SEISMIC ANALYSIS CONSIDERATIONS FOR UPLIFTED STORAGE TANKS

Kangarloo K., Bashgah P. J.

Scientific adviser: Tryshin C.I.

State University of Civil Engineering (MSUCE), Moscow

Abstract

The objectives of this paper are to highlight the principal effects of base uplifting on the seismic response of ground supported cylindrical steel tanks that are unanchored at their Base. In practice, however, complete anchorage is not economical or may not be warranted for certain class of tanks; as a result, many existing tanks are unanchored and may uplift during ground shaking. Base uplifting completely changes the dynamic characteristics such as the stiffness and energy dissipation capacity of the system. The dynamic response of the system also becomes highly non-linear. Studies of the performance of uplifting tanks during past earthquakes have revealed that such systems are prone to extensive damage due to: (i) buckling of the tank wall, caused by large compressive stresses; (ii) rupture at the plate-shell junction, caused by excessive plastic yielding; and (iii) failure of the piping connections to the wall that are incapable of absorbing large base uplifts.

Keywords: Uplift, overturning, sloshing, cylindrical vertical steel tanks, seismic loading, simulation, elephant-foot, natural period, hydrodynamic pressure.

1-Introduction

Steel ground-based tanks consist essentially of a steel shell that resists the outward liquid pressure, a thin flat bottom plate that prevents the liquid from leaking out, and a thin roof plate that protects the content from the atmosphere. Circular vertical tanks are more numerous than any other type because they are efficient in resisting the liquid hydrostatic pressure by membrane stresses, simple in design, and easy in construction. It is common to classify such tanks in two categories depending on the support condition: anchored and unanchored tanks. It is common, particularly for large size tanks, to support the shell on a ring wall foundation without anchor bolts and to support the bottom plate on a compacted soil though, sometimes, ring walls are omitted.

For anchored tanks, the tank wall is effectively fixed to a foundation which is sufficiently heavy to prevent uplift in the event of an earthquake. This means that the anchor bolts must be able to transmit the earthquake induced vertical tension in the tank wall to the foundation. In practice, anchoring a tank requires a large number of anchor bolts and suitable attachments welded onto the tank wall, so that the tension forces in the anchor bolts can be distributed evenly in the tank wall. Poorly designed attachments, or an
attempt to carry too high a bolt force on a single attachment could result in tearing of the tank wall. Also, a fairly massive foundation may be required, especially for a larger tank. Thus, anchoring a tank is expensive, and, as a result, many tanks are unanchored, even in seismic areas. This is especially true for large capacity, broad tanks [1].

Evidence of uplift can be found in the 1964 Alaska earthquake, during which snow found its way underneath the base plate of some tanks [Hanson (1973)] and during the 1971 San Fernando earthquake, when an anchor bolt of a 30 ft tall and 100 ft diameter tank was pulled up by 14 in (Figure 1) [2].

Figure 1. base uplifting, (a) 1964 Alaska earthquake, (b,c) 1971 San Fernando earthquake

2- Axisymmetrically uplift problem

When an unanchored tank is subjected to strong ground shaking, overturning moment caused by the hydrodynamic pressure tends to lift shell off its foundation. Unless the tank wall uplifts, the overturning moment can only be balanced by the stabilizing effect of the weight of the tank. For typical steel tanks the weight of the tank is much less than the weight of the contained liquid. Therefore, the weight of the tank is insufficient to balance the overturning moment due to hydrodynamic pressures acting on the tank wall, and the tank wall uplifts locally, as shown in Fig. 2. As a result, a crescent-shape strip of the base plate is also lifted from the foundation. The weight of the fluid resting on the uplifted portion of the base plate provides the resisting moment against further uplift.

Since the radial displacements of the shell are relatively small, the linear theory for an axisymmetrically loaded cylindrical shell (Timoshenko and Woinowsky-Krieger, 1959 [3]) is applicable. The axisymmetric uplift problem considered is shown in Fig. 2. Point E will be referred to as the edge, and point C, as the contact point. The displacements are taken to be u and w in the r and z coordinate directions, respectively.

It is assumed that a) the foundation is rigid and frictionless; b) the tank is weightless and stress free when it is empty; c) both the base plate and the shell remain elastic, but a plastic hinge can form at the edge, E.

Shell at the edge is found to be given by:

\[ u = \frac{R(pR - v^p_s)}{E_s t_s} + \frac{\lambda^2}{2D_s} [M - \lambda H] \]  

(1)

\[ \phi_s = \frac{\lambda^2}{2D_s} [2M - \lambda H] \]  

(2)

In which

\( u \) = radially outward component of displacement of the edge;

\( \phi_s \) = rotation of the shell-wall at edge, taken to be positive in the anti-clockwise direction, as shown in Fig. 2;

\( H \) = radially inward force acting on the shell;
M = moment acting on the shell at the edge, defined to be positive when it acts in the same sense as the rotation \( \tau_s \);

\[
D_s = \frac{E_s t_s^3}{12(1-\nu_s^2)},
\]
the flexural stiffness of shell;

\[
E_s, \nu_s = \text{Young's modulus and Poisson's ratio for the shell, respectively;}
\]

\[
\lambda = \frac{[t_s R]^{3/2}}{3(1-\nu_s^2)^{3/4}},
\]
the characteristic length, which determines the rate of decay of bending moments in the shell;

\( t_s \) = thickness of base plate;

\( p \) = Fluid pressure at the edge (point \( E \) in Fig. 2).

