Differential foundation settlements and thermal loading sensitivity of a typical reinforced concrete cooling tower against phenomena of ageing

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ABSTRACT: The aim of this paper is to determine the main parameters influencing the geometrical evolutions measured on a reinforced concrete cooling tower erected in France twenty years ago. Nonlinear finite element analysis is carried out using multi-layered shell elements. The constitutive models for concrete and steel are based on the plasticity theory. The structure is analysed for different load combinations (dead load, thermal stresses and differential foundation settlements) in order to provide some answers to the interrogations concerning the origins of the damages observed on this structure.

1- INTRODUCTION

The R.C. cooling towers erected in France by the thermic and nuclear centers of Electricité De France are typically 150m high, with a base diameter of about 135m, a thickness about 20cm, and their weight about 50000 tons. Such a structure is difficult to construct accurately and imperfections in geometry of the tower shell can occur during the construction phase caused by the combination of shrinkage, cracking, creep and ring bending effects. Under operating conditions the structure is also subjected to accidental loads (differential foundation settlements, strong gales, etc...) and the thermal gradient may cause the concrete to crack vertically along the meridians. This cracking results in the reduction of the eigenfrequencies of the structure, shifts the dynamic system into frequency areas of larger spectral densities and thus increases the dynamic amplification with respect to wind. Moreover the cooling tower shell is exposed to bad weather condition and moist generated in the process of cooling hot water. These factors are affecting the materials properties and cause for instance the corrosion of reinforcement leading to the reduction of the load bearing capacity of the structure. Full nonlinear analyses [1-7] have shown that these shell structures subjected to dead weight and wind pressure do not buckle, but reach their ultimate load which is initiated by rapid propagation of cracks in the tensile zones (about 2.5 times the extreme wind load!). The influence of the geometric imperfections on the load bearing capacity of the structure depends strongly on the distribution of these imperfections, the amplitude of the radius deviation and the location of the bulges [8]. However for the cooling tower analysed in this paper, it has been demonstrated recently [9] that the geometric imperfections with maximum amplitudes of the order of 25 cm (which correspond to the minimal thickness of the shell), do not affect significantly the ultimate load as well as the stress distribution.
The aim of the present paper is to investigate the origins of the geometric imperfections and the damages observed on a seventeen years old cooling tower erected in France. Measurement of the amount of the imperfection of the shell, the differential foundation settlements and the crack patterns have been recorded in 1982, after two years operating, then a second inspection was performed in 1994. A finite element model is presented for thin shells employing a layered representation through the thickness to account for the material nonlinearities (evolution of cracking, concrete crushing and steel yielding). The structure is analysed for different load combinations, considering the shell as perfect or introducing the shell imperfections as initial geometry. The main objective of this study is to attempt to appreciate the possible evolutions of the shell structure in the future on the basis of the measurement database and the numerical simulations.

2 CONSTITUTIVE MODELLING OF REINFORCED CONCRETE

The assumption of a plane state of stress in the concrete layers allows biaxial constitutive models to be used. Reinforced concrete is described by simple superposition of a concrete model and a steel model, considering a perfect bond between steel and concrete. The constitutive laws for uncracked concrete and for steel are based on the plasticity theory. The concrete is assumed as a softening material both in tension and in compression with a correct description of the unilateral behaviour of the cracked concrete (opening, closing and reopening of the cracks). Indeed, even for structures loaded monotonously local unloading can occur due to the redistribution of the released stresses. The failure criterion for concrete is based on four-parameters Ottosen-model [10] involving the three stress invariants \( I_1, J_2 \) and \( \theta \):

\[
f(I_1, J_2, \theta) = \frac{a J_2}{f_c^{\sigma_2}} + \frac{\lambda \sqrt{J_2}}{f_c^{\sigma_e}} + \frac{b I_1}{f_c^{\sigma_1}} - 1 = 0
\]

