NUMERICAL ANALYSIS OF THE SEISMIC RESPONSE OF UNDERGROUND STEEL STORAGE TANKS

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ABSTRACT

The seismic behaviour of underground structures is generally satisfactory compared to the above-ground ones. However, despite of the buried structure confinement, in recent strong seismic events some underground structure as tunnels, pipelines and tanks suffered high damage; for this reason, a strong effort is focused to evaluate the seismic vulnerability of these structure and to give codes instruments to the designers. Concerning the underground water and oil storage tanks, in literature there is no indication on their seismic vulnerability.

Starting from the previous experiences in the seismic analysis of above ground industrial equipments and in the seismic analysis of tunnels in soft soils, an integrated geotechnical and structural approach to underground industrial equipment is under development. In particular, attention is focused on the influence of fluid-structure and structure-soil interactions on the overall dynamic response.

In this paper, an overview of case histories of post-earthquake damages of underground tanks is reported. Design guidelines and construction process are analyzed. Preliminary seismic analysis of sample underground steel tanks are carried out under simplified assumptions and described in order to point out key aspects of the problem.

Results are reviewed in order to identify critical issues of the soil/structure/fluid interaction during the seismic event. An introduction to advanced numerical codes able to perform reliable interaction analyses using real strong motion records is given.

Keywords: FEM, Numerical analyses, Soil-structure interaction, Steel tanks, Underground structures

INTRODUCTION

For a long time it was a general belief that earthquake effects on underground structures is not very important. This is because these structures have generally experienced a low level of damage in comparison to the surface engineering works. Nevertheless, some underground facilities were significantly damaged during recent strong earthquakes (Hashash et al., 2001).

In modern urban areas, underground space has been used to locate a wide range of under-ground structures, both for civil and industrial use. Most underground structures are essential to human life and it is very important to study how these structures are damaged during earthquakes to protect the service efficiency and avoid any possible risk to the human life. Moreover, when the structures are used for liquid or for liquefied substances transport and storage the interaction of the structure and the soil with the fluid must be taken into account (sloshing waves).

In this paper, an overview of case histories of post-earthquake damages of underground tanks is firstly

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reported. Design guidelines and construction process are analyzed. Fabbrocino et al. (2010) reported preliminary seismic analysis of sample underground reference tank, in order to point out key aspects of the problem. Results are reviewed and updated here to identify critical issues of the soil/structure/fluid interaction during seismic events.

DAMAGE CASE HISTORIES TO SIMILAR STRUCTURES

Before 70’s very few data are available concerning damages to underground structures after earthquakes including underground tanks. Generally speaking, to understand the behavior of these structures, it is possible making references to reports written for similar structures like underground tunnels and pipelines and above-ground tanks.

Bardet & Davis (1998) reported many cases (61) of steel tubes, which were strongly damaged during the Northridge (1994) earthquake. They showed the mechanisms of deformation which are peculiar of thin steel tubes: in most cases they underwent a kind of shriveling due to lateral buckling for lack of confinement. Miyajima & Hashimoto (1999) studied the damages suffered by the water supply system during the same earthquake: the Water Works Association of the Chinese Republic estimated that around 50% of cracking was caused by soil shaking, and the other 50% was due to slopes failure and liquefaction occurred near the tubes. Lau et al. (1995) described the main damages suffered by lifelines in the city of Los Angeles during the Northridge earthquake. This report included data about the steel pipelines for water and gas, which suffer failures due to tension or compression cracks, buckling, shear rupture by landslide and fault (Figure 1).

Figure 1. Damage to steel pipelines: a) compression, b) tension cracks, c) buckling and d) shear failure (Lau et al., 1995)
Moreover, Lau et al (1995) reported some data of damage for above-ground steel tanks, for water, oil and gas storage, which included collapsed roof, rocking motions, in-plane buckling, fracturing of connecting pipes, elephant foot buckling and total collapse of the tank (Figure 2).

Figure 2. Damage to above-ground tanks: a) elephant foot, b) anchor pull out, c) uplift and d) total collapse (Lau et al., 1995)

CONSTRUCTION AND DESIGN ASPECTS

In the recent years, a research project was dedicated to the study of seismic behavior of some kind of underground structures in Italy. Numerical models of shallow tunnels were carried out (Bilotta et al., 2007) to improve the simplified formulation for the seismic design of these structures. Experimental data, moreover, were obtained from dynamic centrifuge models of tunnels (Lanzano, 2009), to create some “artificial” case histories. The final scope was to calibrate the simplified tools for the designers, as the closed-form solution for the seismic increments of the internal forces in the tunnel cross section (Wang, 1993). Starting from this previous experience the next study on underground tanks was planned.

