Floor-Response-Spectra Calculation with a 3D-Building-Model Comparison with an Axisymmetric Analysis

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Abstract

The paper describes the seismic analysis of the reactor-building of a 1300 MW pressurized water reactor with a very detailled three-dimensional finite element model consisting of about 1900 elements. Soil properties are included in the analysis in form of equivalent springs, determined from the theory of the elastic half-space. In order to decouple spring reactions in translational and rotational degrees of freedom, a special construction with rigid elements is used. These elements, whose translations resp. rotations are suppressed, and the fundamental plate are interconnected by the rotational resp. translational springs representing soil.

Since the whole model has more than 7500 degrees of freedom (d.o.f.), it was necessary, to reduce its size for dynamic calculations. To find out those d.o.f. to be retained in the analysis, the magnitude of the quotients of corresponding terms in the stiffness- and mass-matrices were considered.

In the response analysis all eigenvectors found below 28 Hz were used. Seismic excitations are applied simultaneously in horizontal and vertical direction. Floor-response-spectra were calculated at several locations on every floor to recognize differences caused by asymmetry.

A comparison of the results of the eigenvalue-analysis with those obtained from an axisymmetric model shows good agreement for the first modes, higher modes mostly are difficult to compare. Both models give similar response-spectra in horizontal direction. The curves agree very well at locations, where no deviations from rotational symmetry exist. Generally extreme peaks of acceleration are less significant in the 3D-model. In vertical direction accelerations in the 3D-model are lower in most cases, which can be mainly explained by the absence of radial wall-disks in the 2D-model.

The results can be summarized as follows: An analysis with a 2D-model leads in the case under review to conservative results, the calculated accelerations are generally higher than those obtained from a 3D-model.

1. Introduction

Seismic design of nuclear power plant components requires the knowledge of the dynamic behaviour of the involved structures under seismic loading. Generally components are treated uncoupled from the supporting building, and the floor-responses are used as excitation for components and equipment. To determine the dynamic response of the various parts of the building-structure, it is necessary to perform a complete analysis of the building and foundation system.

This paper describes the seismic analysis performed for the reactor-building of a 1300 MW pressurized water KWU-reactor (PWR) of the "KONVOI"-type.
The foundation-soil of the considered plant is consisting of very stiff rock. Usually the lowest eigenfrequencies of the reactor-building of other comparable plants occur at about 2 Hz. In this case a value of about 4 Hz was expected, which causes - together with a relatively high peak ground-acceleration - difficulties in the design of some important components as the steel containment for example.
Therefore a very detailled three-dimensional 3D-finite element model of the building was created. Soil was represented by equivalent springs and dashpots.
A three-dimensional model has several advantages compared with an axisymmetric one: It allows a more realistic consideration of unsymmetrically arranged masses and stiffnesses, and it is not necessary to modify the stiffness of walls and floors to cover the effects of radial wall-disks. On the other hand the requirements of computer time and hardware are much lower using a symmetric model.
The results of the dynamic calculations were compared with those obtained from another analysis, which based upon an axisymmetric building-model.

2. Building model

The outside cover of the building is consisting of a cylinder of reinforced concrete bounded by the fundamental plate at the bottom, a hemisphere at the top and a lateral annex. The building has an outer diameter of 66.8 m and a total height of about 64 m, the thickness of the outer walls is 1.8 m. The inner structure is composed in the main of concentrically arranged cylinders, interconnected by floor- and wall-disks, and the reactor-containment, a steel-sphere with a diameter of 56 m and a wall-thickness of 38 mm.
Inner and outer structure are decoupled in horizontal direction. Floor-disks and outer wall are separated by expansions joints, their only horizontal interconnection is the fundamental plate.
As mentioned above, the finite element model, constructed for the dynamic analysis, is fully three-dimensional, it chiefly consists of thin flat isoparametric shell elements. Special attention was given to a realistic

representation of those regions of the building, where deviations from axisymmetry occur.

Horizontal decoupling of inner and outer structure was modelled with rigid elements connecting only vertical degrees of freedom (d.o.f.) of corresponding nodes and leaving unconstrained all others. The whole structure was subdivided over its circumference into twenty equal sectors. The masses of the concrete structures were attached to the corresponding nodal points as lumped masses. Components and equipment were also taken into account as lumped masses, their structural properties were neglected with few exceptions: the reactor containment, the polar crane and the reactor pressure vessel. The reactor containment was modelled with shell elements, the polar crane with bar elements, and for the reactor pressure vessel a special construction with rigid elements was used, distributing the mass realistically to the supporting concrete.

The complete building-model then consists of about 1400 elements with 6000 d.o.f.. A plot of the model is shown in fig. 1.