where \( \lambda \) is a function of \( \cos(\theta) \), \( I_1 \) = first invariant of stress tensor, \( J_2 \) = second invariant of stress deviator tensor, \( \theta \) = angle of similarity or Lode angle lying in the deviatoric plane (function of third invariant), \( f_c^{\sigma_e} \) = uniaxial compressive strength of concrete, \( a \) and \( b \) are constants that correspond to different values of \((f_c^{\sigma_1}/f_c^{\sigma_2})\) where \( f_c \) is the uniaxial tensile strength. The yield criterion for uncracked concrete is assumed on the basis of the known failure criterion by selecting it as a proportionally reduced shape of the failure surface. Associated flow rule and isotropic hardening are assumed. For concrete in tension a smeared fixed crack approach is considered. With this formulation cracking is distributed over the area that belongs to an integration point and cracked concrete is treated as an orthotropic material with principal axes normal and parallel to the crack direction. Rough crack behaviour is incorporated in the model transmitting shear forces across the crack (the cracked concrete can still transfer 40% of the total shear in a shell by aggregate interlock). Then the concrete behaviour in the orthotropic directions is governed by a uniaxial stress-strain relationship.

3 ANALYSIS OF R/C COOLING TOWER

3.1 Cooling tower geometry and in-situ measurement

The geometry is given in figure 1 and the isotropic material properties for the two components, namely tower and columns are given in Table 1. The shell section is subdivided into 10 concrete layers and the grid is made of two orthogonal bar layers. The steel strain can
be directly obtained from the strain tensors of the middle surface. A mesh sensitivity analysis has shown that a finer discretization of shell is necessary (2652 elements), especially around the circumference, in order to capture correctly the geometric imperfections [9].

The geometric imperfections have been recorded in 1982 and 1994 by photogrammetry on the outer surface of the tower, in a regular mesh with 60 meridional and 130 circumferential measurements. The analysis of these photogrammetric statements suggests next remarks:

- The amplitude of the radius deviation has not evolved significantly in 12 years. The average imperfection progression is about 5 cm. However, near the throat, a maximal evolution of about 15 cm is recorded.

- These imperfections appeared very early (they were existing already in 1982, that is two years after the construction). Their origin was ascending probably to the phase of construction.

### Table 1. Concrete and steel properties.

<table>
<thead>
<tr>
<th>Basic materials parameters</th>
<th>CONCRETE</th>
<th>STEEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (MPa)</td>
<td>34500</td>
<td>210000</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.18</td>
<td>0.18</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>0.0035</td>
<td>0.004</td>
</tr>
<tr>
<td>Ultimate compressive strain</td>
<td>0.00035</td>
<td>0.00014</td>
</tr>
<tr>
<td>Ultimate tensile strain</td>
<td>-</td>
<td>400</td>
</tr>
<tr>
<td>Yielding limit (MPa)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

3.2 Numerical analyses

The different load combinations that will be studied are the following:

- Loading 0 : \( P_0 = W + \text{Set}_{82} \)
- Loading 1 : \( P_1 = P_0 + P_{th} \)
- Loading 2 : \( P_2 = P_0 + \Delta \text{Set} \)

with: \( W \) : dead weight.
\( \text{Set}_{82} \) : Differential settlements of the ground measured in 1982.
\( \Delta \text{Set} \) : Increase of differential foundation settlements between 1982 and 1994.
\( P_{th} \) : Loading corresponding to a maximum thermal gradient in the concrete between external and internal walls including functioning phase of the nuclear powerplant and phase of stop.

Loadings 0 and 1 are applied on the perfect shell in order to analyse the origin of the geometric imperfections observed in 1982, whereas loading 2 is applied to the deformed shell considering the initial materials properties, in order to simulate the evolution of the imperfections between 1982 and 1994.
3.2.1 Differential settlements sensitivity of the cooling tower shell

Two analyses were carried out to investigate the effects of the differential foundation settlements on the initiation of the geometric imperfections on one hand and on their evolution between 1982 and 1994 on the other hand.

The foundations and the ground are modelled by spring type elements connected individually to the basis of each supports (figure 2). The stiffness of these elements are composed by three terms related to translation: \( K_{rr} \), \( K_{tt} \), \( K_{zz} \) and three other ones that lie to rotation: \( K_{\Omega r} \), \( K_{\Omega t} \), \( K_{\Omega z} \). These quantities are computed from recorded differential settlements realised in 1982.

The supposed "perfect" structure (without imperfections) is then submitted to its dead load. The analysis of both radial displacements of the shell and the stress field in concrete (figure 5) shows that the total differential settlements recorded in 1982 are not in fact the main cause of the initial imperfections observed on the cooling tower shell. Indeed the computed radius amplitude deviations do not exceed 2.4 cm, while the maximal measured amplitudes are about 2.5 cm. These differential foundation settlements do not explain the shell cracking state observed since the construction of the cooling tower (figure 4). Indeed, the numerical results show that the principal stresses in concrete do not exceed the tensile strength (figure 5).

