$\sigma_z = \frac{N}{t_s}. \tag{9}$$

The axial forces per unit circumferential length on the compressive and tensile sides are given as

$$N = \mp \frac{1.273M}{D^2} - w_t, \tag{10}$$

where M is the moment above the bottom plate and w_t is the load per unit circumferential length caused by the shell and roof weight.

For unanchored tanks, the compressive axial stress in the shell plate can be evaluated using the Cambra's formula [21]. Given Q_1 is the reaction force at the right end when the bottom plate is rocking about that point, then the compressive axial stress is given as

$$\sigma_z = \frac{9}{\pi}\frac{Q_1}{Rt_s}. \tag{11}$$

The rotation demand of the shell-to-bottom connection associated with an uplift of w and an uplift length of L is given as (see Figure 2)

$$\theta = \left(\frac{2w}{L} - \frac{w}{2R}\right). \tag{12}$$

The maximum sloshing of the free surface is provided mainly by the first convective mode and is given as [20]

$$\eta_{\max} = 0.84RA_c/g. \tag{13}$$

3.2. Limit state calculations

It is important to first identify the critical failure modes of tanks. As observed from past earthquakes, the common failure modes include the shell plate buckling, material yielding under extreme hoop tensile stresses, anchor bolt failure (i.e., in the case of anchored tanks), roof damage due to sloshing and plastic rotation of the shell-to-bottom connection (i.e., in the case of unanchored tanks).

The buckling of shell courses near and above the base should be verified for two possible modes, i.e., elastic buckling (or diamond-shaped buckling) and elastic-plastic buckling (or elephant's foot buckling). The critical buckling stresses for elastic and elastic-plastic buckling can be calculated using the formulas developed by Rotter [22, 23]; these formulas are later adopted in Eurocode 8 [11], given as

$$\sigma_{eb} = \sigma_{c1}\left(0.19 + 0.81\frac{\sigma_p}{\sigma_{cl}}\right), \tag{14}$$

$$\sigma_{efb} = \sigma_{cl}\left[1 - \left(\frac{pR}{t_s f_y}\right)^2\right]\left(1 - \frac{1}{1.12 + \left(\frac{R}{400t_s}\right)^{1.5}}\right)\left(\frac{\frac{R}{400t_s} + \sigma_y/250}{\frac{R}{400t_s} + 1}\right), \tag{15}$$

where $\sigma_{cl} = 0.6E_t t_s/R$ is the ideal critical buckling stress, σ_p is the buckling stress increase caused by the internal pressure, p is the maximum interior pressure, and t_s is the thickness of the considered shell course.

The other common failure mode is the material yielding of the shell plate subjected to extreme hoop tensile stress.

As described in API 650 [20], the maximum allowable hoop tension stress can be calculated as the lesser of the basic allowable membrane of the shell plate increased by 33% and $0.9\sigma_y$.

In the case of anchored tanks, the performance of the anchor bolts should be investigated, which can be done through their maximum allowable stress. This value for the anchorage components does not exceed 80% of the minimum yield stress.

In the case of unanchored tanks, the rotation demand of the shell-to-bottom connection is less than the estimated rotation capacity of 0.2 rad, as mentioned in Eurocode 8 [11].

4. Seismic response and damage analysis of case study

4.1. Description of case study

In this section, a cylindrical above ground tank is presented as a case study. The tank geometry selected with a moderately-broad configuration, which can be considered for both anchored and unanchored conditions. The tank has a diameter of 27.77 m and a total height of 16.51 m. It is assumed to be filled with water with a density of 1000 kg/m^3 and the filling level is 15.7 m (about 95% of the total height).

Hence, the aspect ratio of the tank, $\gamma = H/R$, is given as 1.131. The shell plate thickness is ununiformed, which varies from 6.4 mm at the top course to 17.7 mm at the bottom course. By using the weighted average method, the equivalent shell plate thickness is calculated as 13.1 mm [10]. The bottom plate has a thickness of 8 mm, and the annular plate is neglected in this study. The structural steel S235 (equivalent to A36 steel) with yield stress σ_y = 235 Mpa is used for whole the tank.

