RESPONSE OF HYPERBOLIC COOLING TOWER FOR INCREASING EARTHQUAKE TAKING MATERIAL NONLINEARITY OF COLUMNS, SLIPPING AND LIFT-OFF INTO ACCOUNT

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SUMMARY

The supporting columns and their foundation influence the cooling tower's seismic response decisively. At the same time, they represent the most vulnerable part of the structure. For increasing seimsic excitation, the constitutive laws of the concrete and of the reinforcement steel in the columns as well as slipping and lift-off of the foundations are incorporated into a nonlinear analysis. A large reduction in response results from the behaviour of the columns in tension. Slipping and lift-off play a less important role.

INTRODUCTION

In recent years, an increasing number of high hyperbolic, natural--draught cooling towers have been designed and erected in seismic active zones. As for the case of wind loading, which traditionally governed its design, a detailed analysis of the tower's earthquake response has to be performed. Possible nonlinearities are to be considered, especially when performance and the corresponding reliability of the tower for increasing seismic loads are determined.

Hyperbolic cooling towers are nowadays routinely designed for a (moderate) earthquake which can reasonably be expected at the site during the life of the structure. For such a design basis earthquake (DBE), the tower is usually required to remain elastic, as this level corresponds to that seismic excitation for which all structures of the industrial plant have to remain fully operational (plant-availability earthquake). In addition, the tower is examined for the most severe earthquake which could possibly occur, to ascertain that no danger of collapse exists. Especially for cooling towers in nuclear-power plants, such a possible collapse for, e.g., the safe--shutdown earthquake, has to be avoided, as the falling debris could endanger neighbouring safety-relevant structures. In zones of high seismicity, this additional analysis will be strongly nonlinear.

As the supporting columns form the weakest link in a tower subjected to the uniformly distributed lateral loads resulting from horizontal earthquake excitation, large relative displacements between the upper and lower ends of the columns arise. This global (i.e. with respect to the entire tower, not the individual column) shear distortion is similar to that encountered in a structure with a "soft first storey". It leads to large forces in the foundation, in the columns and in the lower edge beam of the shell. Hence, for increasing seismic loads, these parts of the tower are the first whose design is governed by the loading case of earthquake instead of by the wind.

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NONLINEAR ANALYSIS

For all investigations the 144-m-high reinforced-concrete cooling tower of the nuclear-power plant in Leibstadt, Switzerland, forms the base. Its main dimensions are specified in Figs. 1 and 2. The selected nomenclature for the local displacements and the forces of the shell is depicted in Fig. 3. Each of the 36 pairs of columns rests on a separate foundation (Fig. 2). The material properties of the shell are as follows: modulus of elasticity = 30 GPa, density = 2.5 Mg/m^3 , Poisson's ratio = 0.2. For the prefabricated circular columns with the diameter d = 0.85 m, the modulus of elasticity is increased to 35 GPa. The shear modulus, the density and Poisson's ratio of the soil, which is regarded as an elastic halfspace, equal 0.12 GPa, 2.4 Mg/m^3 and 0.4, respectively.

The axisymmetric shell, which is assumed to remain linear-elastic, is discretised with 30 higher-order finite-element frusta with an isoparametric expansion in the meridional and the standard Fourier expansion in the circumferential direction [1]. The stiffening-ring beam, the supporting columns and the individual foundation springs are synthesized into a stiffness matrix compatible with the axisymmetric shell element [2].

The cooling tower is analysed for multiples of the horizontal design--basis earthquake (DBE). No vertical excitation is assumed. In the actual design, the tower has been dimensioned for a DBE with a peak horizontal ground acceleration of 0.12 g. The corresponding response spectrum closely follows that of the USNRC Regulatory Guide 1.60. For all nonlinear calculations performed in this section, the artificially generated time-history $\ddot{x}_g(t)$, shown in Fig. 4 (and multiples thereof), is used. It envelopes the DBE response spectrum for 7% damping.