3. Soil model

Due to its consistency the behaviour of the considered soil at the forecasted maximum seismic loading remains linear-elastic. Preliminary investigations with finite element models of the soil including simplified building models, have shown beyond that, that the influence of different soil-models on the results cannot justify such an effort. Therefore an equivalent spring approach based upon the theory of the elastic half space was used. The average soil properties are:

mass density: $\varrho = 2500\ \mathrm{kg/m^3}$
Poisson's ratio: $\nu = 0{,}33$
dyn. shear modulus: $G_{dyn} = 5300\ \mathrm{Mn/m^2}$

From the theory of the elastic half-space, spring-values can be obtained for vertical, sliding, rocking and torsional vibration of a rigid circular footing /1/. As in the finite element model springs must be attached to every node of the fundamental-plate, these values had to be split up in an appropriate manner. Using a 3D-model this distribution causes some problems: The calculated spring-values are valid only for pure sliding, rocking etc. In reality, however, these movements always occur simultaneously.

To decouple spring reactions, a special construction was used:

Two rigid elements were created, each having the same configuration and number of nodes as the fundamental plate. One of them is free to rotate and connected with the fundamental plate by spring elements for translational movements, the other is free to move in translational directions and connected with the footing by spring elements for rotations; the rotational d.o.f. of the element itself are constrained.

The constants of the individual spring elements were determined by following equations:

$$k_j(u) = K_u \cdot \frac{A_j}{A} \qquad (1)$$

for the translational d.o.f.,

$$k_j(\psi) = K_\psi \frac{A_j}{I_a} \qquad (2)$$

for rocking, and

$$k_j(\varphi) = K_\varphi \cdot \frac{A_j}{I_p} \qquad (3)$$

for torsion with the d.o.f. u, φ, ψ, total and individual spring constants K and k_j, total footing area A and the partial area A_j corresponding to element j, and finally the axial I_a and polar area moment of inertia I_p of the complete plate.

4. Eigenvalue analysis

The dynamic calculations were split into two steps: The first one was an eigenvalue analysis, in the second one the seismic excitation was applied, and the floor responses were calculated.

The magnitude of the structure (the complete model has more than 7500 d.o.f.) and the great number of eigenvalues expected in the frequency-range under review (up to 33 Hz) made it necessary, to reduce the number of dynamic d.o.f. substantially. Therefore a static condensation ("Guyan-reduction") was performed. The reduced set of d.o.f. should as a rule meet the following requirements: It should contain those d.o.f., which

- carry great lumped masses
- are suited to reproduce the expected modes of the structure.

Since a reasonable reduction, using only that approach, is very difficult and cumbersome for great structures with complicated topologies, a further criterion was applied: The eigenvalue-equation of an undamped system may be written in matrix notation as

$$K - \lambda M = \{0\} \qquad (4)$$

with the stiffness- (K-), mass- (M-) and the eigenvalue matrix λ .

Regarding the quotients of corresponding diagonal-elements of the stiffness- and the mass-matrix, one can get an indication for the magnitude of the eigenvalues.

In a preliminary run these quotients were calculated, and after testing of plausibility on the basis of the above mentioned criteria, those d.o.f. associated with the lowest values were retained in the reduced set.

In the frequency-range examined ($0 < f < 33$ Hz), 284 eigenvalues were found. The first mode was calculated at 4.27 Hz. A comparison of that value with the results of another analysis with fixed-base building-model shows the relatively small influence of soil: The lowest eigenfrequency of the fixed building was found to be 5.5 Hz.

5. Floor-response calculation

Seismic excitation is given as free-field acceleration spectrum with a peak-ground acceleration of 0.17 g in horizontal direction (fig. 2), vertical accelerations are 50 % of the horizontal values. From that two statistically independent, spectrum-compatible acceleration time-history records were generated as horizontal and vertical excitation. Both are 15 s long, subdivided into 1500 time steps. They are applied to the structure simultaneonsly.

Two loading-cases with perpendicular horizontal excitation directions were considered to find out the maximum responses.

All modes calculated below 28 Hz were used in the analysis. That restriction was made for two reasons: Having a statically condensed system, the number of utilized eigenvalues should not exceed substantially half the number of dynamic d.o.f. (the chosen frequency-range contains 224 eigenvalues, the reduced set 429 d.o.f.). Beyond that in the range between 28 and 33 Hz the remaining amplifications from ground-spectra are very small.

Soil-springs were given, damping-values determined by half-space theory, valid at the resonant frequencies of the springs. For concrete and steel structures frequency-independent dampings of 7 % and 4 % of critical are prescribed /2/. Modal damping was calculated for those modes, showing distinct deformations of soil-springs, the reactor-containment etc..

Then the structural damping of 7 %, input as a tabular function of frequency, was modified where necessary. Total damping was bounded to an upper limit of 30 % for vertical and 15 % for horizontal and rocking modes. These values are stated in /2/.

On most floors responses were calculated for two or three points, located on the circumference at 0°, 90° and 180°.

