ABSTRACT

This work presents as its main objective the analysis and structural design of a reinforced concrete cylindrical tank, originally conceived in the middle of the last century. The most relevant aspects in light of the structural Eurocodes are covered, which include specific regulations that apply to reservoirs.

The main durability requirements in order to ensure proper operation of these constructions over their lifetime are addressed. It is described how to quantify indirect actions, through a temporal evaluation of the joint effect of concrete shrinkage and creep, by applying the effective modulus method.

The structural behavior of cylindrical shells in presence of static forces is analyzed in detail, and simplified calculation methods are introduced. Using the finite element method, it is further assessed the influence of the deformability of the soil on the stresses generated in the structure.

It is performed a seismic analysis of the tank, in which the distribution of the hydrodynamic pressure on the walls are found, and different forms for combining the pressure components are examined. In addition, based on the finite element method, a modal analysis of the structure is carried out and the seismic stresses are attained.

The structure is designed based on the provisions included in the relevant Eurocodes, wherein the criteria for limiting crack width in tanks stand out. A reinforcement solution for the original structure’s geometry is found considering a rigid homogeneous foundation soil.

Lastly, a non-linear analysis was conducted on a generic frame corner with different reinforcement detailing, in order to assess its efficiency in terms of ductility and resistance. Further in, the same type of analysis is applied to the node joining the wall and the slab bottom of the tank.

Keywords: Cylindrical tanks, Reinforced concrete, Structural analysis, Structural design, Seismic analysis, Non-linear analysis

1. INTRODUCTION

Reservoirs are liquid, gas or solid retaining structures that may be conceived in a wide range of structural solutions, since there are several types of different applications that possess different requirements, which can be in terms of contained substance aggressiveness, operational conditions, size, or site conditions.

In Portugal, it has been common practice to use reinforced or prestressed concrete in liquid retaining structures (tanks), namely in water supply systems and waste-water treatment plants, due to its lower cost. Reinforced concrete can provide long life with low maintenance costs, but only if appropriately designed and constructed. However, steel structures have advantage when there is a need for complete tightness.

This work’s main objective is to cover the most relevant matters of analysis and design for ground reinforced concrete cylindrical tanks, by studying a
tank originally conceived by Santarella [1] (Figure 1), in light of the new regulations (Eurocodes).

This tank has a full capacity of 1113 m³, by possessing a 8,2 m radius, and a maximum depth of 5,4 m. The ground slab, walls and dome have, respectively, 0,1 m, 0,2 m and 0,08 m thickness. The retained liquid is mineral oil ($\gamma = 9,5 \text{kN/m}^3$).

In order to define the durability requirements, it is first necessary to identify the aggressive substances and its transportation mechanisms, and the reactions involved in deterioration.

The Portuguese norm Esp. LNEC E 464 [2] classifies environmental actions into six categories (Table 1), based on type and severity of exposure that the construction will experience.

<p>| TABLE 1 - EXPOSURE CLASSES |</p>
<table>
<thead>
<tr>
<th>Description</th>
<th>Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No risk of corrosion or attack</td>
<td>X0</td>
</tr>
<tr>
<td>Corrosion induced by carbonation</td>
<td>XC1 / XC2 / XC3 / XC4</td>
</tr>
<tr>
<td>Corrosion induced by chlorides</td>
<td>XD1 / XD2 / XD3</td>
</tr>
<tr>
<td>Corrosion induced by chlorides from sea water</td>
<td>XS1 / XS2 / XS3</td>
</tr>
<tr>
<td>Freeze/Thaw attack</td>
<td>XF1 / XF2</td>
</tr>
<tr>
<td>Chemical attack</td>
<td>XA1 / XA2 / XA3</td>
</tr>
</tbody>
</table>

Based on this information, the same norm defines the protective measures that need to be taken, which are centered on minimum concrete cover over reinforcement and concrete mix. For tanks, this requirements might be higher, due to the impermeability needs. For example, British norm BS 8007 [3], that regulates design of RC tanks, states that concrete should possess minimum cement content of 325 kg/m³, maximum water/cement ratio of 0,55, or 0,50 if pulverized-fuel ash is present, and minimum characteristic cube strength at 28 days not less than 35 MPa.