4.2. Spring-mass model parameters

As presented in Section 2, the dynamic parameters of the simplified model for the tank are shown in Table 1. Both anchored and unanchored conditions of the tank are considered.

Table 1. *Parameters of the spring-mass model for the sample tank*

Parameter	Anchored	Unanchored
Impulsive mass, m_i (T)	5639	5639
Convective mass, m_c (T)	3870	3870
Equivalent mass, m (T)	-	6815
Impulsive natural period, T_i (s)	0.22	0.22
Convective natural period, T_c (s)	5.60	5.60
Impulsive mass height, h_i (m)	6.69	6.69
Impulsive mass height with base pressure, h_i' (m)	10.25	10.25
Convective mass height, h_c (m)	9.99	9.99
Convective mass height with base pressure, h'_c (m)	11.71	11.71
Equivalent height, h (m)	-	9.91

When the tank is unanchored, the uplift mechanism of the bottom plate is considered by a resisting spring. The behavior of the spring can be represented by the $M_{OT} - \psi$ relationship. This relationship can be obtained from the static pushover analysis on the 3D finite element model of the tank, as illustrated in Section 2.2. The von Mises stress and displacement contours of the tank with the base uplift at a A_g = 0.62 g obtained from the nonlinear static pushover analysis is shown in Figure 5. It can be seen that the tensile stress concentrates around the shell-to-bottom connection region and reaches the material yielding. In addition, due to the uplift, the right side of the tank is subjected to a high axial reaction force, resulting in a high meridional compressive stress on this side.

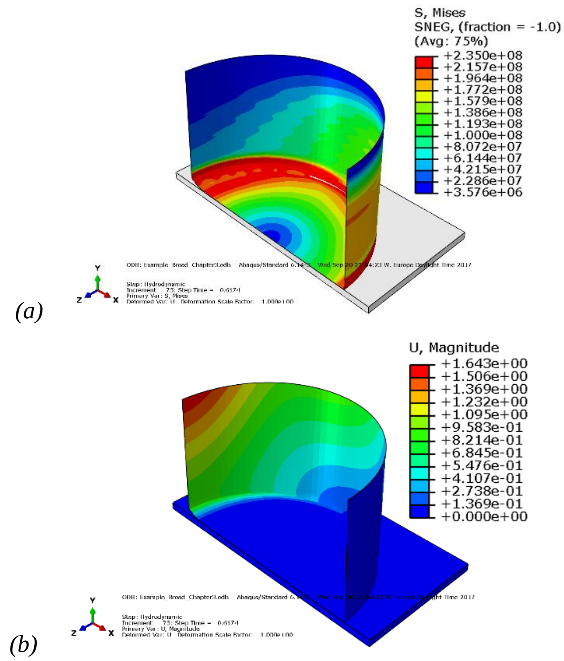


Figure 5. *(a) Contours of the von Mises stress and (b) the vertical displacement of the tank obtained at an acceleration of 0.62 g*

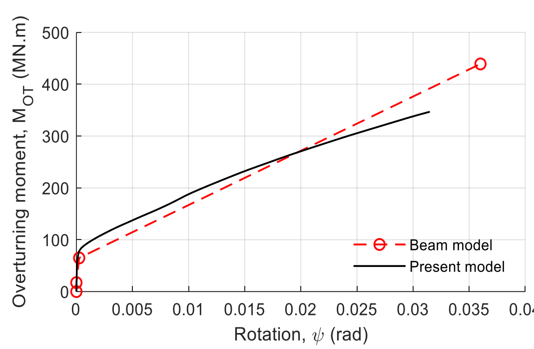


Figure 6. *Moment-rotation curve of the sample tank*

A comparison of the $M_{OT} - \psi$ relationship between the present model and the beam model by Malhotra and Veletsos [12] is shown in Figure 6. A quite good agreement between the two curves is observed, despite the discrepancy found in the post-yield zone. The curve obtained by the beam model seems to underestimate the response of the unanchored tank; however, for the very large deformation, i.e., $\psi > 0.02$ rad, the beam model curve is overestimated.