Structures present in industrial plants are extremely complex. Great efforts are required to ensure the safety of the operations because of the large quantities of toxic and flammable contents often stored inside. Risk assessment related to these facilities becomes a key issue and tools for designing prevention and mitigation measures are necessary (Lees, 1996). In this regard, the structural performance of buildings, components and equipment are certainly of primary interest, when natural hazards such as earthquakes, are concerned. In recent years, the quantitative risk analysis (QRA) procedures, originally developed for nuclear power plants have been extended to other local industrial plants (Di Carluccio et al., 2006), i.e., petrochemical, chemical plants or facilities for the storage of hazardous materials (Salzano et al. 2007, 2009). To this end, the main classes of industrial equipment have been drawn (Di Carluccio et
al., 2007; Fabbrocino et al., 2007): in particular, a detailed study on the common industrial layout has been conducted with particular attention to the storage facilities. Products or feeding of industrial processes are generally stored in steel tanks. The evaluation of risks due to the interaction of earthquake with equipment is mainly related to the evaluation of risks produced by structural damage of tanks, which can be in turn the basis for evaluation for the economical losses, provided the cost of damage repairs is given. Due to the hazard of storage tank or reservoir, it is common use to bury or mound storage tanks (Figure 3) if possible, in order to avoid any escalation effects in the case of fire, and any damage to the shell of equipment (underground gas and vapor reservoir or tank).

Figure 3. Typical layout of underground tanks of industrial use

Table 1. Standard geometry for underground tanks

<table>
<thead>
<tr>
<th>Volume (m³)</th>
<th>Diameter (m)</th>
<th>Total Length (m)</th>
<th>Thickness of cylinder (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>3.2</td>
<td>25.8</td>
<td>14</td>
</tr>
<tr>
<td>420</td>
<td>4.0</td>
<td>34.8</td>
<td>18</td>
</tr>
<tr>
<td>500</td>
<td>4.0</td>
<td>41.2</td>
<td>19</td>
</tr>
<tr>
<td>1000</td>
<td>5.5</td>
<td>44.0</td>
<td>24</td>
</tr>
<tr>
<td>1500</td>
<td>6.0</td>
<td>55.0</td>
<td>26</td>
</tr>
<tr>
<td>2000</td>
<td>6.6</td>
<td>60.0</td>
<td>30</td>
</tr>
<tr>
<td>3000</td>
<td>7.4</td>
<td>72.1</td>
<td>33</td>
</tr>
<tr>
<td>4000</td>
<td>7.8</td>
<td>86.4</td>
<td>34</td>
</tr>
<tr>
<td>5000</td>
<td>8.0</td>
<td>102.2</td>
<td>35</td>
</tr>
</tbody>
</table>

NUMERICAL ANALYSES

To understand the behaviour of underground tanks under seismic loading a set of numerical analyses were carried out using LS-Dyna (Livermore, 2003), a finite element code which integrates the dynamics equations in the time domain. The numerical model was in plane strain conditions (2D), considering the cross section of underground steel tank (Figure 3). This scheme was reasonable for a simple analysis on the middle section of the tank.

Three groups of analyses were carried out: free-field analyses; single tank analyses; two tanks analyses. In the following parts the three analyses types are described.
Free-field analyses
The free-field analyses are carried out without the structure and any soil modification, in order to study the soil response alone to the input motion. The initial thickness of the soil layer, before the construction of the underground tank, was considered \( H = 30 \text{ m} \) and \( H_T = 39 \text{ m} \) where the structure was positioned. Therefore, as in the common practice for the underground tanks, the structure was partially buried and recovered by a soil layer (Figure 3).

Figure 4. 2D Calculation domain for free-field dynamic analyses (ff1)

to study the free-field behaviour, the first analysis was carried out considering a soil layer with a constant thickness of \( H = 30 \text{ m} \) along the entire numerical domain (ff1) (Figure 4). The soil layer has increasing value of shear wave velocity with depth (Figure 5), but a single value of damping ratio is considered (\( D = 1\% \)). According to the Italian (NTC, 2008) and European codes (EN 1998, 2003), the average shear wave velocity is \( V_{s,30} = 241 \text{ m/s} \), which corresponds to a Class C soil. This stiffness profile was assumed in also all the subsequent analyses.

Figure 5. Profiles of shear wave velocity along the soil layer (Class C)

Figure 6. Acceleration time history and Fourier spectrum of the Tolmezzo recording of the Friuli earthquake (1976)

The numerical analyses were performed using a visco-elastic model for the soil, considering the input parameters showed in the Table 2. The employed numerical domain was large enough not considering the influence of the lateral boundaries on the soil response in the middle of the model. The total length of the model was ten times larger compared to the thickness of the layer (\( L = 10 \text{ m} \); \( H = 300 \text{ m} \)). For this reason,
no adsorbent boundaries were used at the model lateral frontiers. Under the deformable soil layer was considered rigid reflecting bedrock, where a time history of horizontal acceleration was applied.

