SEISMIC DESIGN GUIDELINES FOR LIQUEFIED NATURAL GAS TANKS

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ABSTRACT

A methodology for assessing the seismic qualification of liquefied natural gas storage tanks is presented. Under earthquake excitations, the hydrodynamic pressure exerted on the tank walls produces overturning moment which may cause either a failure of the anchors or a buckling of the tank shell near its base. A systematic procedure for evaluating the demand on such tanks and for estimating their capacity against the possible modes of failure is presented. This procedure is further illustrated through a numerical example of a representative, existing storage tank.

INTRODUCTION

To achieve seismic safety of critical facilities such as tanks for storage of liquefied natural gas (LNG), the tank must be seismically rugged and adequately anchored. Otherwise, the tank may experience buckling of its shell, failure of its anchors and connections, uplift from its foundation and loss of gas content upon the occurrence of a relatively strong earthquake.

A survey of the literature on the dynamic performance of fluid-filled tanks, in general, and LNG tanks, in particular, was conducted in order to select the most appropriate analytical methodologies and criteria for assessing the seismic safety of LNG tanks. For the response to a horizontal component of an earthquake, the analytical model, well known as the mechanical analog developed by Haroun and Housner [5], was selected for use in estimating the seismic demand on LNG tanks. It should be noted that the internal gas pressure has no effect on the parameters of this mechanical analog. The model contains sufficient details to accurately represent the behavior of flexible tanks while possessing sufficient simplicity for implementation in design guidelines. The effect of the vertical component of an earthquake and the soil-structure interaction are readily accounted for in the proposed qualification procedure.

Guidelines for seismic qualification of LNG tanks have been developed [6] in the form of simple computation procedures using figures, tables and equations to compute the tank seismic demand and its capacity to resist such demand. An LNG tank, which is a component of Memphis Light, Gas and Water (MLGW), was used as an illustrative example demonstrating the implementation of the guidelines. This specific tank was further evaluated with a more elaborate analysis and the results are compared to the approximate analysis suggested by the guidelines to verify their validity.

EARTHQUAKE DAMAGE

Earthquake experience has shown that the majority of tank failures for anchored containers are the development of elephant foot or diamond buckles near the base of the tank shell, and sometimes, stretched anchor bolts. These failure modes are caused by the seismic overturning moment resulting from the pressure exerted by the contained fluid on the tank wall and base plate. Thus, the main concerns in evaluating the seismic safety of tanks are the overturning moment response and the strength of the tank shell base to resist this overturning. Flat bottom tanks resist base shear mainly by friction between the tank base and the supporting material under the weight of the content. Though seismic experience has not indicated that anchored flat-bottom tanks had slid, it is important to check the available friction to resist shear.

Both the hydrostatic and hydrodynamic pressures on the tank wall cause circumferential or hoop tensile stresses in the tank shell. Earthquake experience data indicated that no seismically induced failures due to the hoop stress had occurred. Thus, this stress is not in itself considered critical in causing a failure that may lead to loss of tank contents. Rather, the primary effect of the circumferential hoop stress is to influence the axial buckling capacity of the tank wall.

Earthquake experience has also shown occasional failure in the form of buckling in the roof or upper segment of the tank wall due to lack of sufficient freeboard which causes the sloshing fluid to exert a significant load on the tank roof. Since the roof of an LNG tank is typically a spherical dome and the free surface is under pressure, it is unlikely, for LNG tanks, to experience this mode of failure.

Because of the aforementioned observations of and/or speculations on the modes of failure, the seismic demand of the tank is directly related to those seismically induced loads which develop overturning moment, base shear and compression stress in the tank shell. The capacity of the tank is its resistance to these loads in terms of its buckling capacity, anchorage strength and slippage resistance. The tank is considered seismically adequate if its capacity exceeds its demand. Also, from the capacity calculation the designer can estimate the maximum safe ground motion that may occur without producing hazard to the tank.

STRUCTURAL CONFIGURATION

The structural configurations for low-temperature LNG storage tanks vary appreciably from one tank to another; however, the most typical shape is of the cylindrical flat-bottom type. Another important classification pertains to whether the tank is in-ground or above-ground. For this particular study, an LNG tank located in Memphis, Tennessee, which is a component of MLGW (Memphis Light, Water and Gas) was used to illustrate the procedure for the seismic assessment of LNG tanks.

