CHARACTERIZATION OF OBSERVED UPLIFT AND BUCKLING OF AN UNANCHORED TANK DURING THE NORTHRIDGE EARTHQUAKE

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SUMMARY

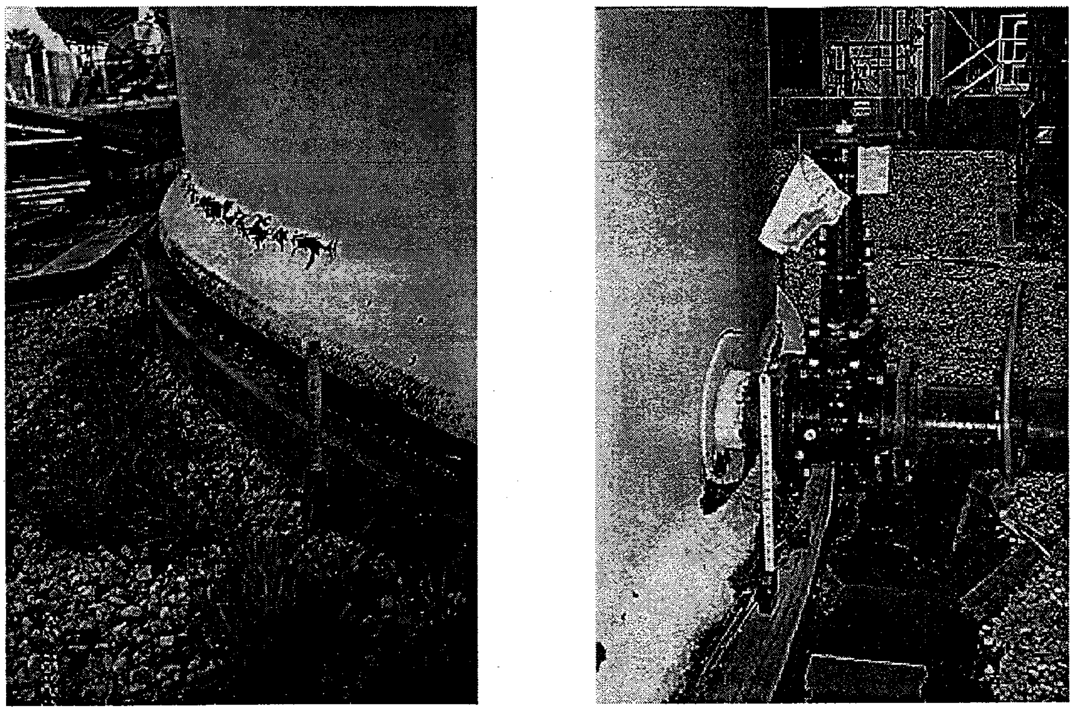
The observed behavior of an unanchored cylindrical liquid storage tank during the 1994 Northridge earthquake is correlated with numerical simulations of its base uplifting and shell buckling. The tank experienced base uplift during the earthquake and suffered elephant-foot buckling near its base. Important lessons learned, in particular, as related to the application of current seismic design standards for predicting the actual behavior of the tank, are outlined. Detailed nonlinear quasi-static as well as full-transient analyses are presented to demonstrate the use of state-of-the-art finite element codes in assessing the seismic safety of such tanks.

A CASE STUDY IN THE NORTHRIDGE AREA

The Northridge earthquake of January 17, 1994, caused severe damage to a number of cylindrical liquid storage tanks, and even resulted in the collapse of a tank. Typical modes of tank damage, primarily buckling of the shell and failure at the roof-shell connection, were observed throughout the affected area. A description of the damage to tanks can be found in [5]. The majority of failures in containers during past earthquakes occurred with the development of extensive buckling near the tank shell bottom (elephant-foot buckling) and the Northridge earthquake was no exception. It should be noted that both hydrostatic and hydrodynamic pressures on the tank wall cause circumferential (hoop) tensile stresses in the shell. Earthquake data have indicated that no seismically induced failures due to hoop stress have occurred. However, the primary effect of the circumferential hoop stress is to influence the axial buckling capacity of the tank wall.