In the tank studied, the ground slab was classified as exposure class XC2, while the remainder elements were set at class XC4. Because of the elevated slenderness of the slab and dome, they can only afford one set of reinforcement. The nominal reinforcement covers were established 3,5 cm at the walls and dome, and 4 cm at the slab.

3. ACTIONS ON TANKS

Actions on tanks are identified in Eurocode 1-4 [4]. Besides the actions that are accounted in most cases, there are some loads that apply specifically to tanks, such as hydrostatic pressure, suction on unloading due to inadequate ventilation, loads resultant from filling process, thermal loads from
liquid, or accidental loads (due to overfilling, or spills, for example).

Unlike other types of structures, tanks are very sensitive to cracking and therefore it is essential to correctly quantify actions that will be present in service limit states. Thus, the definition of indirect actions (imposed loads), such as temperature variation and concrete shrinkage and creep, achieve special relevance and are frequently determinant.

In tanks, the following situation should be accounted when considering thermal loads [5]:

- Thermal actions from climatic effect due to the variation of shade air temperature and solar radiation;
- Temperature distribution for normal and abnormal process conditions;
- Effects arising from interaction between the structure and its contents during changes (e.g. shrinkage of the structure against stiff solid contents);

The total shrinkage strain is composed mainly by the drying and autogenous components. The drying shrinkage is a function of the migration of the water through the hardened concrete and develops slowly over a several year period and it relates more to higher water/cement ratio concretes. On other hand, the autogenous shrinkage develops in the early days after casting.

Temporal evolution of shrinkage extension was calculated for the tank and is presented in Figure 2.

Under constant stress, concrete gradually allows for greater deformations, which can be measured as:

\[ \varepsilon_c(t, t_0) = \varphi(t_\infty, t_0) \cdot \left( \frac{\sigma_c}{E_c} \right) \]  \hspace{1cm} (3.1)

Where \( \varphi(t_\infty, t_0) \) is creep coefficient, that represents the ratio between the deformation at time \( t_\infty \) and at time of loading \( t_0 \).

Temporal evolution of creep coefficient was calculated for the tank, for two different loading times (Figure 3).

Because of effect of ageing of concrete, namely creep, in situations where a load is applied during large periods of time, the elastic modulus can’t be assumed as constant. For direct actions, this consideration doesn’t change much, as the stresses in structure counter directly the applied loads. However, stresses caused by imposed loads (e.g. shrinkage) are directly proportional to the material’s stiffness.

The effects of creep and shrinkage can’t be evaluated separately because they’re interdependent: Shrinkage origins creep that diminishes its effect over the structure.

For a more rigorous analysis on the evolution of total deformation, the effective modulus method can be used, based on the equation:

\[ \varepsilon_c(t, t_0) = \frac{\Delta \sigma_c(t, t_0)}{E_{c,\text{eff}}(t, t_0)} \]  \hspace{1cm} (3.2)

\( E_{c,\text{eff}}(t, t_0) \) can be calculated by equation (3.3).
$E_{c,\text{adj}} = \frac{E_c}{1 + \chi(t, t_0) \cdot \phi(t, t_0)}$ (3.3)

$\chi(t, t_0)$ represents the ageing coefficient, and assumes values less than one. This parameter relates the deformation caused by a load that grows concomitantly with creep, such as shrinkage, with the deformation caused by a constant load. Its precise value can be obtained in the work of Bazant [6] but generally 0.8 is a good estimate.

Based on this method, the resulting stress from shrinkage in the tank, for a fully restrained element, was obtained (Figure 4). The maximum occurs at around 3000 days, where $\sigma_{ct}=3.2$ MPa, $\varepsilon_{ct}=0.36\%$ and $E_{c,\text{eff}}=8.89$ GPa. Because all this values maintain practically unchanged, the shrinkage can be obtained at infinite time (it’s easier to calculate), and the stresses can be calculated applying an elastic modulus of $E_c/3$. This observations combine with those of Camara & Figueiredo [7]. According to the same authors, the uniform temperature variation, that take place slowly during months’ time, can be calculated with $E_c/2$. This value was also applied in uniform temp. variation to account for same cracking due to its high value (Table 2).