In that way, differences caused by deviations from axisymmetry could be detected.

In the first instance horizontal and vertical acceleration time-histories for each loading-case were calculated, followed by floor-response-spectra generation.

The differences appearing in the responses of three points on the same floor of the outer cover are shown as an example in fig. 3 (The spectra shown in that and the following figures have 2 % of damping and were drawn in a slightly simplified manner). In the lower frequency-range, containing only global building modes, curves are quite similar. Above 6 Hz substantial deviations occur, mainly caused by the lateral annex. Horizontal spectra of a point in the lower part of the inner structure for the two loading-cases represents fig. 4.

In most cases the differences are smaller than those shown in figures 3 and 4.

6. Comparison with an axisymmetric analysis

Comparing the results of modal analyses, it is remarkable that the number of eigenvalues calculated with a symmetric (2D-) model is much smaller. Main reasons for that phenomenon are:

- higher order harmonics are neglected in the axisymmetric analysis
- every mode in the 1. harmonic of the 2D-model corresponds to two approximately equivalent modes of the 3D-model in two perpendicular horizontal directions
- the 3D-model has partially lower local stiffnesses associated with lumped masses, this could cause additional local modes.

Vertical sections of both models shows fig. 5.

The first mode of the axisymmetric analysis appears at 3.82 Hz, corresponding values of the 3D-model are 4.26 Hz and 4.38 Hz. In both cases we have a global bending-rocking-mode. Quite good agreement in frequencies and deformations show the fundamental modes of the inner structure and the reactor-containment at about 5.5 Hz and 6.5 Hz. A typical example of a mode not reproduceable by the 2D-model is a rotational vibration of the lateral annex at 7.06 Hz. Generally it can be seen, that the vertical deformations in the outer range of the fundamental plate and the lower part of the inner structure are smaller than in the 2D-model, and the obviously higher stiffness of the 3D-model in these parts of the structure results in higher eigenfrequencies for corresponding modes. This may be traced back to the absence of radial wall-disks in the 2D-model.

For the comparison of floor-responses, all spectra of the 3D- model obtained for one floor were envelopped to get a representative spectrum. Both models give similar curves in horizontal direction. Spectra agree very well at locations, where no deviations from rotational symmetry exist. Extreme peaks of acceleration are generally less significant in the 3D-analysis.

Regarding the responses in vertical direction, greater differences occur. Vertical accelerations in the 3D-model are in most cases substantially lower than those in the 2D-model.

Figures 6, 7 and 8 represent the results of comparisons of floor-response-spectra for some selected floors, at other locations results are similar in quality.

The comparisons show, that in the case under review an analysis with an axisymmetric model gives conservative results for most locations; the calculated accelerations are generally higher than those obtained from the 3D-model.

REFERENCES

/1/ F.E. Richart, Jr., J.R. Hall, Jr., R.D. Woods, Vibrations of Soils and Foundations, Prentice-Hall International, London, 1970

/2/ KTA 2201, Auslegung von Kernkraftwerken gegen seismische Einwirkungen, Teil 1 - 4, Kerntechnischer Ausschuß, Köln, 1975 - 1982

/3/ Biggs, J.M., Introduction to Structural Dynamics, Mc Graw-Hill, N.Y., 1964

/4/ Robert J. Hansen, editor, Seismic Design for Nuclear Power Plants, The M.I.T. Press, Cambridge, Massachusetts, 1970

/5/ M.A. Gockel, MSC/NASTRAN, Dynamic Analysis, The Mc Neal-Schwendler Corporation, Los Angeles, 1981