The tank, shown in Fig. (1), is an above-ground cylindrical tank consisting of two metal tanks, one inside the other. The inner tank, which is directly in contact with the liquefied natural gas and cold gas vapor,

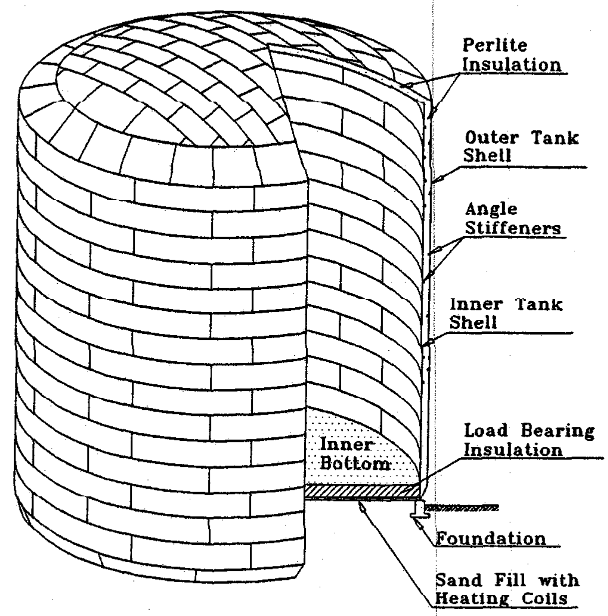

Figure 1: MLGW LNG tank

has a radius $R = 774''$ and a height $H = 1570''$. The tank is composed of 16 steel rings, each is 98.125″ high. The thickness varies from a maximum of 0.732″ at the bottom to a minimum of 0.3125″ at the top. The roof of the inner tank is a stiffened spherical dome which has a radius $R_s = 1238''$ and the roof plate thickness is 0.25″. The shell is made of 9 percent nickle steel, for which the mass density ρ_s is 7.34×10^{-4} lb.sec^2/in^4 and the yield stress σ_y is 36 ksi. The internal pressure of the liquid P_o is 0.50 psi whereas its mass density ρ_l is 4.39×10^{-5} lb.sec^2/in^4. There are 44 anchor straps holding the internal shell to the foundation, each strap has a tension area of $0.5'' \times 3.0''$. The inner tank is stiffened with ring stiffeners which are located at various heights along the shell. These stiffeners are made of single angles.

THE MECHANICAL ANALOG

Seismic response of fluid-filled tanks to the horizontal component of an earthquake has been found to be reasonably represented by two modes of response. One is a low-frequency mode referred to as the sloshing or convective mode, in which the contained fluid sloshes within the tank with negligible interaction with the deformation of the tank shell. The other is a high-frequency mode wherein the structure and fluid move together, called the impulsive mode. The tank shell deformation has significant effect on the impulsive mode. Thus, this mode is divided into two components: the impulsive-rigid component in which part of the tank and contained fluid move as rigid body with no contribution from the shell deformation, and the impulsive-flexible component which represents the interaction between the fluid and the tank deformation. The mechanical analog shown in [5] contains three equivalent masses: m_s, m_r, and m_f, and three equivalent springs with periods T_s, T_r, and T_f, located at equivalent heights H_s, H_r, and H_f, respectively. In the case of LNG tank, the inner tank alone will be represented by the mechanical analog while the outer shell should be designed to resist the live and wind loads. The thickness of the outer tank should not be less than 50% of the inner shell to ensure seismic adequacy of the outer tank.

The parameters of the sloshing (convective) mode are given as follows

$$\begin{aligned} m_s &= 0.455\pi\rho_l R^3 \tanh(\frac{1.84H}{R}) \\ &= 29,123 \text{ lb.sec}^2/\text{in} \qquad (1) \\ H_s &= H\left(1-(\frac{R}{1.84H})\tanh(\frac{0.92H}{R})\right) \\ &= 1169 \text{ inch} \qquad (2) \\ T_s &= 4.632\sqrt{\frac{R}{g}\coth(\frac{1.84H}{R})} \\ &= 6.56 \text{ sec} \qquad (3) \end{aligned}$$

where g = acceleration of gravity.