### Table 2. Input material parameters for the analyses

<table>
<thead>
<tr>
<th>Material parameters</th>
<th>thickness (m)</th>
<th>E (MPa)</th>
<th>$\rho$ (kN/m$^3$)</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil layer 1</td>
<td>1</td>
<td>78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil layer 2</td>
<td>2</td>
<td>114</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil layer 3</td>
<td>2</td>
<td>162</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil layer 4</td>
<td>5</td>
<td>212</td>
<td></td>
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<td>Soil layer 5</td>
<td>5</td>
<td>290</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil layer 6</td>
<td>5</td>
<td>334</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil layer 7</td>
<td>10</td>
<td>400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tank (steel)</td>
<td>0.03</td>
<td>210000</td>
<td>77.11</td>
<td></td>
</tr>
</tbody>
</table>

The input signal for the analyses was the acceleration time history of the 1976 Friuli earthquake recorded at Tolmezzo site. In Figure 6 the acceleration time histories and the Fourier spectrum are reported. This input time history was chosen as a preliminary example.

The finite element mesh was built using rectangular elements; the average value of mesh dimension was 0.5 m near the structure (obviously not considered in these analyses) and 1 m in the rest of the model. These values were compatible with a correct propagation of the waves inside the model due to the input signal; therefore, the maximum frequency, which is correctly propagated inside the model, is comparable to the frequency content of the natural acceleration time histories, which is shown in Figure 6 (Lysmer & Kuhlemayer, 1973):

\[
f_{\text{max}} = \frac{V_s}{k \cdot d} = 20 \text{Hz}
\]

in which $d$ is the maximum size of the mesh (1 m), $V_s$ is the minimum value of shear wave velocity (133 m/s) and $k$ is a non-dimensional parameter, which varies between 4 and 10 (it was 6 in this case). The damping used in the analyses was frequency independent.

In order to evaluate also the soil amplification due to the “small hill” built to partially bury the tank inside the soil, another free-field model (ff2) was carried out considering a modified shape of soil surface, but again without the structure inside (Figure 9). The numerical analyses of the free-field models (ff1 and ff2) were compared to the results of 1D classical wave propagation performed by EERA (Bardet, 2000), a free-field numerical codes which integrates the dynamic equations in the frequency domain. Considering the features of the EERA codes, only analyses similar to the free-field ff1 could be carried out. The results of the analyses are showed in Figure 7, in terms of profiles of maximum acceleration, shear stress and shear strain with depth.

The comparison between the free-field analyses shows a good agreement between the LS-Dyna and EERA analyses for the model ff1, especially in terms of acceleration. Just only the modification of the horizontal soil surface determined a strong amplification of all the plotted parameters along the “small hill” (thus determining a kind of topographic effect).
The geometry of the model was initially built considering a single structure with circular cross section D=8.0 m and a steel lining s=0.03 m, according with the standard geometry of the underground tanks (Table 1). Two numerical analyses with single tank were performed: the first one considered only the soil structure interaction (ss) (Figure 8); the second analysis modeled also the fluid inside the tank (ssf).

The soil/structure/fluid interaction analyses were performed to account of the sloshing wave effect on the internal forces on the tank structure. In Figure 9 the particular of the structure was showed for the fluid analysis: generally the 70% of underground tank are filled with the storage fluid during the normal operation of the structure.

Also the material model used for the structure was visco-elastic. The input parameters for the tank, assumed made of steel, are showed together with the soil parameters in the Table 2. The material properties for the soil around the tank corresponds the softer material (i.e., layer 1). The interface between the soil and the structure was rigid, i.e., the relative displacement between soil and tank was not permitted. The material model for the fluid was the Null Hydrodynamic model: this material model allows an equation of state, which was a polynomial one in this case, without computing the deviator stresses. This is a first approach to the modeling of stored substances. Refinements of the model require the selection of appropriate material representation for typical liquefied gas and careful evaluation of effects of internal pressure on the steel component.
In Figure 10 the profiles of maximum acceleration, shear stress and strain along depth are showed for both the interaction analyses. These profiles were compared to the results of the analyses of the free-field model 2 (ff2).

