Influence of anchor behaviour on the earthquake response of liquid storage tanks

Politopoulos, I.
CEA - C.E./SACLAY, DRN/DMT/SEMT/EMSI, Gif-Sur-Yvette Cedex, France

1 INTRODUCTION

The dynamic response of thin liquid storage tanks to earthquakes is a very complicated phenomenon, because it can be highly non linear. Among others, one can meet material and geometric non linearities of the tank shell leading eventually to static or dynamic buckling, non linear behavior of anchor bolts, contact non-linearities due to the uplift of the tank base and to the unilateral character of the fluid pressure on the shell and high amplitude fluid oscillations. Moreover, linear or non linear soil structure interaction affects considerably the response of the fluid structure system under consideration.

In this paper we focus our attention on problems related only to the base uplift and anchors plastification. We study a tank similar to the Hualien project tank, but we neglect the soil structure interaction. The studied tank is representative of medium height to radius ratio tanks with relatively thick bottom plate. The contact is simulated via a simple discrete penalty method in order to facilitate the calculation of the impact forces. Modal coordinates calculated for various Fourier harmonics are used for the dynamic analysis and the coupled modal equations of motion are solved with an explicit time integration algorithm.

Obviously, this approach is less precise than a direct finite element analysis on the nodal basis but is less expensive. The scope of this paper is to discuss the efficiency of the proposed method to deal with problems like those aforementioned and to give some qualitative results concerning the influence of anchor bolts behaviour on the earthquake response of tanks.

2 MODE SUPERPOSITION METHOD

The mode superposition method is widely used for linear dynamic analysis. In non linear dynamics its application is less frequent, because in this case, the equations of dynamic equilibrium remain coupled by the non linear terms. However, the method can be very efficient even for some classes of non linear problems (Bathe et al [1]). In most of the successful approaches the semidiscretized equations of motion are written in the form:

$$M\ddot{X} + C\dot{X} + KX = F_e + F_{NL}$$
in which, \( M, K \) and \( C \) are the mass, stiffness and damping matrices corresponding to the initial state, \( X \) is the vector of nodal displacements, \( F_e \) is the vector of equivalent nodal forces which are given time functions and \( F_{NL} \) is a vector of nodal forces accounting for all the non-linearities. Modal superposition involves a coordinate transformation from nodal coordinates to modal ones. The new basis is formed by the eigenvectors corresponding to the initial state and it is kept constant during the whole computation.

The disadvantages of the method are that:

a) the eigenmodes of the system have to be calculated prior to the non-linear analysis,
b) operations of mode superposition are necessary at every time step in order to calculate the non-linear forces,
c) it is not always easy to define an apriori criterion for mode truncation and a posteriori estimate of the error due to this truncation. Static contribution of the neglected eigenmodes has usually a beneficial effect, but it leads to iterations within each time step, suppressing the simplicity of explicit time integration algorithms.

The advantages are that:

a) a reduced number of degrees of freedom can be used for the non-linear analysis,
b) the diagonal consistent mass matrix in combination with the frequency truncation enables the use of an explicit time integration with a time step not extremely small. This simplifies considerably the consideration of complex non-linearities, such as discontinuities due to contact problems.

The modal basis is calculated using finite elements for the fluid and the structure. There are no kinematic boundary conditions imposed on the tank, because we made the choice to consider the non-linear forces as completely exterior forces. The decomposition in Fourier series is used in order to reduce the problem size and to avoid frequently meted difficulties in the solution of the eigenvalue problem of this type of structures, due to the existence of many eigenfrequencies in small frequency ranges. The following eigenmodes are calculated:

- eigenmodes for Fourier number \( N = 0 \) which are the only ones excited by the axisymmetric static loading (mainly hydrostatic pressure), up to 200 Hz (34 modes),
- eigenmodes for \( N = 1 \), which are the only ones excited by the d'Alembert's inertial forces, up to 600 Hz (75 modes),
- eigenmodes for \( N = 2 \), which may be excited by the non-linear forces, up to 100 Hz (18 modes).

Eigenmodes for higher Fourier numbers are not calculated because the scope of this work is not an exhaustive quantitative analysis but a qualitative one.