For the calculation of the impulsive parameters, one replaces the variable wall thickness by an equivalent thickness (t_{ef}). Now, one uses the figures in [5] to find the flexible-impulsive mode parameters m_f, H_f and T_f, and the rigid mode parameters m_r and H_r. Assuming firm soil, the mechanical analog periods need not to be adjusted. Hence,

$$\begin{aligned} m_f &= 91,619 \text{ lb.sec}^2/\text{in} \\ m_r &= 101,640 \text{ lb.sec}^2/\text{in} \\ T_f &= 0.5486 \text{ sec} \\ T_r &= 0.0 \text{ sec} \\ H_f &= 756.1 \text{ inch} \\ H_r &= 666.0 \text{ inch} \qquad (4) \end{aligned}$$

SOIL-STRUCTURE INTERACTION

Due to soil-structure interaction effects, a stiff tank resting on a soft soil may have an earthquake response significantly different from the case where it rests on a stiff soil. Soil-structure interaction usually lengthens the period of vibration of the impulsive modes and also increases the damping. Generally, an increase in the impulsive mode period may result in an increase in response but the additional damping tends to counteract this effect. Depending on the relative stiffness between the tank and soil, soil-structure interaction may change both the impulsive and convective periods but the effects on the convective mode are negligibly small. Veletsos [12] has developed a simplified procedure in order to take the soil-structure interaction into consideration.

SEISMIC DEMAND

Base Shear and Overturning Moment

The vertical component of an earthquake has a negligible contribution to the base shear and overturning moment which are mainly caused by the two horizontal components. Using the periods obtained from the mechanical analog, the corresponding damping ratios and the design response spectra at the tank site, one finds the modal accelerations A_s, A_f and A_r. For the tank under consideration, the recommended peak ground acceleration for zone 1 is 0.1 g. Using soil type 1 curve, one can find

$$\begin{aligned} A_s &= 15.44 \text{ in/sec}^2 \\ A_f &= 73.34 \text{ in/sec}^2 \\ A_r &= 38.60 \text{ in/sec}^2 \qquad (5) \end{aligned}$$

Hence the overturning moment and base shear can be estimated from

$$Q_b = \sqrt{Q_s^2 + Q_f^2 + Q_r^2} \qquad (6)$$

$$M_b = \sqrt{M_s^2 + M_f^2 + M_r^2} \qquad (7)$$

in which

$$\begin{aligned} Q_s &= m_s A_s \\ Q_f &= m_f A_f \\ Q_r &= (m_r + m_{rf} - m_f)A_r \\ M_s &= m_s H_s A_s \\ M_f &= m_f H_f A_f \\ M_r &= (m_r H_r + m_{rf} H_{rf} - m_f H_f)A_r \qquad (8) \end{aligned}$$

where m_{rf} = mass of inner tank roof and H_{rf} = height of center of gravity of inner tank roof. Note that Eqs. (6) and (7) employ the rule of SRSS since the maximum response of all modes will not occur at the same time. This yields

$$Q_b = 6.75 \times 10^6 \text{ lb and } M_b = 5.1 \times 10^9 \text{ lb.in} \qquad (9)$$

Fluid Lateral Pressure at Tank Base

The total pressure at the tank base, including the hydrostatic pressure and horizontal and vertical components of the earthquake, is given by

$$P_{max,min} = P_{static} + P_{horizontal} \pm 0.4P_{vertical} \quad (10)$$

The factor 0.4 is introduced to account for the assumption that the maximum pressure from the horizontal and vertical components will not occur simultaneously. The negative sign for the vertical earthquake pressure needs to be considered in determining axial buckling stress capacity at the tank base for the diamond buckling mode, as this capacity decreases with decreasing pressure.