The present paper provides an overall view of the use of seismic codes as well as up-to-date analyses for the prediction of the response of an unanchored tank which experienced a noted uplift of its base and suffered elephant-foot buckling in the shell (Fig. 1a). The tank was located at the American National Can site in Northridge and was subjected to severe forces during the earthquake. The free-field motion recorded at the Arleta site, which is fairly close to the tank site, showed a peak ground acceleration of $0.344g$. The tank uplifted during the earthquake and, when dropped, it severed the adjacent piping (Fig. 1b) and released its content. The tank was designed in 1990 and

FIGURE 1
UNANCHORED TANK AT THE AMERICAN NATIONAL CAN SITE IN NORTHRIDGE



was situated on a concrete pad foundation. It has a diameter of 37 ft, a height of 32 ft, and was filled with water to a depth of 28.3 ft at the time of the earthquake.

SEISMIC DESIGN STANDARDS

Seismic performance evaluation of ground-based tanks should include an assessment of tank sliding, tank overturning, and the level of hoop, shear and axial stresses. The tank should also be evaluated for freeboard, for column support, for piping considerations, and for foundation considerations. The most critical of these criteria are the global overturning of the tank, the excessive compressive stresses, and the large amplitude of liquid sloshing.

OVERTURNING MOMENTS

Overturning moments in the AWWA standard [1] as applied to the bottom of the shell and to the foundation, respectively, are determined using the simplified procedure developed for rigid tanks. A lateral force coefficient is specified to represent the amplified tank acceleration as a ratio of the acceleration of gravity; it does not take into account explicitly the effects of shell flexibility, site conditions, or support conditions (anchored vs. unanchored). Each of these factors may amplify or reduce the tank acceleration. It is noted that the site amplification factor affects only the value of the convective forces. When the value of a "moment parameter" exceeds 1.57, the tank is deemed to have overturned, and according to the AWWA procedures, it must be anchored.

In the New Zealand guidelines [7], a mechanical analog [2] which takes into account the deformability of the tank wall, is used to reduce the tank and its content to equivalent masses and springs at

TABLE 1
COMPUTATIONS ACCORDING TO AWWA STANDARDS AND NEW ZEALAND GUIDELINES

Analysis Type	AWWA Standard		New Zealand Guidelines	
Response Spectra	—	Arleta	—	Arleta
Overturning Moment $\times 10^8$ lb.in	0.916	1.323	1.433	1.755
M/WR	0.22	0.31	0.34	0.46
Allowable Stress, ksi	4.8		14.1	12.3
Compression Stress, ksi	*****	*****	11.4	14.7
Compression Stress for Anchored Tank, ksi	1.36	1.92	2.46	3.13
Moment Parameter	3.14	4.54	N/A	N/A
Factor of Safety	N/A	N/A	1.24	0.84
Conclusion	Anchor	Anchor	Safe	Unsafe

**** indicates tank is unstable, compression stress is not computed.

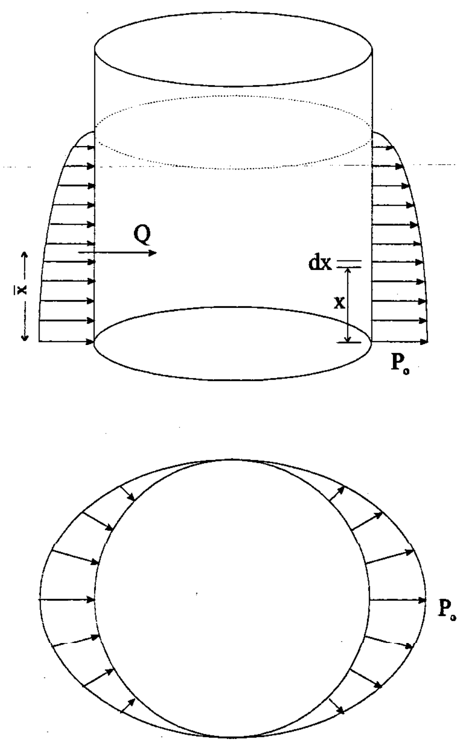
different heights. It should be noted that this mechanical analog was developed for tanks anchored at their base. However, due to the lack of readily available methods, the recommendations include this model for calculating the amplified overturning moments in unanchored tanks! A correction formula is used to modify the frequencies of the mechanical analog, and the corresponding damping ratios, depending on the soil type and its properties.