<table>
<thead>
<tr>
<th>Load</th>
<th>Value</th>
<th>$E_{c,\text{eff}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shrinkage</td>
<td>-30 °C</td>
<td>10 GPa</td>
</tr>
<tr>
<td>Uniform temp. variation</td>
<td>±20 °C</td>
<td>15 GPa</td>
</tr>
<tr>
<td>Differential temp. variation</td>
<td>±15 °C</td>
<td>15 GPa</td>
</tr>
</tbody>
</table>

Admitting that the ground slab is cast first, only half of the shrinkage load was applied in this element. Both temperature variation loads were applied solely on the top part of the structure, to account the 3 m high embankment (Figure 1) sheltering effect against solar radiation.

4. STRUCTURAL ANALYSIS

4.1. STRUCTURAL BEHAVIOR OF CYLINDRICAL TANKS

A cylindrical tank is an axisymmetric structure. If the load is also axisymmetric, then the same stresses will be generated along any meridian section of the tank.

The lateral deformation resistance of the wall, facing a hydrostatic load inside out, can be divided into two components:

- Resistance provided by a set of infinitesimal horizontal ring beams, that develop axial traction forces (hoop forces) ($N_h$);
- Resistance provided by a set of infinitesimal vertical cantilevered beams that develop shear force and bending moment ($M$ and $V$).

Logically, the cantilevered beams resistance depends on the stiffness of the connection between the wall and the ground slab. For a given height, the larger the radius of the tank is, more stresses are absorbed by the vertical cantilevers, because the sections get increasingly more plane. On other hand, the higher the tank gets, more stresses are driven into the ring beams, since the deformed shape of the cantilevers increase in height, in opposition to the ring beams, offering lower stiffness.

The last description represents the typical structure behavior of a fixed tank wall: predominance of horizontal axial forces in the middle and upper part of the wall, and predominance of bending in the lower part of the wall (Figure 5).
The configuration of the forces in the wall, in case of rigid connection, can change drastically depending on the behavior of the ground slab.

Assuming the vertical hydrostatic pressure is countered directly by the soil above, and consequently doesn’t generate any stresses, the main question relies on how does the system behave under the vertical axial loads and bending moments transferred from the walls.

On soft soils and stiff slabs, the soil’s reaction will be distributed along the entire slab (curve 1 on Figure 6). On other hand, having hard soils and relatively slender slabs, the slab will deform easily near the edges, which will generate a concentrated reaction by the soil near those areas (curve 2).

In general, more deformable soils will generate more stresses in the slab, as the reactions migrate to the center. This can easily be pictured by imagining the system inverted, where the walls act like columns.

In the tank studied, the slab is extended beyond the walls. In this case, another factor arises under deformable soil. In the situation where there is no embankment loads acting on the outside portion (during test phase), the vertical component of the hydrostatic pressure will cause displacements on the ground, which by turn will produce reactions under the entire slab. Consequently, the outside portion of the slab will act as a cantilever, generating large bending moments (Figure 7). When the tank is empty and the embankment is in place, the condition is reversed.

4.2. Modelling

The structural behavior of the cylindrical tanks are highly dependent on the type of connection between the wall and the ground slab that can be fixed, pinned or simply supported. In this work, the tank (Figure 1), having rigid connection, was evaluated through two FEM models, in which one of them the slab isn’t extended beyond the walls and has no roofing (model A). The software used was CSI’s SAP2000.

To evaluate the influence of deformability, each model was run under four different soil stiffness ($k_s$), modeled with Winkler springs (Table 3).

<table>
<thead>
<tr>
<th>Soil</th>
<th>$k_s$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely deformable</td>
<td>5.000</td>
</tr>
<tr>
<td>Deformable</td>
<td>40.000</td>
</tr>
<tr>
<td>Very stiff</td>
<td>200.000</td>
</tr>
<tr>
<td>Undeformable</td>
<td>2E10</td>
</tr>
</tbody>
</table>

The results were validated by comparison with the ones obtained from the Hangan-Soare method, and with the exact solutions provided by differential
equations deduced from elasticity theory (for fixed base wall only).

