Strength and Stability of Uplifting Earthquake—Loaded Liquid, Filled Tanks

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ABSTRACT

Based on analytically calculated maximum pressure distributions during earthquakes the strength and stability of liquid filled tanks are computed. It is shown that free base tanks - with possible partial uplift of the base - have a significantly different behaviour than anchored tanks. Therefore, care has to be taken in modelling the uplift mechanism. A critical investigation of the uplift models currently used in some design rules gives rise to the development of improved models as presented in this paper.

1. INTRODUCTION

Most of the papers dealing with the analysis of liquid storage tanks under earthquake excitation deal with anchored tanks.

The latest state of research was presented in a very comprehensive manner at the 8th World Conference on Earthquake Engineering, San Francisco 1984. The authors refer here to papers by Yamamoto et al /1/ and Sakai et al /2/ which report on the influence of a weak foundation on the coupling of the different modes excited during an earthquake.

The evaluation of damage data for different tanks injured by an earthquake, - e.g. the papers of Moore et al /3/ and Haroun /4/ - shows the main reason for damage of tanks being the buckling of the shell near the bottom ("Crippling" or "Elephant - Footing"). Therefore, it is necessary to know the distribution of the axial membrane forces in the shell which is easy to be found for anchored tanks due to usual membrane shell theory.

The situation is totaly different for uplifting tanks - and many broad tanks in the chemical industry are not anchored. In the case of uplifting one has to investigate a nonlinear fluid-structure-soil interaction problem. The (approximatively) antimetry for the horizontal earthquake is lost. The stiffness of the bottom plate has now an essential influence on the longitudinal membrane forces in the shell.

The axial membrane compression forces are much more concentrated in the case of tall uplifting tanks and may have maxima out of the direction of the earthquake in the case of broad tanks. As a consequence different buckling mechanisms appear.

2. APPROACHES TO THE UPLIFT-MECHANISM

2.1 Current models

D.P.Clough /5/ assumes that the uplifted tank rests on a line with an angle at the centre of $2\varphi^*$ along the circumference and in the area of a circle with the radius $r < R$ (tank - radius). Within $-\varphi^* < \varphi < \varphi^*$ the reaction force on the tank wall is distributed linearly, $N_x = N_x^D(1 - |\varphi| / \varphi^*)$. A "fixed" geometrical relation exists between the angle φ^* and the radii r, R. $r = \mu R$, $\tan \varphi^* = \mu / (1 - \mu)$. D.P. Clough does not take into account the deformation behaviour of the tank wall and the bottom plate. Further he assumes on the circle with radius r the static pressure being the constant reaction pressure. The overturning moment M effected by the dynamically activated fluid consists of the moment M on the tank wall and the moment BM on the tank bottom.
Analytical relations for M and BM can be found based on Housner's analysis for tanks with rigid walls in a paper by Epstein /6/ and for deformable walls e.g. in a paper by Fischer /7/.
N_x^D and φ^* are found by solving two nonlinear equations following from the global force equilibrium in vertical direction and the global moment equilibrium due to a horizontal axis. The model can be improved by assuming the reaction pressure as the sum of the static pressure and the dynamically activated bottom pressure.
A further model was developed by Wozniak /8/ assuming that the uplifted tank rests on a line with an angle at the centre of $2\varphi^*$ along the circumference, and on the area of a sector of a circle with the radius R with an angle at the centre of $2\varphi^*$ and on the area of a sector of a circle with the radius $(R - L)$ with an angle at the centre of $(2\pi - 2\varphi^*)$, L is the uplift length. The reaction force N_x is assumed to be distributed by a cosinus-function. Wozniak does not consider the flexibility of the tank wall but does take into account the deformation behaviour of the tank bottom plate, which is represented by strips of the width "1" and the length L. This strip, acting as a beam with bottom plate-thickness t which rests on the floor, is loaded by the bottom pressure p and is lifted up by a force V at its left end, see Fig. 1. The floor is assumed to be rigid. Now Wozniak calculates the maximum value of V as that force which brings the beam to a kinematic mechanism with two plastic hinges in the sense of the limit load of the beam. With the yield stress σ_y it follows $L = 1.707\, t(\sigma_y/p)^{1/2}$, $V_{max} = t(\sigma_y\ p)^{1/2}$. N_x^D and φ^* are again found by solving the equilibrium equations and can be corrected by omitting an inconsistency in N_x. Cambra /9/ tried to consider a longitudinal membrane force N in the bottom plate too but he involved some incompatibilities in his assumptions.
Clough's and Wozniak's models are compared for a specific tank for which model tests were performed by Niwa /10/. Fig. 2 shows φ^* and $N_x^D\, R/G$, G weigth of liquid content, demonstrating the difference between both models and the

measurements.