AXIAL STRESS

The resistance to overturning is provided by the weight of the tank shell and by the weight of a portion of the tank content which depends on the width of the bottom annular ring that lifts off the foundation or, in the case of no annular plate, on the portion of the bottom plate that uplifts. To determine this width, an elemental strip of the bottom plate perpendicular to the shell which can be lifted off the ground is considered in the AWWA standard. It is noted that neither the deformability of the tank wall nor the flexibility of the underlying soil are considered in the model. In addition, only small deformation behavior of the bottom plate is taken into account. At higher levels of ground excitation, the capability of the plate to resist the applied loads would not be fully accounted for, leading to conclusions of global instability (overturning). One other discrepancy noted in the AWWA procedure is that the overturning moment at the tank base and the uplift force are independent of each other which is unrealistic as the overturning moment produces the uplift force!

In the New Zealand guidelines, the stress in the tank shell is calculated using global equilibrium of the shell and its base. It is assumed that the bottom plate remains in contact with the foundation on a circular area of a radius slightly less than the radius of the tank, and that the edge of the shell rests on an arc of the tank's perimeter of unknown central angle. One disadvantage of this model is that it does not consider the deformation behavior of either the tank wall or the bottom plate. Furthermore, it neglects the variation of the dynamic pressure on the bottom plate and uses a constant value equal to the static pressure.

FIGURE 2
PSEUDO-DYNAMIC LOADS APPLIED ON TANK WALL



A commercial finite element analysis program MARC was used for the analysis. An optimized and vectorized version of MARC was used on a CONVEX mainframe computer. The salient features of the finite element analysis were:

- A high mesh density was used to correctly model the buckling behavior.
- The aspect ratio of the shell elements was close to 1.
- A large-displacement analysis method was used instead of the eigenvalue approach. This approach incrementally loaded the structure until displacements increased rapidly and caused divergence. It is more accurate when material nonlinearity may influence buckling.
- A linearly-elastic perfectly-plastic material model was used to model steel.
- A full Newton-Raphson solution approach was used because of better convergence characteristics for the very large displacements obtained.
- The hydrostatic and hydrodynamic pressures were applied as pressure at the element integration points through a user subroutine, and were integrated using the element shape functions to obtain nodal forces.

The resulting deformed finite element mesh is shown in Figure 3. The maximum overturning moment (M/WR) obtained by this analysis, i.e., when the radial displacements at the bottom of the

ALLOWABLE STRESS

The allowable buckling stress in the AWWA standard is based on the classical value of buckling under axial load, significantly reduced by a large knock down factor due to shell imperfections and also increased to account for the effects of the internal liquid pressure. In the New Zealand guidelines, the classical buckling stress in membrane compression is calculated, then corrected according to the imperfection amplitude and the internal pressure. In addition, an elastic-plastic collapse stress is also calculated. The lower of the two stresses is used in the computation of the factor of safety. A recent nonlinear finite element study [6] showed that elephant-foot buckling is the most likely mode in a typical tank, and generally, it confirmed the accuracy of the assumed buckling capacity presented in the New Zealand guidelines.