![Figure 2: Equivalent stiffness schematisation.](image1)

![Figure 3: Computing process schematisation of the differential settlements increases.](image2)

![Figure 4: Cracking state of the cooling tower shell observed in 1982 (represented in plane)](image3)
Figure 5: Radial displacement and major principal stresses $\sigma_{11}$ calculated under combined effects of the dead load and differential settlements recorded in 1982 (Loading 0).

Figure 6: Radial displacement and major principal stresses $\sigma_{11}$ calculated under combined actions of the dead load, differential settlements recorded in 1982 and the maximum thermal gradient through the thickness during the functioning phase (Loading 2).
A second analysis was performed in order to verify the influence of the increases of the differential settlements (measured between 1982 and 1994) on the evolution of the shell degradation observed during 12 years. For this purpose we have modelled the foundation by spring elements (figure 3), whose stiffness were calibrated from differential settlements recorded in 1982. Then the increases of the differential settlements have been applied to the base of the supports (figure 3) in the form of imposed displacement, in order to deduce the equivalent reactions of the foundation. Finally the loading 2 composed of the dead load and the equivalent reactions has been applied on the structure considering the deformed shell (with the imperfections recorded in 1982).

The analysis of the calculated radial deviations of the shell and the stress distribution shows that the structure is not very sensitive to the increases of differential settlements between 1982 and 1994. Indeed, the increase of the displacements are negligible in comparison to the recorded values (progression about 5 cm). Moreover for this level of loading the concrete behaviour remains practically linear.

3.2.1 Sensitivity to thermal loading (sunshine and gradient of functioning).

To better understand the thermal loading effects on the ageing of the cooling tower, it is necessary to distinguish in thermal loads, those associated with phases of stop of the nuclear power plant and those related to the phase of functioning.

During phases of stop, the structure is submitted only to ambient meteorological factors. Consequently, the maximal temperature knowledge recorded during this phase gives an estimation of the thermal loading due to the sunshine.

During the phase of functioning, the thermal gradient of functioning between inner and outer surface is added to the climatic thermal effects.

Circumferential distributions of temperatures in the concrete on the inner surface of the shell, outer surface and middle surface as well as the thermal gradient in the thickness, are presented on figure 7.

Figure 6: Circumferential distributions of temperatures in concrete, on inner surface, outer surface, middle surface and the thickness gradient
Loading is then applied on the structure, composed of dead load, differential settlements measured in 1982 and thermal loading corresponding to the functioning phase (which is the most discriminant).

The investigation of the radial displacement and concrete stress distribution (figure 6) shows that the shell with imperfections, supposed initially non damaged, is not very sensitive to thermal loads such they are applied (cyclic effects are neglected). Indeed extreme values of radial displacements do not exceed 4.1 cm and the concrete remains practically elastic.

4 - CONCLUSIONS

In this study we focused the investigation on the interpretation of the geometric imperfections observed on a reinforced cooling tower shell in 1982 and on their evolution in 12 years. To bring some answers to the interrogations concerning the evolution and the initiation of these damages, the structure has been analysed under different combinations of loadings (dead load, differential foundation settlements and thermal loads generated during the functioning phase). It has been demonstrated that the shell is not very sensitive to the different loads combinations adopted here.

These conclusions tend to confirm the already advanced hypothesis, namely that the geometric imperfections recorded after two years operating were probably generated during the construction phase. The shrinkage and the creep of the early-aged concrete have not been investigated. Moreover the evolution of these imperfections in 12 years, is not explained by the action of classically envisaged load combinations such that they were considered in this study. Indeed, one of the strong hypotheses admitted in this work consisted to suppose that the perfect shell or the shell with imperfections is initially uncracked without constraint. Nevertheless, we consider that it would be interesting to take into account this notion of initial state in the future. To capture this initial state of stress and strain, a first approach will consist to apply on the perfect shell the recorded radius deviations observed in 1982, by displacement control. Then the increases of measured displacements will be applied in the same way. Moreover a precise study on the evolution of the concrete properties during the construction phase should be necessary to estimate the shell deformation imposed by the mode of construction. Indeed two-days-old concrete is supporting a portion of shell of about 1.5 m height at each step of casting.

REFERENCES