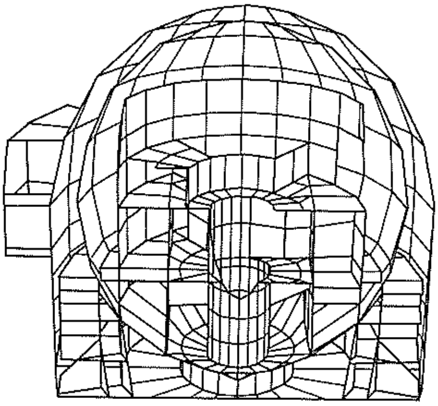


fig. 1 - Finite element model of the building

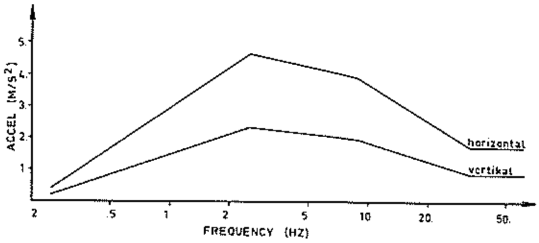


fig. 2 - Free-field acceleration-spectrum

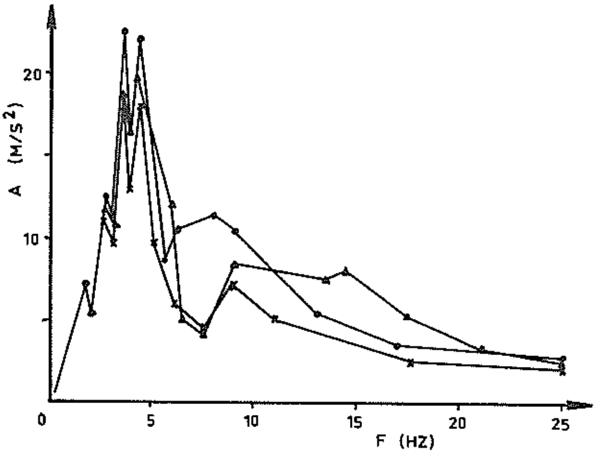


fig. 3 - Horizontal spectra of the outer cover at 0°, 90°, 180° and an elevation of 23 m above footing

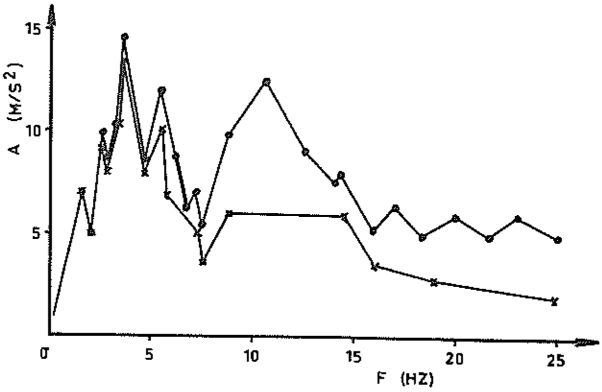


fig. 4 - Horizontal spectra of an interior point for two loading-cases with perpendicular horizontal excitation

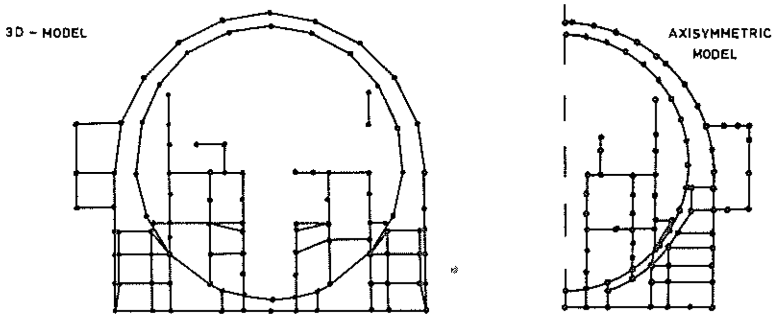
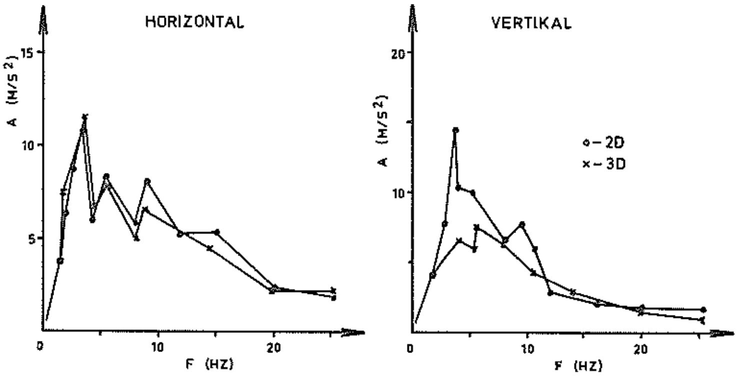


fig. 5 - Vertical section of the 3D-model compared with 2D-model

fig. 6 - Outer border of the fundamental-plate



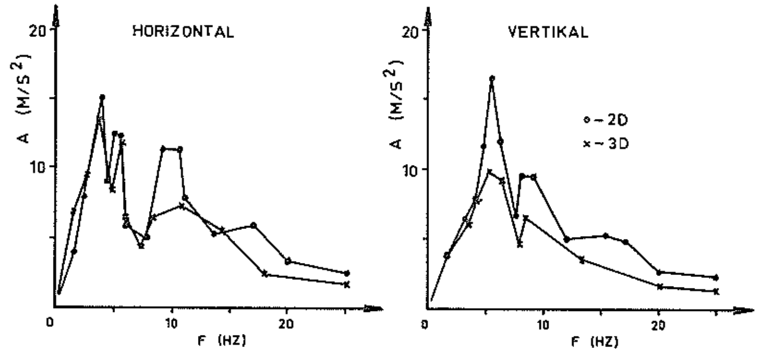


fig. 7 - Clamping zone of the Containment

fig. 8 - Runway of polar crane