Computed versus Measured Response of HDR Reactor Building in Large Scale Shaking Tests *

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1 INTRODUCTION

The earthquake resistent design of NPP structures and their installations is commonly based on linear analysis methods. Nonlinear effects, which may occur during strong earthquakes, are approximately accounted for in the analysis by adjusting the structural damping values.

Experimental investigations of nonlinear effects were performed with an extremely heavy shaker at the decommissioned HDR reactor building in West Germany. The tests were directed by KfK (Nuclear Research Center Karlsruhe, West Germany) and supported by several companies and institutes from West Germany, Switzerland and the USA. The objective was the dynamic response behaviour of the structure, piping and components to strong earthquake-like shaking including nonlinear effects (see Malcher L., H. Steinhilber 1987). This paper presents some results of safety analyses and measurements, which were performed prior and during the test series. It was intended to shake the building up to a level where only a marginal safety against global structural failure was left.

2 TEST ARRANGEMENT

The HDR building consists of an essentially axisymmetric external structure, an internal concrete structure of complex geometry and a common foundation (Figure 1a). The internal structure is completely enclosed in a steel containment, which is supported by an egg-cup-like concrete calotte. The building was designed in 1965-67 when earthquake design provisions were not yet required in seismically inactive areas of Germany. Wind loads were the only horizontal design loads.

The shaker operates in the frequency range between 0 and 8 Hz and can generate horizontal forces greater than 10,000 kN. That is more than can be sustained by the structure. The shaker has two excentric masses which are driven to spin in a balanced position. When the target speed is reached, the drive is stopped, and a lock between the two masses is released. Due to an offset between their respective axes the masses move quickly together into a maximum unbalanced position, in which they are locked again. They produce now the desired shaker force, driven only by their kinetic energy (Ibanez, P. et al. 1987).

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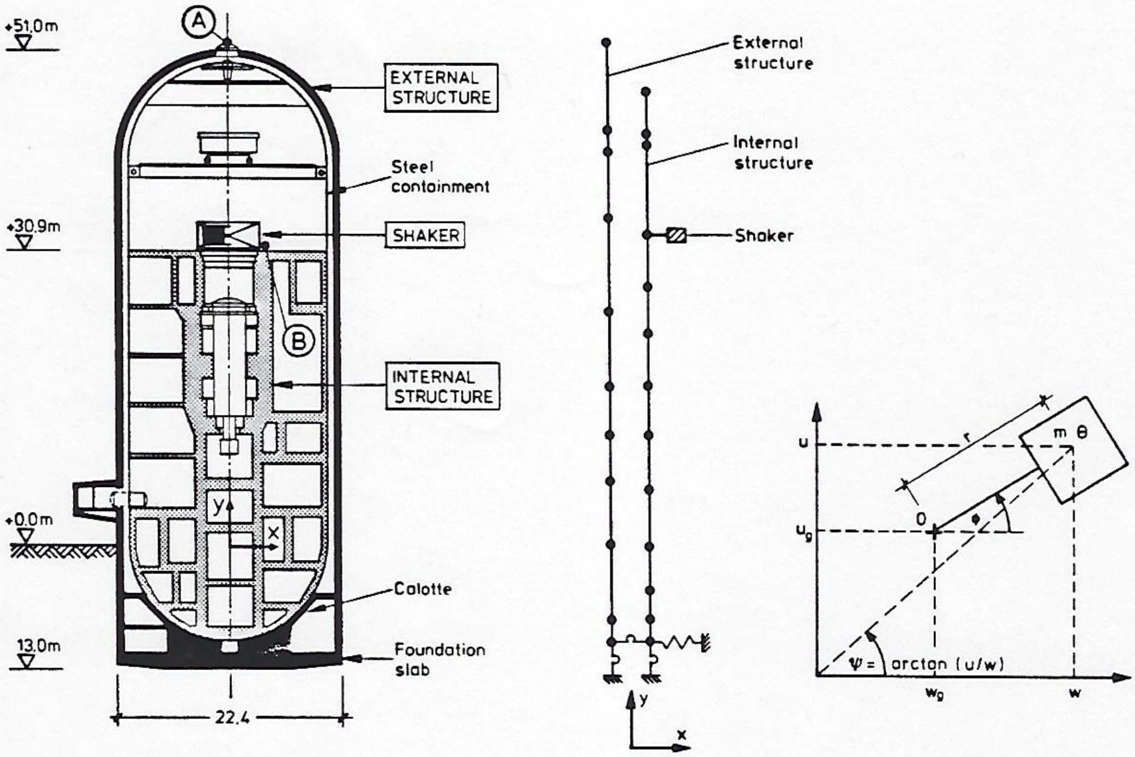


(a) (b) (c)

Figure 1. Reactor building with shaker (a); beam model (b); analysis model of shaker rotating in horizontal plane (c)