2.2 An improved analytical model

The authors developed in /11/ a refined analytical strip model for the bottom plate based on the second order beam theory equation

$$EJw^{IV} - Nw'' = p, \; N \sim \text{const. in } 0 < x < L \qquad (1)$$

and the boundary conditions (see Fig. 1)

$x = 0,\ w = 0;\ x = 0,\ w' = 0;\ x = 0,\ w'' = 0;\ x = L,\ EJw'' = 0$ (free) or $w' = 0$ (clamped), $x = L,\ EJw''' = V + Nw'$.

The missing 6th equation for calculating the 6 unknowns (4 coefficients of the homogeneous part of the differential equation, L, N) is given by integration of the nonlinear strain equation

$$\varepsilon_{xx} = u' + \frac{w'^2}{2} = N/Et \qquad (2)$$

Because within the interval $-B < x < 0$ N is 'transferred' into the foundation by friction (coefficient μ), $u(0) = N^2/(2\,E\,t\,\mu\,p)$. The elongation can be assumed to be $u(L) \sim 0$ for a very stiff shell or can be found from the coupling condition shell/bottom (effective spring constant γ). The solution of this highly nonlinear system of equations leads to a relation between the uplift height w(L) and V. Using the dimensionless quantities

$$V^* = V/(p\ \ t),\ w^* = w(L)\ E/(p\ \ t),\ p^* = (12\ p/E)^{1/2} \qquad (3).$$

Fig. 3 shows the results for different values μ and rotation conditions at $x = L$. In Fig. 4 the relation between V and w is shown for the Niwa tank, taking the coupling condition and $\mu \to \infty$ into account.

Using the strip model, the stiffness effect of the bottom plate on the shell can then be represented approximatively by an axial spring distribution on which the lower edge of the shell rests. These axial springs have nonlinear characteristics in tension (V as a function of w(L)) and compression (deformation of foundation). The main difference to Wozniak's model consists in the use of not only one specific value for V (see horizontal lines in Fig. 3) but of a spring with a deflection-dependent force V.

2.3 A numerical approach

Fig. 5 shows an axisymmetric finite element model for the computation of the nonlinear effective spring characteristic. This model allows the consideration of the elastic plastic material behaviour as well as the variation of the bottom plate thickness and foundation stiffness over the radius (which is relevant for large storage tanks) by simply varying the model parameters. Results obtained for a full scale tank (40 m diameter, 20m height) are also shown in Fig. 5.

3. STRENGTH AND STABILITY

Using the nonlinear effective spring characteristics obtained as described above finite element models consisting of the tank wall, loaded by the analytically calculated maximum pressure distributions and dead weights, resting on nonlinear springs represent the complete earthquake loaded tanks in an incremental-iterative analysis analogous to ref. /12/. The relevant results from the nonlinear strength analysis, namely the axial membrane force distribution at the base, is shown in Fig. 6 for two different tanks: a) the uplifting Niwa-tank /10/ under the action of a modified El Centro earthquake and b) an uplifting full scale tank (see above) excited by an earthquake corresponding to the USAEC response spectrum. The results were obtained for the action of the horizontal component of the earthquake only. The influence of the vertical component is described below.

Fig. 6 shows that the base uplift results in a concentration of the axial membrane compression forces in the case of rather stiff tall tanks, e.g. Niwa´s model tank , but in the case of very flexible broad tanks (e.g. analysed full scale tank) excentrical compression force maxima may appear.

Mainly these axial membrane forces are responsible for the stability loss of earthquake loaded storage tanks. The application of the stability analysis algorithm as described in ref. /12/ leads to results represented in Fig. 7. The surprising excentrical local buckling of the full scale tank is the consequence of the excentrical maxima of the axial membrane compression force distribution.

A consideration of the estimated critical axial membrane stresses confirmes the statement made in ref. /12/ and the fact observed in several experiments that the application of the buckling formula for axisymmetrically compressed cylinders leads to an unjustificable underestimation of the stability limit load, i.e. the critical value of the maximum ground acceleration. However, it must be mentioned that in the case of unanchored tanks a given earthquake leads in general to considerably higher values of the maximum axial compression force than in the case of anchored tanks. Hence, the danger of damage by buckling is for unanchored tanks more pronounced.

A vertical earthquake component may also significantly increase or decrease the stability limit load. The according axisymmetric pressure distribution has its maximum at the bottom and can be found due to Luft, /13/. It can be estimated for the full scale tank 20 % and the Niwa-tank 80 % of the static pressure. In case of internal (positive) pressure the buckling load is increased unless plastification does not occure due to circumferential overstressing. This drastically decreases the buckling strength. Some authors, /14/, point out that plastification and not buckling is the only reason for the well known "Elephant Footing" - deformation shape. Negative pressure reduces the internal pressure and therefore the buckling load.