The nonlinear constitutive laws of the concrete and of the reinforcement steel used in the columns as well as the corresponding normal force--strain relationship are depicted in Fig. 5. The stress-strain ($\sigma - \varepsilon$) curve for concrete as recommended by CEB/FIP [3] for loads of short duration is idealised as shown in Fig. 5a. No tensile capacity is introduced. The elastic-ideal-plastic stress-strain relationship of the reinforcement steel is shown in Fig. 5b. When determining the normal force-strain ($N - \varepsilon$) relationship (Fig. 5c) of a column, an area of reinforcement steel A_s = 136 cm^2 is used (15 re-bars of 34 mm diameter). In addition to this curve, denoted as "2-material law", the familiar relationship indicated as "elasto-plastic" is sometimes used for comparison. The hysteretic behaviour of the materials is indicated in the figure. The vertical reaction force R_w of an individual foundation is plotted as a function of the vertical displacement w in Fig. 6. The ultimate pressure of the soil equals -1 MPa. The radial, R_u, and the tangential, R_v, reactions are limited by Coulomb's friction law (friction coefficient = 0.58).

The aximsymmetric computer code [1, 2], used for all linear and geometrically nonlinear analyses in the process of design is expanded to include material nonlinearities. The coupling terms, which occur between the linear case) including only up to the first harmonic for horizontal earthquake excitation and dead load. To be able to judge how many harmonics have to be included, comparative studies with the 0th - 1st harmonic, with the 0th - 5th harmonic and with the 0th - 20th harmonic, using the 2-material law for 2·DBE, are conducted. The nonlinearity leads to a significant distortion of a cross-section of the shell. In contrast, including only up to the first harmonic in a nonlinear analysis produces no visible difference in the displacement around the circumference ($u_0 \cong -v_0$). The results obtained by using up to the 5th harmonic coincide from a practical point of view with those of the analysis with the 0th - 20th harmonic. This is demonstrated in Fig. 10, where the in-structure response spectra at the top of the shell for total acceleration in the x-direction ($\theta = 0^{\circ}$) are depicted. As expected, an analysis adopting only up to the first harmonic results in a significant deficit in the higher-frequency range. The vertical R_w and tangential reaction forces R_v at 2.7 s are plotted versus the circumferential angle in Figs. 11a and b, respectively. Once again, the use of up to the 5th harmonic, which is adopted for all calculations in this article, is justified. The results corresponding to the analysis with only the two lowest harmonics tend to average the response over the circumference. For a further comparison, the reaction forces of a linear analysis are also shown. The general characteristics of this type of nonlinear earthquake analysis [4] are clearly visible: smaller global horizontal-shear force, smaller global overturning moment and larger global vertical-reaction force.

As a sensitivity study, the nonlinear analyses are also performed for even higher earthquake-acceleration values, using the 2-material law. For 3·DBE (0.36 g), all individual foundations slip at least once during the earthquake. The two neighbouring separate foundations located symmetrically to $\theta = 180^{\circ}$ just start to lift off for this level of excitation. However, the extreme normal forces in any column, 3.37 MN and -14.6 MN, are still well below the corresponding yielding values 6.80 MN and -19.9 MN (Fig. 5c). All columns thus behave bilinear-elastically. The first 5 s of the time history of the radial u and tangential displacements v of the foundation at $\theta = 105^{\circ}$ are plotted in Figs. 12a and b, respectively. The remaining slips of the individual, separate foundations cause permanent displacements and self-equilibrating forces in the tower.

Finally, the earthquake scaled to a level of 5·DBE (0.60 g) is adopted. Although for such strong shaking the linear-elastic model of the shell is questionable, this extreme case should nevertheless give valuable insight into the problem, as it overstresses the nonlinear phenomena. All the individual foundations are subjected to extensive lift-off and slipping. As an example, the first 5 s of the horizontal displacement of the separate foundation at $\theta = 145^{\circ}$ is shown in Fig. 13. Even such large displacements do, however, hardly lead to inelastic behaviour of the columns (extreme values 6.27 MN and -20.4 MN). The minimum vertical soil pressure equals -0.8 MPa, which is larger than the ultimate value (= -1.0 MPa). Thus, even for 5·DBE, no yielding of the soil in compression occurs. Significant permanent displacements and forces remain after the earthquake.

Normal projections of the permanent displacements of the entire tower onto the planes y = o and x = o are depicted in Fig. 14. The deformed shape is not symmetric with respect to the y - z plane. The upper part of the shell is tilted towards x < o.

To characterise the nonlinear response, the maximum global reactions (vertical, horizontal (shear) and overturning moment) of the tower are plotted as a function of the level of earthquake excitation in Fig. 15. These results are consistent with the conclusions derived using quite different types of structures [4].