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**Figure 2. Definition of axisymmetric uplift problem**

An estimate of the magnitude of the shell uplift may be obtained by using a modified version of a formula derived by Cambra (Eqn. 3).

\[
\begin{align*}
\max u &= \frac{1}{c} \left[ f_s t_s^2 \left( \frac{p_0 L_s}{6 f_{rs} t_s} + \frac{L_s}{2} - \frac{\bar{E} t_s^3}{12 f_{rs} t_s} \right) \right]^\frac{1}{2} \\
\end{align*}
\]

(3)

Where

\[
f_{rs} = \frac{1}{t_s} \left[ \frac{2\bar{E} t_s p_0^2 R^2 (1-\lambda)^2}{3} \right]
\]

\[
\bar{E} = \frac{E}{(1-\nu_s^2)}
\]

\( P_0 \) = Hydrostatic pressure on the base

**3-Uplifting Tanks**

If overturning moment (\( M_{OT} \)) exceeds \( M_R \) calculated by Eqn. 4, and the tank is unanchored, uplift will occur. The primary effect of uplift is to increase the compressive axial stress in the shell. In addition, distortion of the shell and tension forces in the tank base may need to be considered.

The mechanism of tank uplift is complex and not completely understood. To describe it fully, the effects of; large displacements of the uplifting base, yielding of the base-shell joint, membrane forces in the base, imperfections of the shell geometry, and
foundation flexibility need to be included. In Fig. 3, the overturning moment, \( M_{OT} \) is resisted by the action of three forces; \( W_s \), \( W_f \), and \( W \) forming \( M_R \) [4]:

\[
M_R = W_s \cdot KR + W_f \cdot (R - r)
\]

Where

- \( W = \) total weight of the fluid
- \( W_f = \) weight of the fluid supported directly by the foundation over the area that does not uplift (radius \( r \))
- \( W_s = W + W_W - W_f = \) compression reaction at shell base
- \( R = \) is the radius of the tank, and
- \( \theta = \) is the half angle which defines the arc of the shell base in contact with the foundation
- \( KR = \) distance from centre of compression reaction to tank centerline.

\[
\theta = \arctan\left(\frac{\mu}{1 - \mu}\right); \quad k = \frac{1}{\theta^2} \left(1 - \cos \theta\right)
\]

**4-Axial membrane stress in shell**

Rigid-plastic beam model used by Wozniak and Mitchell (1978) [5] to calculate the hold-down force due to the weight of fluid resting on the uplifted portion of the base plate. Note: Since the moment at the plastic hinge location, \( H \), is a maximum, the shear must vanish there. Therefore, the unknown distance \( HE \) and the force \( N \) can be determined by balancing the vertical forces and moments for the free body \( HE \). The hold down force is equal to the weight of fluid resting over the portion \( HE \) of the base plate.

\[
N_0 = t \left( f_y P \right)^{1/2}
\]

\( N_0 \) = hold down force per unit length along the circumference of the shell, which is also the vertical membrane tension developed in the shell at the base. Where \( t = \) thickness of the base plate, \( f_y = \) yield stress of the base plate, \( P = \) hydrostatic pressure acting on the base plate. For a rigid-plastic beam, the force \( N_0 \) is independent of the amount of uplift.
The resulting assumed distribution of vertical forces in the tank wall contains two unknown parameters: The maximum vertical force in the tank wall, denoted by \( N_{\text{max}} \) in Fig. 4, and the angle spanned by the contact region, \( 2\theta \). These two unknowns can be determined by balancing the vertical forces and moments acting on the shell.

The maximum axial stress in the shell is computed as [6]:

\[
N_{\text{max}} = 2.5 \frac{cW_s}{R\theta}
\]

Where \( c \), is a foundation stiffness factor (\( c=1.0 \) for a rigid foundation, \( c=0.5 \) for a flexible foundation)

![Rigid-plastic beam model](image)

**Figure 4. Rigid-plastic beam model used by Wozniak and Mitchell (1978) to calculate the hold-down force due to the weight of fluid resting on the uplifted portion of the base plate**

**5-Comparison between seismic standards**

The overturning moment caused by the hydrodynamic pressure tends to lift the shell off its foundation, thus developing highly-concentrated compressive stresses which may cause buckling of the shell. A literature review of available design standards shows that the API 650 standard for oil tanks (or alternatively, the AWWA standard for water tanks) has been the most commonly used standards for seismic design of tanks. In recent years, the New Zealand recommendations for seismic design of storage tanks have gained wide acceptance internationally. A comparison between the analysis procedures for unanchored tanks in these standards and guidelines is presented, and a critical evaluation of their accuracy is made. It should be noted that the performance of tanks during past earthquakes has revealed a much more complex behavior than is implied by current design procedures and continually demonstrates the need for more reliable analyses to assess their seismic safety. Note that the uplifting problem is nonlinear in nature because of the successive separation and contact of the bottom plate with its foundation [7].
Figure 5. Comparison between seismic standards; (1) seismic overturning moment, (2) shell compression stress, (3) allowable buckling stress

Reference:

2. Choon-Foo and Shih, "Failure of liquid storage tanks due to earthquake excitation", EERL 81/04.