BEHAVIOR ACCORDING TO SEISMIC STANDARDS

Code computations were carried out for the AWWA standard and the New Zealand guidelines. A summary of the results is presented in Table 1. Note that W denotes the total weight of the contained liquid and R is the tank radius. Both the AWWA and the New Zealand guidelines allow use of site specific response spectra. The spectra of the free-field ground motion recorded at Arleta, which is fairly close to the site of the tank, resulted in significantly higher overturning moment demands for the tank. The AWWA standard showed that the tank is unstable and required anchorage. If the tank is anchored, the computed compressive stresses are fairly low. The New Zealand guidelines predicted membrane compression buckling whereas this tank sustained elephant-foot buckling as was also demonstrated by the finite element analysis.

QUASI-STATIC ANALYSIS UNDER SEISMIC LOADING

A quasi-static large-displacement finite element analysis was performed to determine the "true" capacity of the tank. A full 3-D model using shell elements and large-deflection contact analysis was used. To confirm that the model used has sufficient mesh density to correctly capture the buckling behavior, simply-supported cylindrical shells were subjected to increasing axial loads. A mesh with 96 elements along the circumference was selected as this captured 93% of the classical buckling load.

As shown in Figure 2, a hydrodynamic pressure distribution was assumed on the tank wall as

$$p = p_o \left(1 - \frac{x^2}{H^2}\right) \cos\theta \tag{1}$$

where x is the elevation of a point on the shell measured from the base, H is the liquid depth, θ is the angle measured from the axis of excitation and p_o is the pressure amplitude at the tank base at $\theta = 0$. An analysis of rigid tanks showed that the hydrodynamic pressures exerted on the wall of the tank are indeed similar to those presented in the above equation.

If Q is the base shear, then

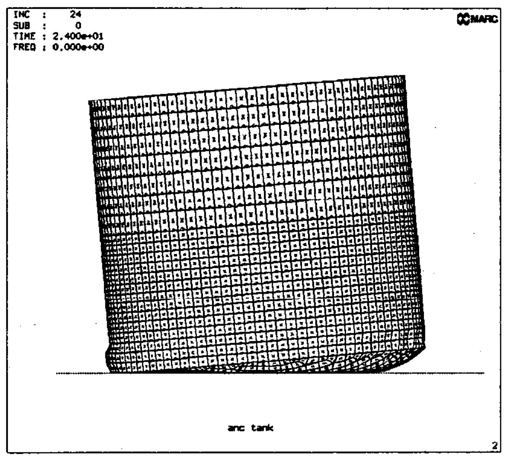
$$Q = \int_0^H \int_0^{2\pi} p \cos\theta \, R \, d\theta \, dx = \frac{2\pi}{3} H p_o R \tag{2}$$

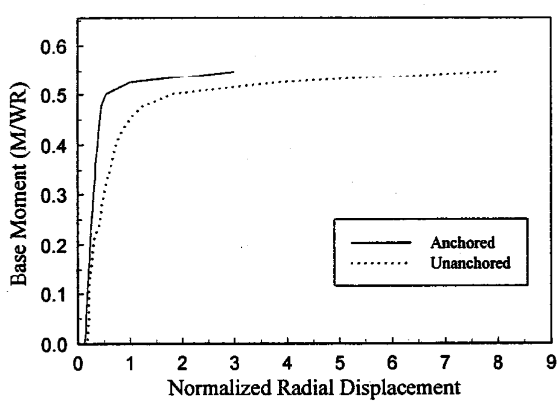
and similarly, if M denotes the overturning moment about the center of the base, then

$$M = \int_0^H \int_0^{2\pi} p x \cos\theta \, R \, d\theta \, dx = \frac{\pi}{4} H^2 p_o R \tag{3}$$

shell increase rapidly without bound, is 0.56 whereas those obtained by the AWWA standard and the New Zealand guidelines were 0.31 and 0.46, respectively, under the Northridge Arleta response spectrum. Considering dynamic magnification and shell imperfections, a reasonable estimate of the allowable overturning moment ratio is in the vicinity of 0.30.