CONCLUSIONS

1. A linear-elastic analysis exaggerates the seismic response considerably and cannot even be used to determine the results for the DBE (0.12g).
2. A nonlinear analysis can no longer be limited to the zeroth and first harmonic only, as in the linear case. However, satisfactory results are achieved when based on quite a small number of harmonics (up to the 5th).
3. The behaviour of the columns in tension (cracked concrete, elastic steel) represents by far the most important nonlinearity. Even for an excitation level of 0.24 g this is the only nonlinearity which appears. When this bilinear-elastic law is introduced in the design (0.12g), no reinforcement steel in the columns is theoretically necessary.
4. The elastic-ideal-plastic material law cannot be used.
5. Substantial slipping and slight lift-off occur for an excitation of 0.36 g. Even for 0.60 g the ultimate compressive strength of the soil is not reached. At this level of excitation, the compressive capacity of the concrete (but not of the steel) in the columns is reached, whereas in tension the reinforcement steel still behaves linearly.
6. Accounting for the nonlinearities, a more flexible system with correspondingly smaller seismic input results. The well-known reduction in the global response in the horizontal direction and the increase in the global vertical response are apparent.

ACKNOWLEDGEMENT

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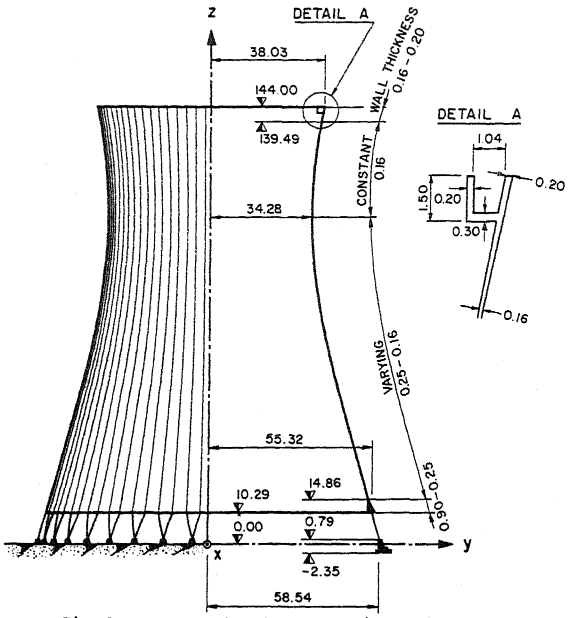


Fig. 1 Natural-draught hyperbolic cooling tower Leibstadt [m]

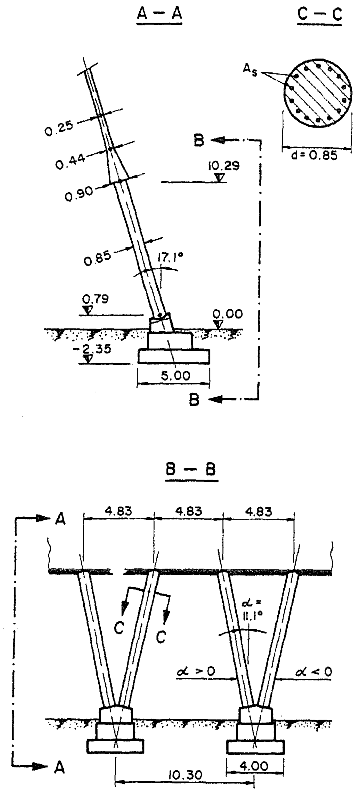


Fig. 2 Geometry of supporting columns

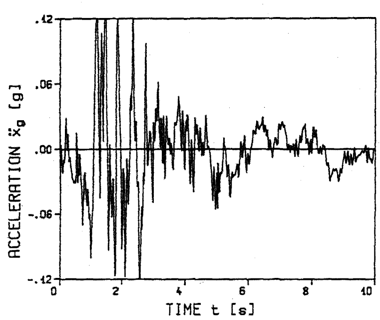


Fig. 4 Acceleration time-history, DBE (0.12 g)