4.3. Seismic response and damage analyses

The simplified models of the anchored and unanchored conditions of the tank are analyzed dynamically using a time history accelerogram. In this example, a horizontal component of the ground motion recorded from the Duzce 1999 earthquake in Turkey is considered; the acceleration traces for which is shown in Figure 7, together with the elastic response spectrum with 5% damping shown in Figure 8.

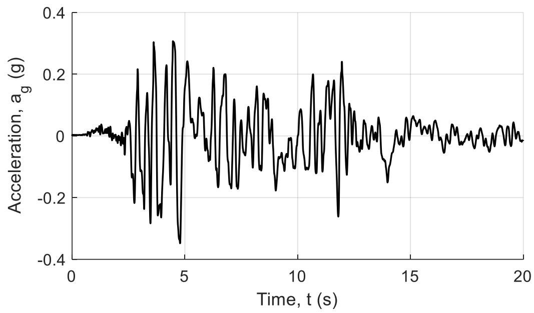


Figure 7. *Time history data of the accelerogram*

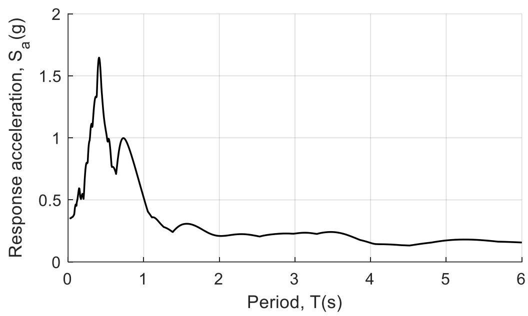
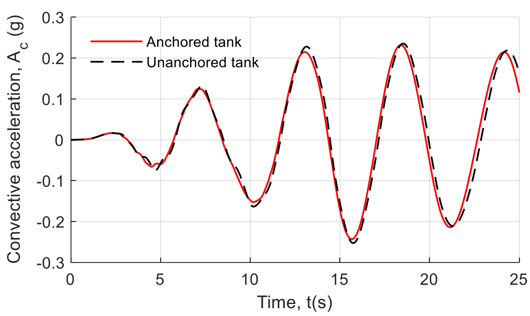
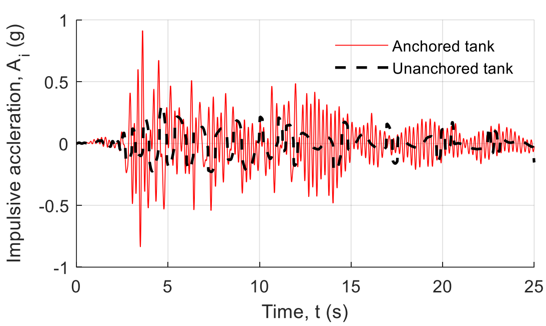


Figure 8. *5% damping elastic response spectrum*



(a)



(b)

Figure 9. *Time history of the acceleration for both anchored and unanchored conditions: (a) convective response and (b) impulsive response*

The response histories of the convective and impulsive components for both anchored and unanchored conditions of the tank are shown in Figure 9. It is observed that the convective responses for both cases are almost the same, as shown in Figure 9(a). Hence, the uplift may not affect the sloshing mode of the tank. For the impulsive response, as shown in Figure 9(b), the acceleration time history of the unanchored tank exhibits smaller amplitudes and longer periods of oscillation and shows nearly uniform amplitudes. This finding demonstrates the significant effect of the uplift on the impulsive pressure acting on the tank.

The time history responses of the uplift displacement at the two ends of the base in the unanchored condition are shown in Figure 10. The maximum base uplift is observed as about 0.3 m; this value is appropriate with the flexibility in the design of the piping system attached to the shell plate.