The hydrostatic pressure is given by

$$P_{static} = \rho_l g H + P_o = 27.0 \text{ psi} \quad (11)$$

The pressure at the base of the tank due to the horizontal component of an earthquake is made up of both impulsive and sloshing components. It is found in [8] that this pressure has its maximum value, at a height 0.15 H, given by

$$\begin{aligned} P_{horizontal} &= \left(\frac{m_f H_f}{1.36RH^2}A_f\right) + 0.837\rho_l R A_s \\ &= 2.4 \text{ psi} \end{aligned} \quad (12)$$

The second term of Eq. (12) is the contribution of the convective mode which is small and may be neglected.

The pressure due to the vertical component of the earthquake is induced by the vertical spectral acceleration of the tank which is considered 2/3 A_f as an approximation. Thus, this pressure is given by

$$P_{vertical} = 0.533\rho_l H A_f = 2.68 \text{ psi} \quad (13)$$

Finally, the total pressure at the tank base is given by

$$\begin{aligned} P_{max,min} &= P_o + \rho_l R \\ &\times \left(g\frac{H}{R} + (2.31\frac{m_f}{m}\frac{H_f}{H} \pm 0.21\frac{H}{R})A_f\right) \\ &= 30.47,\ 28.33 \text{ psi} \end{aligned} \quad (14)$$

CAPACITY

The capacity of the inner tank and its anchorage to resist base shear and overturning moment are of main concern. The base shear is mainly resisted by sliding friction between the tank base plate and the supporting foundation material. The contribution from the anchors is conservatively neglected. The overturning moment is mainly resisted by compression in the tank wall and tension in the anchors. The overturning moment capacity is thus governed by shell axial buckling and anchor yield and slippage loads.

Elephant Foot Buckling Capacity

Under earthquake and gravity loading, the tank shell is subjected to a bi-axial stress state consisting of vertical compression stress induced mainly by the overturning moment and hoop tension due to the internal static and dynamic fluid pressures. Near the tank base, the high fluid pressure causes radial deformation of the shell which introduces eccentricity and bending stress in the axial plane. This causes the tank shell to bulge at the base. The buckling capacity of a vertical tank is most likely to be controlled by this mode of buckling. The effect of the stiffeners on the buckling capacity is conservatively neglected.

The buckling stress capacity in the elephant foot buckling mode is given by [11]

$$\begin{aligned} \sigma_e &= \frac{0.6Et_b}{R}\left(1 - (\frac{P_{max}R}{\sigma_y t_b})^2\right) \\ &\quad \left(1 - \frac{1}{1.12 + S_1^{1.5}}\right)\left(\frac{S_1 + \sigma_y/36}{S_1 + 1}\right) \\ &= 2670 \text{ psi} \leq 36,000 \text{ psi} \end{aligned} \quad (15)$$

where σ_y = yield stress of tank shell material in (ksi); E = modulus of elasticity of tank shell material; t_b = thickness of tank shell at the base; and

$$S_1 = (R/400t_b) = 2.64 \quad (16)$$

Diamond Buckling Capacity

Another possible buckling mode for vertical tanks subjected to earthquakes is the diamond buckling mode. This is the allowable buckling stress for tank shell under membrane compression. The allowable buckling stress in the diamond buckling mode can be conservatively estimated as follows

$$\phi = \frac{1}{16}\sqrt{\frac{R}{t_b}} = 2.03 \quad (17)$$

$$\gamma = 1 - 0.73(1 - e^{-\phi}) = 0.366 \quad (18)$$

$$\sigma_d = (0.6\gamma + \Delta\gamma)\frac{Et_b}{R} \leq \sigma_y \quad (19)$$

where $\Delta\gamma$ is an increase factor due to internal pressure given in Fig. (2) versus the dimensionless factor χ where