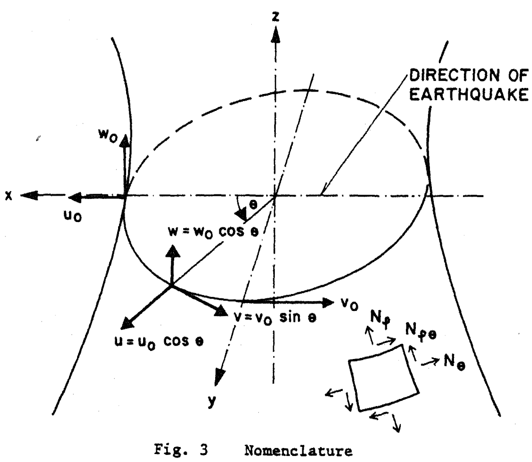


Fig. 3 Nomenclature

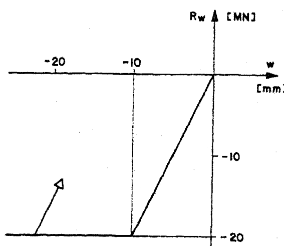
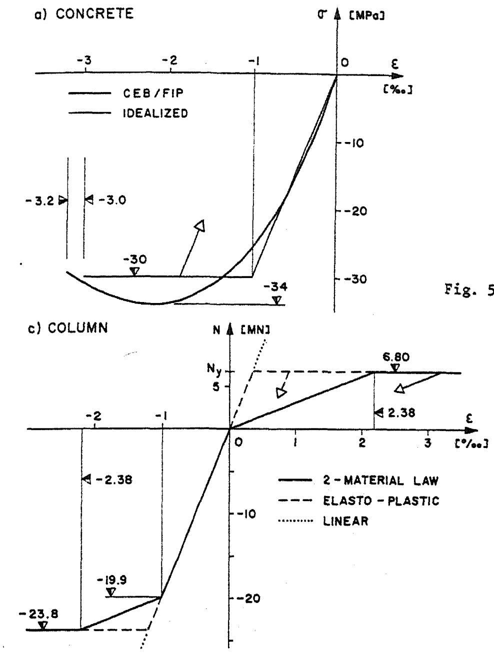


Fig. 6 Vertical reaction-displacement relationship of separate foundation



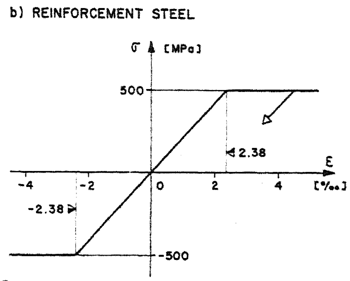
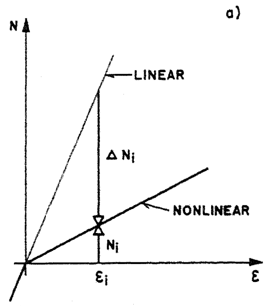
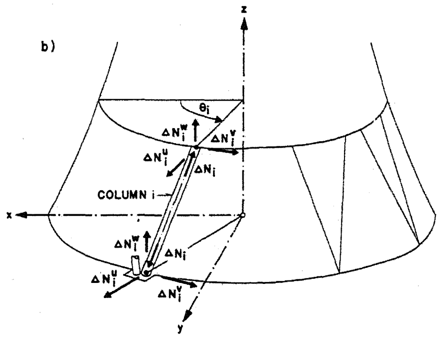


Fig. 5 Constitutive laws and normal force-strain relationship of columns





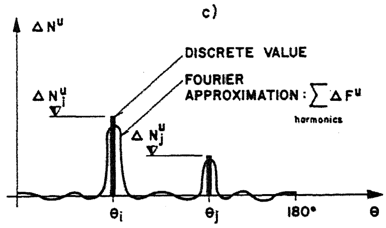


Fig. 7 Mathematical procedure to incorporate nonlinearity of column

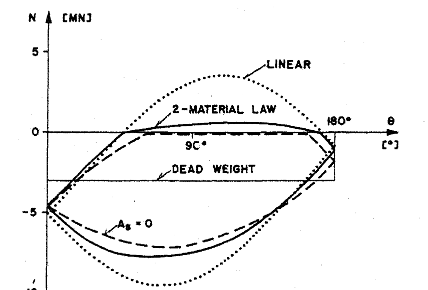


Fig. 8 Normal force in column versus circumferential angle, t = 2.7 s, DBE (0.12 g)

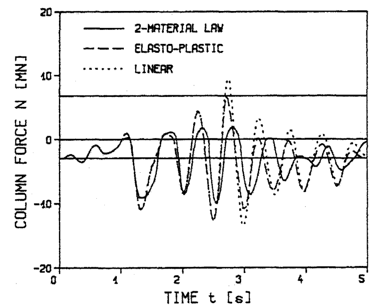


Fig. 9 Normal force in column at $θ = 105°$, $α < 0$ versus time, 2·DBE (0.24 g)