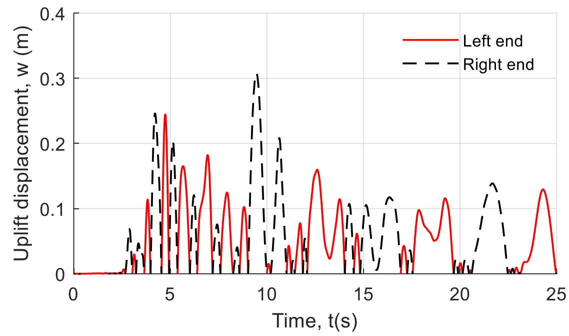


Figure 10. *Time history of the uplift displacement*

The critical responses of the tank for both conditions, including the maximum sloshing of the free surface, the hoop tensile stress in each shell course, the compressive meridional stress in the bottom shell course, and the plastic rotation of the shell-to-bottom connection, are calculated using the above formulas. Their peak responses are summarized in Table 2, together with their corresponding limit state capacities.

It can be seen that the base uplift may reduce the hydrodynamic pressures, in particular the impulsive component, resulting in lower tensile hoop stress in the shell plate in the case of the unanchored condition. Also of note is that this reduction may be associated with the increase of axial stresses in the shell plate and plastic rotations at the shell-to-bottom connection.

Table 2. *Peak value of the tank responses*

Response	Anchored	Unanchored	Limit state capacity
η_{max} (m)	2.83	2.95	0.8
σ_h (MPa) (course 1 - course 8)	212.4, 222.3, 232.5, 243.2, 254.5, 263.9, 252.4	155.1, 158.7, 162.7, 167.3, 173.1, 178.8, 172.7	209.3
σ_z (MPa) (course 1)	55.6	68.0	82.7
θ (rad)	-	0.535	0.2

For the damage assessment with the examined Duzce 1999 ground motion, the sloshing wave height exceeds the freeboard height of the tank, hence this can cause roof damage. In the case of the anchored condition, the hoop stress of the shell courses exceeds the limit state of the steel tank and may cause the fracture of the shell plate.

On the other hand, in the case of the unanchored condition, the axial compressive stress is slower than its limit state, and thus no buckling is observed. However, the plastic rotation of the shell-to-bottom connection is significant (i.e., larger than a limit of 0.2 rad), and this causes the fracture of the connection.

5. Conclusions

In this study, a comprehensive literature review on the seismic response analysis of steel liquid storage tanks was first presented. Possible numerical models were then presented for the evaluation of the response to horizontal ground shaking of above ground steel liquid storage tanks with and without anchorage conditions. The tank-liquid system is simplified as a cantilever beam model considering the most important parameters of the system. A more accurate procedure that is based on a nonlinear static pushover analysis and a proposed spring-mass model for unanchored tanks is presented. As shown from the seismic response and damage analysis of a sample tank for two anchorage conditions, it can be concluded that:

- The convective responses for both cases are almost the same, hence the uplift may not affect the sloshing mode of the tank.

- The base uplift increases the effective period of vibration of the unanchored system as compared to its fully anchored condition. This effect also reduces the impulsive hydrodynamic pressure and the associated overturning base moment, hence decreasing the effects of the tensile hoop hydrodynamic stress.

- When the tank is unanchored, a significant amount of base uplift and plastic yielding at the joint of the shell and bottom plates is exhibited. This increases the axial compressive stress on the shell plate.

- For the examined ground motion, the sloshing wave height exceeds the freeboard height of the tank, hence this can cause roof damage. In the case of the anchored condition, the hoop stress of the shell courses exceeds the limit state of the steel tank. On the other hand, in both cases, the axial compressive stress is slower than its limit state, and thus no buckling is observed. However, the plastic rotation of the shell-to-bottom connection in the unanchored tank is significant, this causes the fracture of the connection.

- The conclution in this study is limited for the case study. For different tank configuratations, a comprehensive analysis should be further conducted.

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