$$\chi = \frac{P_{min}}{E} \times (\frac{R}{t_b})^2 \quad (20)$$

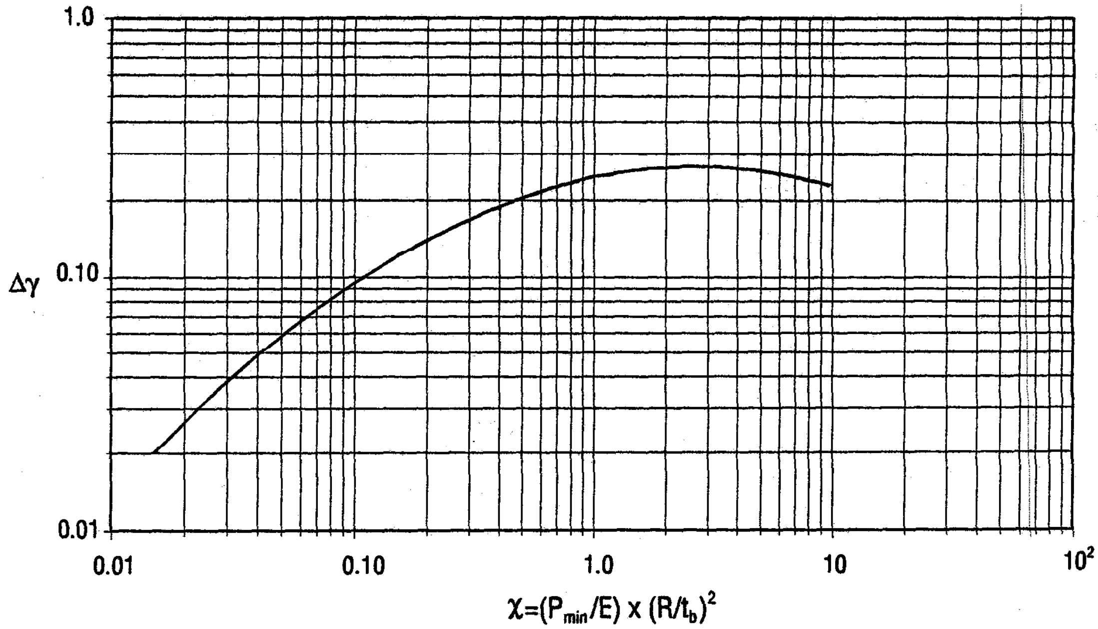

Figure 2: The stability factor due to internal pressure $\Delta\gamma$ (After [1])

For the tank under consideration, this yields

$$\chi = 1.1, \ \Delta\gamma = 0.22, \text{ and } \sigma_d = 12,057 \text{ psi} \leq \sigma_y \quad (21)$$

The buckling stress capacity is recommended to be

$$\sigma_b = 0.72 \text{ least of } (\sigma_e, \sigma_d) = 1922.5 \text{ psi} \quad (22)$$

Overturning Moment and Base Shear Capacity

The overturning moment at the base of an anchored tank is resisted mainly by compression in the tank shell and tension in the anchors. To calculate the overturning moment capacity, plane sections are assumed to remain plane at the tank base under the overturning moment. As the tank uplifts on the tension side, there will be additional overturning resistance provided by the uplifted fluid weight. This contribution is conservatively neglected in anchored LNG tanks.

To make it convenient to compute capacity, it has been assumed that the anchor straps can be replaced by a shell of equivalent thickness t_e of the same material and radius as that of the tank

$$\begin{aligned} t_e &= \frac{\text{Number of anchor straps} \times \text{Area}}{2\pi R} \times \frac{E_{anchor}}{E_{tank}} \\ &= 0.0136'' \end{aligned} \quad (23)$$

The equilibrium and compatibility equations are then written and solved to calculate the overturning moment capacity as the capacity of the equivalent tubular section. It is important to note that the maximum anchor tension stress σ_t must be the least of yield stress, slippage stress and welding stress. The local buckling due to the transmission of the anchorage force from anchors to tank shell will be accounted for by considering the ratio of height of anchor strap above the concrete pad, h_c, to height of whole anchor strap, h_b. The length h_c indicates the local buckling length of the shell while h_b indicates the tension straining of the anchor assumed to occur over the whole anchor length.

Using the dimensionless ratios c_1 and c_2 given by

$$c_1 = \frac{t_e}{t_b} \times \frac{h_c}{h_b} = 0.0089 \quad (24)$$

$$c_2 = \frac{\sigma_b}{\sigma_t} \times \frac{h_c}{h_b} = 0.051 \quad (25)$$

one obtains the dimensionless overturning moment capacity $M'_{cap} = 0.02$. Accordingly, the overturning moment capacity is given by

$$\begin{aligned} M_{cap} &= 2M'_{cap}\sigma_t R^2 t_b \frac{h_b}{h_c} \\ &= 0.66 \times 10^9 \text{ lb.in} \end{aligned} \quad (26)$$

in which it is assumed that σ_t is 18,000 psi.