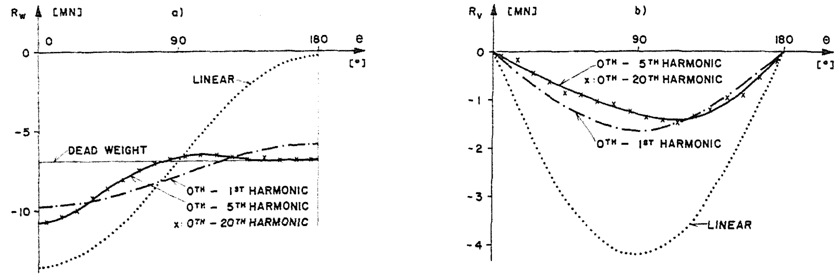


Fig. 11 Reaction force versus circumferential angle, t = 2.7 s, 2-material law, 2·DBE (0.24 g)

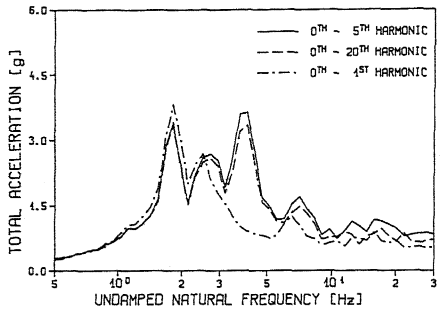


Fig. 10 In-structure response spectra (1% damping), x-direction, top of shell, θ = 0°, 2 material law, 2·DBE (0.24 g)

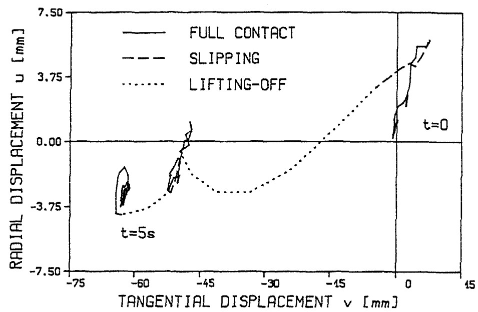


Fig. 13 Horizontal displacement of separate foundation at θ = 145° versus time, 2-material law, 5·DBE (0.60 g)

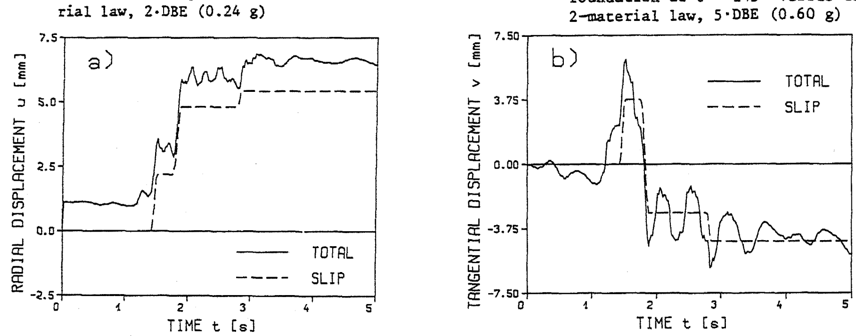


Fig. 12 Horizontal displacement of separate foundation at θ = 105° versus time, 2-material law, 3·DBE (0.36 g)

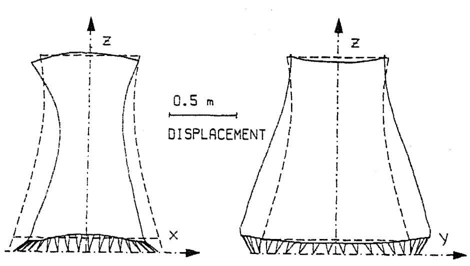


Fig. 14 Permanent displacement after earthquake, 2-material law, 5·DBE (0.60 g)

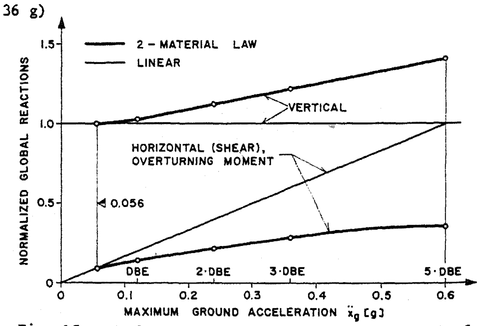


Fig. 15 Maximum global reactions versus level of earthquake excitation, 2-material law