4.3. RESULTS

Due to limited space, only some of the results will be shown. Except otherwise stated (Figure 15), all data refers to the combination of dead load and hydrostatic pressure (PP+PH).

- Model A

It can be observed in the following figures that, as previously discussed, the soil’s deformability plays a major role on the stresses in both wall and ground slab, due to the rotation of the node linking the elements, while hoop force is less affected.

- Model B

When the slab is extended, the soil’s deformability influence over the stresses on the wall decays, because the node no longer experiences large rotation. On other hand, bending moment in the slab is greatly increased due to the difference on vertical loading between interior and exterior sections.

Lastly, it can be observed in Figure 15 that imposed deformations can have a big impact on overall wall stresses. The slab limitation on the walls contraction due to shrinkage causes large hoop forces on the bottom of the wall. The same effect happens between the bottom and top areas of the wall on account of only the top experiencing uniform temperature variation.
5. SEISMIC ANALYSIS

The dynamic response of cylindrical tanks to seismic loads is a complex problem that involves the interaction between the retaining liquid and the deformation of the structure and the ground itself. Therefore, several simplified mechanical models have been proposed in order to turn the seismic analysis simpler through the determination of equivalent static forces, based on some hypothesis.

One of the first models was presented by Housner [8], that assumes the tank being completely anchored at their base and the walls rigid. According to this model, under a horizontal acceleration of the ground, the liquid will produce hydrodynamic pressures that can be divided into two components:

1 – Impulsive component – The wall’s motion forces a fraction of the liquid to move combined with the structure. Because the structure is assumed rigid, the entire system undergoes the same acceleration that the ground.

2 – Convective component – The tank’s motion excites the liquid, generating oscillation at the surface, whose motion is independent from the remainder of the system. This portion exhibit its own vibration modes, which simpler ones are characterized by high vibrational periods, where only the first one is accounted for.

Thereby, globally this model resumes to two masses \( m_i \) and \( m_c \) rigidly connected to the tank at heights \( h_i \) and \( h_c \). The first one (impulsive) experiences the ground’s acceleration (together with the tank’s mass) while the second one (convective) experiences the spectral acceleration linked to its vibration period.

However, it has been shown later that assuming that the tank is rigid can be non-conservative. In fact, due to the tank’s bending flexibility, the impulsive liquid and tank’s joint acceleration should correspond to the natural fundamental vibration frequency of the system, which can be several times higher than the peak ground acceleration.

Therefore, recent models, including the one present in Eurocode 8-4 [9], introduce a third, flexible, component (Figure 16).

The maximum pressure distribution on the tank wall relative to the three components were calculated according to Eurocode 8-4 [9] (Figure 17). This distribution varies circumferentially according to function \( \cos(\phi) \). It was applied the response spectrum for most aggravating seismic action in Portuguese territory.

It should be noted that the convective component acceleration should be deduced using the elastic response spectrum, because it is assumed that the liquid doesn’t have any energy dissipation capacity. The other components should limit the behavior
coefficient to \( q=1.5 \) due to the tank’s low redundancy. The flexible component’s acceleration was calculated using the absolute response spectra instead of relative (to the ground’s movement) since there is evidence that within the normal frequency range where the liquid-structure normally lies in, the value of these accelerations are similar.

The three components were combined using the square root sum of squares rule (SRSS).

Introducing the response spectrum in the software, seismic induced stresses in the walls were obtained.

This stresses correspond to the impulsive-flexible component only, as the convective component is not accounted for in this procedure. It can be observed in Figure 20 that the seismic stresses (SIS) are relatively low when compared to the ones relative to dead load and hydrostatic pressure (PP+PH). This happens because all hydrodynamic pressure is applied in a single direction and most of the force is transmitted to the foundation by membrane (tangential) shear with only small vertical bending.

Due to the low seismic stresses, because in tanks quasi-permanent loads are close to maximum loads in ultimate state limit, and since high crack limitation take place in order to fulfill tightness requirements, it’s clear that the seismic combination of actions will not be limiting. This remarks correlate with observed damage resulting from earthquakes in RC tanks, consisting mainly in rigid body sliding occurrences and soil rupture.