FIGURE 3
DEFORMED SHAPE OF THE ANC TANK UNDER QUASI-STATIC LOADING





TRANSIENT RESPONSE

A nonlinear time-dependent computer simulation of the ANC tank was also conducted taking into consideration large amplitude liquid sloshing and the geometric, material and contact nonlinearities of the tank shell and base plate [4]. The response of the unanchored tank was governed primarily by a rocking motion. This mode was found to have a dominant period of 0.4 sec. and, based on this period, the foundation rocking damping is estimated to be 3%. On the other hand, the response of a similar anchored tank was governed primarily by the flexible-impulsive pressure component which has a fundamental period of 0.09 sec. A Raleigh damping coefficient which provides 3% damping to the first mode and increasing values for the higher modes was chosen for the flexible-impulsive component.

Since the rocking period is relatively large as compared to the flexible-impulsive period, the overturning moment exerted on the anchored tank was found to be larger than that exerted on the unanchored tank. However, due to the nature of the boundary conditions associated with the base of the unanchored tank, the axial and hoop stresses at the bottom of the unanchored tank shell were much larger than those of the anchored tank.

FIGURE 4
OVERTURNING MOMENT IN THE ANC TANK

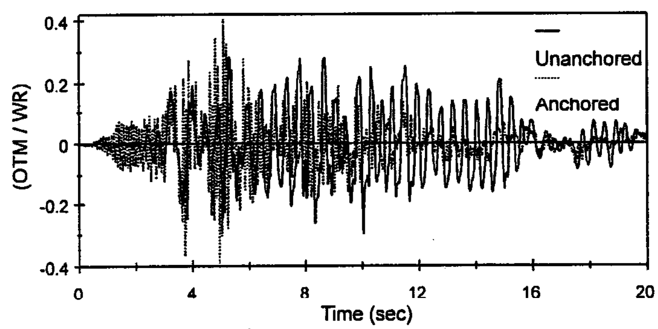


FIGURE 5
BASE AXIAL STRESS IN THE ANC TANK SHELL

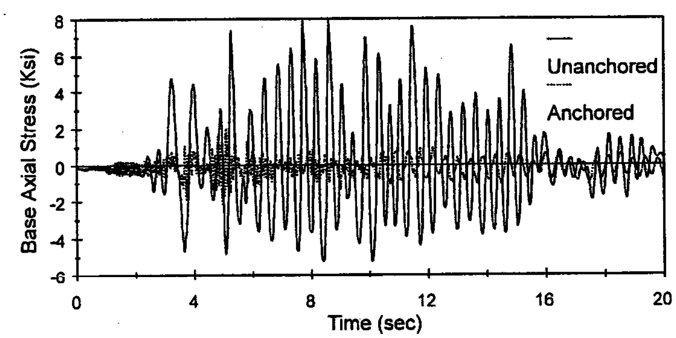


Figure 4 and 5 show a time history comparison between the anchored and the unanchored tank for the overturning moment and the base axial stress, respectively. It should be noted that the results presented here include the effects of contact and geometric nonlinearities. The effect of material nonlinearities, considered by the formation of plastic hinges, increases the uplift displacement.

CONCLUSION

A case study of an unanchored water storage tank that sustained elephant-foot buckling at its shell base during the 1994 Northridge earthquake is presented. The tank design appears not to satisfy the provisions of the AWWA standard which require it to be anchored to avoid being overturned. The New Zealand guidelines predicted that the shell would buckle under the Arleta response spectrum. A nonlinear quasi-static finite element analysis of the tank under assumed pseudo-hydrodynamic loading illustrated the buckling mode and the capacity of the shell. Finally, a time-dependent nonlinear finite element analysis was carried out under the Arleta free-field motion. It was observed that the overturning moment exerted on the unanchored tank is smaller than that exerted on a similar anchored tank due to the long-period nature of the rocking-motion which dominates the behavior of the unanchored tank. However, due to the nature of the boundaries of the unanchored tank at its base, the axial and hoop stresses at its shell bottom are larger than those of the anchored tank.

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