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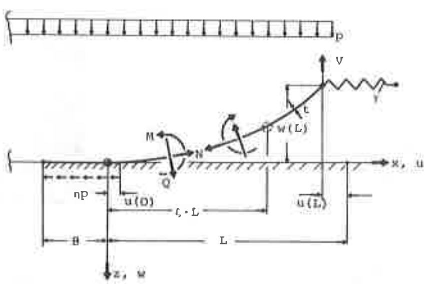

Fig. 1 The strip model representing the uplifting bottom plate

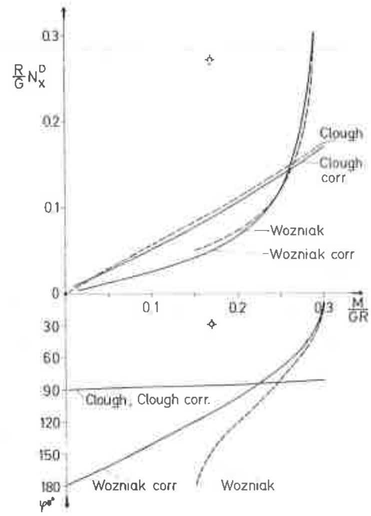

Fig. 2 Contact angle φ^* and maximum axial force N_x^D as functions of the wall moment M - Comparison between current uplift models and tilt test results.

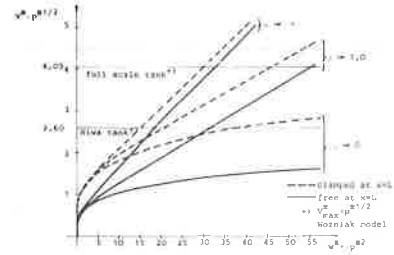

Fig. 3 Relation between uplifting force and uplifting hight for different boundary conditions and friction coefficients.

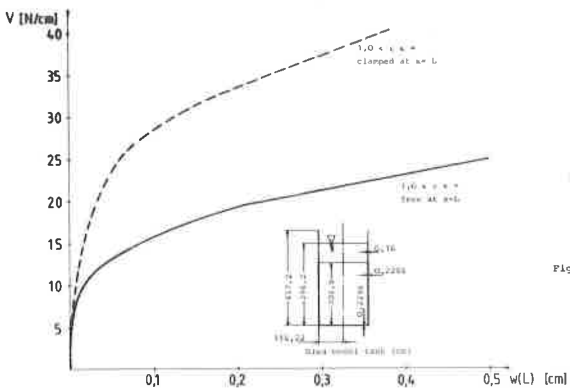

Fig. 4 Uplift Vorce V as a function of edge uplift hight w(L) for Niwa 's model tank /10/.

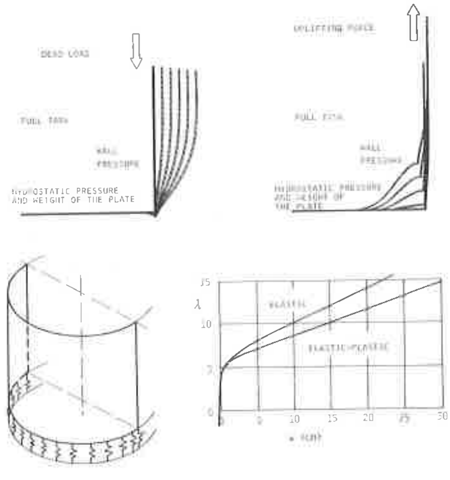

Fig. 5 Axisymmetric model and results of the nonlinear finite element analysis of the effective spring characteristic - full scale tank. (λ = 1 corresponds to the dead load of the empty tank)

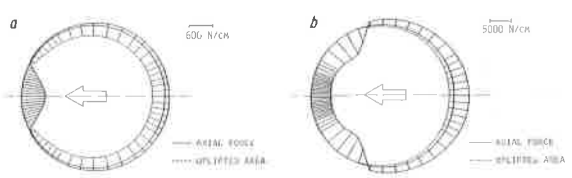

Fig. 6 Axial membrane force and uplifted bottom plate area
a) Niwa's model tank; El Centro, A = 0,25 g maximum ground acceleration
b) full scale tank, USAEC spectrum, A = 0,15 g maximum ground acceleration

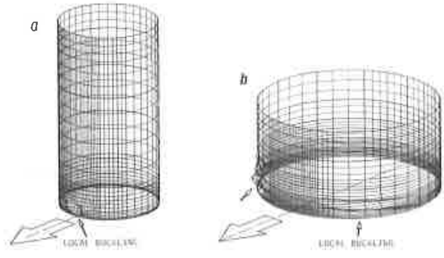

Fig. 7 Fundamental buckling modes
a) Niwa's model tank - central local buckling
b) full scale tank - excentrical local buckling