The base shear is mainly resisted by sliding friction between the tank base plate and foundation material.

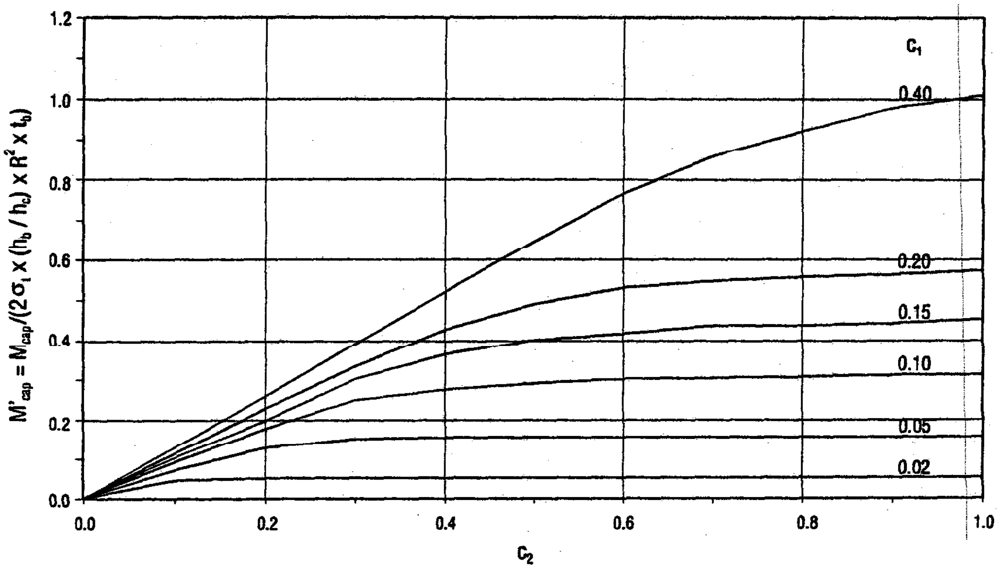

Figure 3: Overturning moment capacity.

The sliding capacity is given by

$$\begin{aligned} Q_{cap} &= c_f P_b \pi R^2 = c_f(1 - 0.21\frac{A_f}{g})W \\ &= 19.3 \times 10^6 \text{ lb} \end{aligned} \quad (27)$$

where c_f = coefficient of friction between tank base plate and foundation material taken to be 0.4; P_b = pressure on tank base plate for gravity and seismic loading and W is the weight of the liquefied natural gas.

Finally, the factors of safety are evaluated from

$$\begin{aligned} \text{Moment F.S.} &= \frac{0.66 \times 10^9}{5.1 \times 10^9} = 0.129 \text{ Unsafe} \\ \text{Shear F.S.} &= \frac{19.3 \times 10^6}{6.75 \times 10^6} = 2.9 \text{ Safe} \end{aligned} \quad (28)$$

OTHER ANALYSES

The tank has also been shown to be unsafe using the New Zealand recommendations [8]. Using the API standard [1], assuming that the tank is adequately anchored, shows that the tank is safe. This is partly due to neglecting the impulsive-flexible component which, in turn, reduces the overturning moment, and partly due to ignoring the elephant-foot buckling mode which, in turn, results in a higher buckling capacity of the wall.

A finite element analysis was also carried out for the present tank [6]. A large-displacement buckling model fixed at the base has shown that the buckling stresses used in the above design procedure are conservative (Table 1). This design procedure uses the buckling stress under axial loads to evaluate the overturning moment capacity. This buckling stress is, as expected, low when compared to the stress required to buckle the tank under hydrodynamic loads seen in a seismic event.