In order to calculate easily the non-linear forces, the 2D modal basis is extended to a 3D one, after generation of a 3D mesh and 3D displacement eigenvectors.

3 IMPACT AND ANCHOR MODELING

The more usual methods for imposing the unilateral kinematic constraints, related to the contact problems, are Lagrangian multipliers method and penalty methods. The Lagrangian multipliers lead to a theoretically "exact" fulfillment of the constraints, but the resulting matrices need a special numerical treatment. Moreover, in the case of explicit algorithms, they result to matrices which are not always diagonal, especially when modal superposition is used, because even a constraint applied to only one node implies a relation between several modal coordinates. On the other hand, penalty
methods are less precise but they are simpler, especially in connection with explicit algorithms. In some cases, some physical interpretation can be given to the penalty functions, relating the partial relaxation of the kinematic constraints to phenomena which cannot be represented by the model used. For example, the finite element mesh used, or the truncated modal basis, or even the structural model itself, underestimates the flexibility of the impacting structures, so, in some cases, a judicious choice of the penalty functions, accounting for this neglected flexibility, can improve the model behaviour.

In our approach we use a simple penalty method just as a numerical approach without any precise physical consideration. We consider that the interface between the tank base and the foundation is an elastic medium similar to a Winkler type soil, with a unilateral linear elastic behaviour. We do not use a consistent approach for the determination of the reaction forces and we restrict ourselves to a discrete and not discretized method, attributing to each node a spring, having a stiffness proportional to the surface around the node. When a node penetrates in the foundation the corresponding reaction is proportional to its penetration while the reaction is zero during the uplift phase. The value of the interface stiffness is chosen to give in the linear case a first cos θ type eigenfrequency near the first cos θ type frequency of the tank fixed at its base.

The anchors are modelled by unilateral elastic perfectly plastic springs. The gap between the foundation and the bolt head is updated according to the residual plastic deformation (displacement) of the spring. The estimation of the spring elastic stiffness is not a trivial task because it depends on the "effective length" of the spring. In reality the anchoring mechanism in the concrete is a highly non linear and complicated phenomenon and its study is out of the scope of this work. For our simulations we used two extreme values of spring stiffness: one based on an effective length equal to the distance between the anchor hook and the anchor head ("flexible" anchors) and another based on an effective length ten times smaller, which is slightly superior to the thickness of the bottom plate ("stiff" anchors).

As we mentioned earlier, we do not consider material or geometric non-linearities of the tank shell, focusing our attention on anchors plastification and base uplift. However, during the uplift phase, second order effects should be considered even for small absolute displacement amplitude if the bottom plate is very thin exhibiting considerable gradients of displacement. The Hualien project tank has a rigid bottom plate (20 mm), so we carried out a small deformation analysis.

4 NUMERICAL RESULTS AND DISCUSSION

The geometric and material characteristics of the Hualien tank can be found in Korean group [2]. Prior to the earthquake analysis, some cases of responses to low level loads, leading to a physical linear behaviour have been compared with linear analysis of the same system. This gives an indication of the accuracy of the model. Generally, the model gives good results concerning the response frequencies and displacements but stresses are not calculated satisfactorily. As an example, in figure 1, the axial stress distribution in the tank shell, due to a static horizontal loading, is compared for a linear static model and for the non linear model under pseudostatic loading. We can clearly see the poor performance of our model. This problem, concerning the accuracy of stresses or other displacement derivatives calculated by a truncated modal basis, is well known and in linear dynamics a correction accounting for the static contribution of the
neglected modes is frequently used with success. But, as we mentioned earlier, in non linear dynamics, these correction terms complicate considerably the explicit time integration and they have not been taken into account in this phase of our work. However, we notice that we can have a very good estimate of the axial stress near the tank bottom, using the vertical reaction forces of the foundation springs on the perimeter of the tank. It is this value which will be used hereafter as axial stress.

The normalized pseudoacceleration spectrum of the input accelerogram, for a 5 % damping, is shown in figure 2. Simulations for peak ground horizontal accelerations 0.3 g and 0.6 g have been performed with both "stiff" and "flexible" anchors, taking into account eigenmodes for Fourier numbers 0 and 1. No viscous damping is considered, the main mechanism of dissipation being the plastification of anchors.