The EC8 simplified method should therefore provide with enough information (basal shear force and overturning moment) to design the base anchorage and check for global equilibrium.

Logically, these conclusions don’t apply to steel tanks, in which ultimate state limit will be limiting, and in addition the flexible component force might be higher due to this type of tank’s higher vibrational period (the range of periods referred lays in the initial ascending curve of the response spectrum)

6. STRUCTURAL DESIGN
Eurocode 2-3 [5] introduces specific design provisions for liquid retaining tanks and classifies tanks based on its requirements for leakage.

The tank being studied classifies as tightness class 1, where according to the regulation, leakage should be limited to a small amount and some surface staining or damp patches are acceptable. In this class, any cracks which can be expected to pass through the full thickness of the section should be limited to \( w_{k1} \).

Where the full thickness of the section is not cracked, the provisions of Eurocode 2-1-1 (in which the cracking limits are based on exposure classification) apply. This condition is valid if at least 50 mm or 0.2 times the section thickness is permanently experiencing pressure at all times, under quasi-permanent loading.

The cracking limit \( w_{k1} \) is calculated as a function of the ratio between the height of liquid \( h_L \) and the element thickness \( t \). For \( h_L/t \leq 5 \), \( w_{k1} = 0.2 \text{ mm} \), and for \( h_L/t \geq 35 \), \( w_{k1} = 0.05 \text{ mm} \). For intermediate ratios a linear interpolation can be made (Figure 21).

![Figure 21 - Recommended Values for \( w_{k1} \)](image)

The dome, which is not in contact with the retained liquid, had its crack width limited to 0.3 mm, accordingly to its exposure class XC4. In the other elements, \( w_{k1} \) applied (Table 4).

<table>
<thead>
<tr>
<th>Element</th>
<th>( z ) (m)</th>
<th>( h_L ) (m)</th>
<th>( t ) (m)</th>
<th>( h_L/t )</th>
<th>( w_{k1} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall</td>
<td>3.4</td>
<td>2.0</td>
<td>0.2</td>
<td>10</td>
<td>0.175</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>4.0</td>
<td>0.2</td>
<td>20</td>
<td>0.125</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>5.0</td>
<td>0.2</td>
<td>25</td>
<td>0.100</td>
</tr>
<tr>
<td>Slab</td>
<td>-</td>
<td>5.4</td>
<td>0.1</td>
<td>54</td>
<td>0.050</td>
</tr>
<tr>
<td>Beam</td>
<td>-</td>
<td>0.1</td>
<td>0.4</td>
<td>0.25</td>
<td>0.200</td>
</tr>
</tbody>
</table>

All elements had their crack width calculated at every section by a MS Excel application that calculates, for a pair of axial force and bending moment, the extensions in steel and concrete and consequently its stresses, and hence is able, based on Eurocode 2-1-1 formulation, to calculate crack width. Therefore, reinforcement was attributed based on an iterative procedure.

Ultimate limit states were also verified and at no case was limiting. In fact, that wouldn’t be expected, because at service the loads have practically the same values and the cracking requirements limit stresses in reinforcement at around 50 to 90 MPa, which represents about 20% of design strength value for reinforcement steel.

It should be noted that the reinforcement was designed without altering the structure’s original geometry and doesn’t represent an optimum solution, which would imply increasing the thickness of both the bottom slab and the dome. Also, this design was possible by considering a very rigid homogeneous foundation soil (characterized by 200 MN/m³).

7. FINAL REMARKS

Reinforced concrete water retaining structures possess some unique characteristics because of high loads that withstand during most of their lifespan and the tightness requirements that is the major parameter on its design, by imposing high cracking limitations.

Because of these, indirect loads must be careful quantified and its effects properly analyzed.

The structural analysis involving different ground stiffness concludes that this is a very important parameter that can greatly increase stresses both in the ground slab and in the wall.

Lastly, it was found that seismic loads should generally not influence design of reinforcing steel, although its global effect on the links to the foundation can’t be ignored.

MAIN REFERENCES