A comparison of the overturning moment required to buckle the tank has also shown that the ring stiffeners currently present in the above tank do not improve the overturning moment capacity of the tank. This is due to the absence of any stiffeners in the vicinity of the elephant buckle. It has been observed that the overturning moment capacity of the tank improves when stiffeners are placed near the bottom of the tank shell. For more details, refer to [6].

Furthermore, a finite element analysis which included the anchors shows that the anchor straps used in the tank yield even under the static loads only. Thus it is inadequate to assume fixity at the base. If the tank is assumed to lose it's anchorage, the API standard shows that it becomes unstable and overturns.

SLOSHING AMPLITUDE

In the above development, it is assumed that there is enough freeboard available between the quiescent liquid surface and the tank roof such that the roof is not subjected to significant forces from the impact of the sloshing liquid. The recommended damping ratio for sloshing is 0.5%. The sloshing height is given by

$$h_s = 0.837R\frac{A_s}{g} = 25.9 \text{ inch} \quad (29)$$

The freeboard between the liquid free surface and the

Type of Analysis	Maximum Overturning Moment	Maximum Compressive Stress
MLGW LNG tank without stiffeners	7.9141×10^9lb.in	10.78 ksi
MLGW LNG tank with stiffeners*	7.9283×10^9lb.in	10.81 ksi

*The stiffeners are not in the vicinity of the elephant foot buckle

Table 1: Overturning moments obtained from FEM analysis

roof should not be less than h_s.

CONCLUSION

Systematic procedures for assessing the seismic qualification of liquefied natural gas storage tanks are presented. Methods for accurately predicting both the demands on and the capacity of such tanks are summarized. The implementation of these procedures was illustrated through a numerical example of a representative, existing storage tank. The use of more elaborate analyses, which are increasingly becoming available, is recommended to complement current design standards.

ACKNOWLEDGEMENT

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REFERENCES

[1] American Petroleum Institute, Welded Steel Tanks for Oil Storage, API Standard 650, 7th Edition, Washington, D.C., 1980.

[2] Asai, K., Yanabu, K., Goto, Y., and Kikuchi, T., "Studies on Earthquake Response of On-Ground LNG Storage Tank Based on Observed Record," Pressure Vessels and Piping Conference, ASME, Vol. 145, 1988, pp. 71-76.

[3] Haroun, M.A., Dynamic Analyses of Liquid Storage Tanks, EERL 80-4, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California, February 1980.

[4] Haroun, M.A., and Housner, G.W., "Dynamic Interaction of Liquid Storage Tanks and Foundation Soil," Specialty Conference on Dynamic Response of Structures, ASCE, Georgia, January 1981, pp. 346-360.

[5] Haroun, M.A., and Housner, G.W., "Seismic Design of Liquid Storage Tanks," Journal of Technical Councils, ASCE, Vol. 107, April 1981, pp. 191-207.

[6] Haroun, M.A., Zeiny, A.A. , and Bhatia, H., Seismic Qualification of Liquefied Natural Gas Tanks - Guidelines and Implementation, Report to the National Center for Earthquake Engineering Research, University of California, Irvine, June 1994 (under preparation).

[7] Joos, F.M., and Huber, P.W., "Coupled Gas-Liquid-Structure Systems: Part 2 - Applications," Journal of Applied Mechanics, ASME, Vol. 54, No. 4, December 1987, pp. 942-950.

[8] New Zealand National Society for Earthquake Engineering, Seismic Design of Storage Tanks, Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering, 1986.

[9] Terada, K., Tsuda, K., and Kano, Y., "Technical View of LNG and LPG Storage Tanks," Mitsubishi Heavy Industries, Vol. 21, No. 3, 1984, pp. 190-196.

[10] Sakai, F., Nishumura, M., and Sakoda H., "Studies on Earthquake Resistance of Liquefied Natural Gas Storage Tanks," Proceedings of the 6th International Conference and Exhibit, LNG, Kyoto, Japan, 1980.

[11] URS Consultants/John A Blume & Associates, Seismic Verification of Nuclear Plant Equipment Anchorage, Report to Electrical Power Research Institute, June 1991.

[12] Veletsos, A.S., "Seismic Response and Design of Liquid Storage Tanks," Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, ASCE, New York, 1984.