The system is very sensitive to very small changes of its parameters, indicating some germs of chaotic behaviour. In figure 3, the plastic displacements of an anchor are compared for two almost identical simulations, differing only slightly in the initial displacement of the first cosθ type mode. We can clearly see a divergence which seems small, but it must be compared with the initial perturbation which is 1x10^-4 times the maximum response of the perturbed eigenmode. Simulations with elastic anchors without plastification indicated a more stable behaviour.

Simulations including harmonics 2 have also been performed. In figure 4, we can see that the contribution of modes with N = 2 becomes very important after some time (6.5 s). As we can observe in figure 5 this time is the time corresponding to the onset of an important plastic displacement of the anchor. On the other hand, if elastic anchors are considered the importance of harmonics 2 is decreased considerably. This is true for all significant quantities like displacements, plastic deformation of anchors and stresses in the tank shell. It can be explained by the fact that a high density of stiff elastic anchors reproduces almost linear axisymmetric boundary conditions.

The values of maximum vertical axial stresses in the shell near the bottom are listed in the table 1. A linear analysis of the same tank gives a stress value of 2.5x10^8 Pa for a ground acceleration of 1 g. Comparison with the results of table 1 shows that the "non linear" stress in the case of stiff anchors is much higher than the "linear" one. For high level of peak ground acceleration (0.6 g), we remark the advantageous behaviour of the flexible anchors which give much smaller displacements and stresses than the stiff anchors (figures 6 and 7). This is due to the important plastic deformations of the stiff elastoplastic anchors, which leads to an important redistribution of stresses on the tank base perimeter.

5 CONCLUSIONS

The mode superposition method is applied to the earthquake analysis of the Hualien project tank. This enables an analysis considerably less expensive if compared to a direct finite element analysis. Non-linearities due to the plastification of the anchors and to the tank base uplift are taken into account. There is some progress to be done in order to ameliorate the accuracy of the method, especially concerning stress determination, but the proposed method seems to be capable of describing the most important phenomena relevant to an earthquake response.

Uplift non-linearities are satisfactorily represented by Fourier harmonics N = 0 and N = 1 in the case of elastic anchors, at least for "moderate" peak ground accelerations (0.3 g). Anchors plastification amplifies the non linear character of the
response. It increases the contribution of higher Fourier harmonics and seems to amplify the chaotic behaviour of the system.

In the case of stiff anchors, stresses exceed considerably the linearly calculated stresses. This can probably result to an increase of buckling risk. Flexible anchors exhibit a much more satisfactory behaviour for high peak ground acceleration (0.6 g). It would be interesting to examine the validity of this conclusion with parametric studies accounting for material and geometric non-linearities.

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REFERENCES

<table>
<thead>
<tr>
<th>Type of anchors</th>
<th>0.5 g</th>
<th>0.6 g</th>
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<tbody>
<tr>
<td></td>
<td>N = 0, 1</td>
<td>N = 0, 1, 2</td>
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<tr>
<td>Stiff</td>
<td>1.12 x 10^8</td>
<td>1.70 x 10^8</td>
</tr>
<tr>
<td>Flexible</td>
<td>1.10 x 10^8</td>
<td>1.12 x 10^8</td>
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Table 1 - Maximum axial stress in the tank shell

![Fig. 1: Axial stress distribution for linear response](image1)

- Linear static model
- Non linear pseudostatic model

![Fig. 2: Normalized pseudoacceleration spectrum for 5 % damping](image2)
Fig. 3: Maximum anchor plastic displacement
(Ag = 0.3 g/stiff anchors)
--- perturbed motion --- fundamental motion

Fig. 4: Maximum base vertical displacement
(Ag = 0.3 g/stiff anchors)
--- N = 0, 1, 2 --- N = 0, 1

Fig. 5: Maximum anchor plastic displacement
(Ag = 0.3 g/stiff anchors)
--- N = 0, 1, 2 --- N = 0, 1

Fig. 6: Horizontal displacement near the top
(Ag = 0.6 g/N = 0, 1)
--- stiff anchors --- flexible anchors

Fig. 7: Axial stress at the bottom edge
(Ag = 0.6 g/N = 0, 1)
--- stiff anchors --- flexible anchors