The two interaction analyses had similar values along the whole depth, which was similar to the behavior of ff2. The main reason of the strong increase of the showed parameters near the tank was the modification of soil horizontal profile, which determined a low level of confinement around the structure. The presence of the structure also determined a local increase of the plotted parameter in proximity of the tank. In the Figure 10 also the maximum rackings of the tank cross section were plotted, which were similar to the value of shear strain in free-field condition; probably the relative stiffness between soil and structure had a value which reproduced the seismic propagation in free-field conditions.

For the interaction analyses, the maximum value of the dynamic internal forces on the lining is also showed (Figure 11), compared with closed-form solutions (Wang, 1993), which give the maximum value of the increments of bending moment and hoop forces due to pseudo-static deformation of the lining cross section buried in an elastic space:
In the previous equations $E_s$ and $\nu_s$ are the elastic modulus and the Poisson ratio of the soil layer around the tank, $R$ is the radius of the cross section, $\gamma_{max}$ is the maximum shear strain due to the earthquake and $K_1$ and $K_2$ are non-dimensional parameters depending on the relative stiffness and the slippage conditions between soil and structure. In this case, the internal forces were calculated using in the (2) and (3) the maximum shear strain in free-field conditions ($\gamma_{ff}$) ($\gamma_{max}\sim1.33\%$ as showed in Figure 10), for both the analyses ($ss$ and $ssf$).

The numerical distribution of the internal forces was not symmetric, but depended on the earthquake time history. The comparison between numerical and analytical values of internal force showed a good agreement, especially for the bending moment; the values obtained from the analytical formulations were generally conservative compared to the numerical results for a common value of the maximum shear strain. The difference between the $ss$ and $ssf$ analyses were recognized only for the bending moments: for a range of $\theta$, the values of the $ssf$ analysis were higher than the corresponding $ss$ analysis. This angle interval corresponded to the arc of circle in which the structure was in contact with the storage fluid inside. This consideration could indicate an effect on bending moment in the dynamic interaction with the fluid.

**Two tanks analyses**

Generally the underground tanks are not built as a single structure, but as a set of structures, as previously displayed in Figure 3. In order to account the interaction between two adjacent chambers, a set of interaction analyses were carried out considering two underground tanks. In total three types of analyses were carried out: two empty tanks ($ss^2$); one empty and one full tank ($ssf^2$) (Figure 12); two full tanks ($ssff$).

Also in this case, the results were plotted in terms of seismic amplification along the soil layer (Figure 13). The profiles were graphed only for the vertical 1 (Figure 12), because the soil responses to the earthquake is equivalent for the two vertical 1 and 2, passes along the two different tanks. The difference between the results of the different analyses with two tanks was small and, also in this case, the presence
of fluid inside the tank did not modify clearly the soil response. Moreover, the difference with the single tank analyses was also moderate, showing a small increase of shear strain and stresses around the tanks.

![Figure 12. Particular of the cross section of the model ssf2](image)

![Figure 13. Profiles of a) maximum acceleration, b) shear strain and c) shear stress with depth for the two tanks models (ss2, ssf2 and ssff)](image)

The dynamic internal forces were separately plotted in Figures 14 and 15, considering the two different tanks modelled in the analyses. The results were compared with the internal forces obtained from the single tank analysis without fluid. The bending moments showed a strong increment for some values of the θ angle, which corresponded to the parts of the tanks which were mutually adjacent (around 0° for tank1 and around 180° for tank2). No clear modifications due to the effect of fluid inside the tank were recognized for the bending moment: considering the observation done in the previous section, this matter is now under further investigations. Also in this case, there were no clear modification of hoop force between the different analyses and compared to the single tank results.
CONCLUSIONS

The paper carried out a simplified numerical study on the seismic behavior of underground steel tanks. A few data of damage to underground tanks are available in literature, but some information could be obtained from the observation of steel pipelines and aboveground structures, which were severely damaged during recent strong earthquakes. The design considerations for these structures include the sloshing effects of liquid inside the tank and the interaction with surrounding soil. According with the standard geometry for underground tanks, a set of numerical models was carried out in plane strain conditions, including free-field, single tank and two tanks models. The finite element model was set in order to obtain a correct propagation of an acceleration time histories, from the rock bedrock to the soil surface. The medium was visco-elastic and had increasing stiffness with depth. Also, the fluid inside the tank was modeled in the analyses in order to evaluate the dynamic effects on the internal forces of the tank. The numerical analysis gave reliable results in terms of maximum acceleration, shear strains and stresses, despite of the simplification adopted in the calculation. Same consideration could be carried out for internal forces, which were compared with analytical expression. Next steps for numerical analyses included the use of a more complex material model for soil, in order to take account of the cyclic degradation of the soil stiffness and damping, as well as the introduction of internal pressure in the tank and the improvement of numerical models for fluids depending on the characteristics of stored material.
REFERENCES