3 METHOD OF ANALYSIS AND STRUCTURAL MODEL

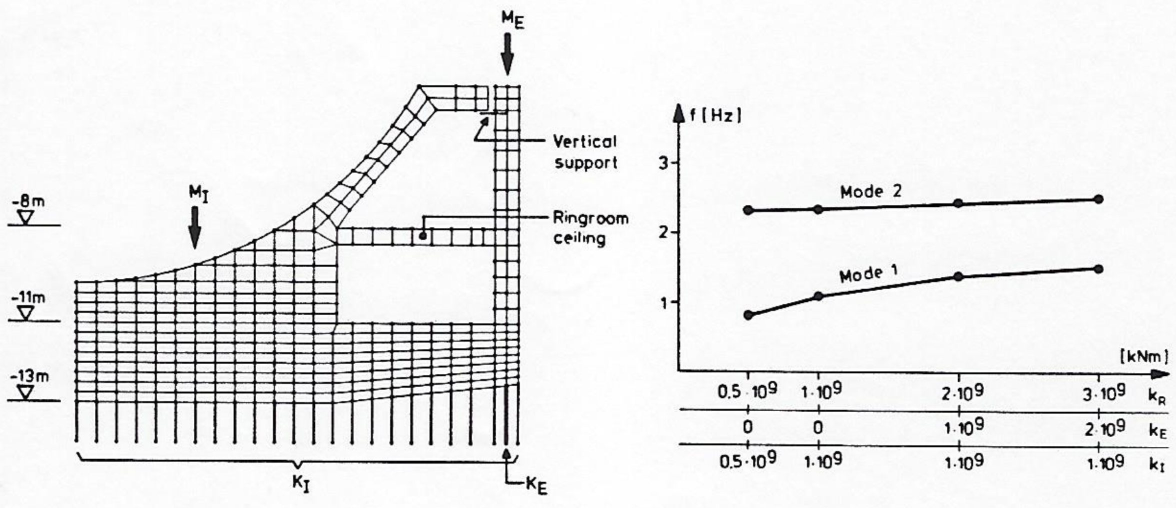
The expected structural response was computed using a relatively simple axisymmetric beam model (Figure 1b) and modal analysis. Nonlinear effects, e.g. the interaction between the structure and the shaker, are included by correction forces on the right-hand-side of the equations of motion.

The analysis starts when both masses are locked in the unbalanced position; i.e. the transition phase is neglected. The equation of motion of the shaker is (see Fig. 1c):

$$m \cdot r^2 \cdot \ddot{\phi} + m \cdot r\,(\cos\phi \cdot \ddot{u}_g - \sin\phi \cdot \ddot{w}_g) + \theta \cdot \ddot{\psi} + \mu \cdot \dot{\phi}^2 = o$$

in which $\ddot{u}$ and $\ddot{w}$ are the accelerations of the shaker shaft, m and θ represent the translational and rotational inertia of the excentric shaker masses, and μ is the coefficient of air resistance. The equations of motion of the shaker and the building are integrated simultaneously using the central difference method.

The stiffness of the foundation structure has a strong effect on the response of the structure. Therefore it is evaluated by an axisymmetric finite element analysis (Fig. 2a) which implies some approximation, since the ringroom ceiling and radial walls are actually not axisymmetric. The soil stiffness is represented by a rocking stiffness k_I, which results from uniformly distributed springs under the foundation slab, and by a rocking stiffness k_E, which is lumped at the edge of the foundation and represents the additional stiffness caused by the embedment of the structure. The total rocking stiffness, $k_R = k_I + k_E$, was



(a) (b)

Figure 2. Axisymmetric model of the foundation (a); eigenfrequencies of the building versus rocking stiffness of the foundation (b)

approximately known from previous low-amplitude shaker tests and dynamic finite element analyses of the soil-structure interaction to be in the range of $2 \cdot 10^9$ - $3 \cdot 10^9$ kNm.

The shaker can excite two global structural modes in each horizontal direction. The first mode shape describes an in-phase motion of the internal and the external structure. Its frequency drops considerably as k_R is decreased (Fig. 2b). The second mode, which corresponds to an out-of-phase motion of the internal and external structure, is only slighly affected by soil-structure interaction.

4 COMPUTATIONAL RESULTS AND STRUCTURAL SAFETY

Results are presented for two tests. The first one, T40.36, has a starting frequency of 5.6 Hz and a small unbalanced mass moment (mass m times excentricity r) of 8200 kgm. A rocking stiffness of $k_R = 2 \cdot 10^9$ kNm is assumed. This corresponds to an eigenfrequency of 1.38 Hz in the first and 2.43 Hz in the second mode. The damping is set to 5% in the first two and 3% in all other modes.

The amplitudes of horizontal accelerations are plotted versus time in Figures 3c,d. The frequency of the shaker decreases with time as shown in Figure 3a. After about 40s and 70s the shaker is in resonance with the second and first structural mode, respectively.

The energy of the shaker and its dissipation is shown in Figure 3b. Most of the energy is dissipated by the air resistance, the remainder by damping in the structure and the soil.

The last test of the series, T40.13, has the largest unbalanced mass moment, 67000 kgm, and a low starting frequency of 1.6 Hz. Hence, only the first structural mode is excited. Because of the large centrifugal forces, a reduction of the effective embedment depth and an increase of soil damping is expected. Therefore the analysis is performed with a reduced soil stiffness of $k_R = 1.18 \cdot 10^9$ kNm and an additional local rocking damper of $2 \cdot 10^7$ kNs. As the shaker passes the first structural eigenfrequency, the acceleration amplitudes drop quickly (Fig. 4).

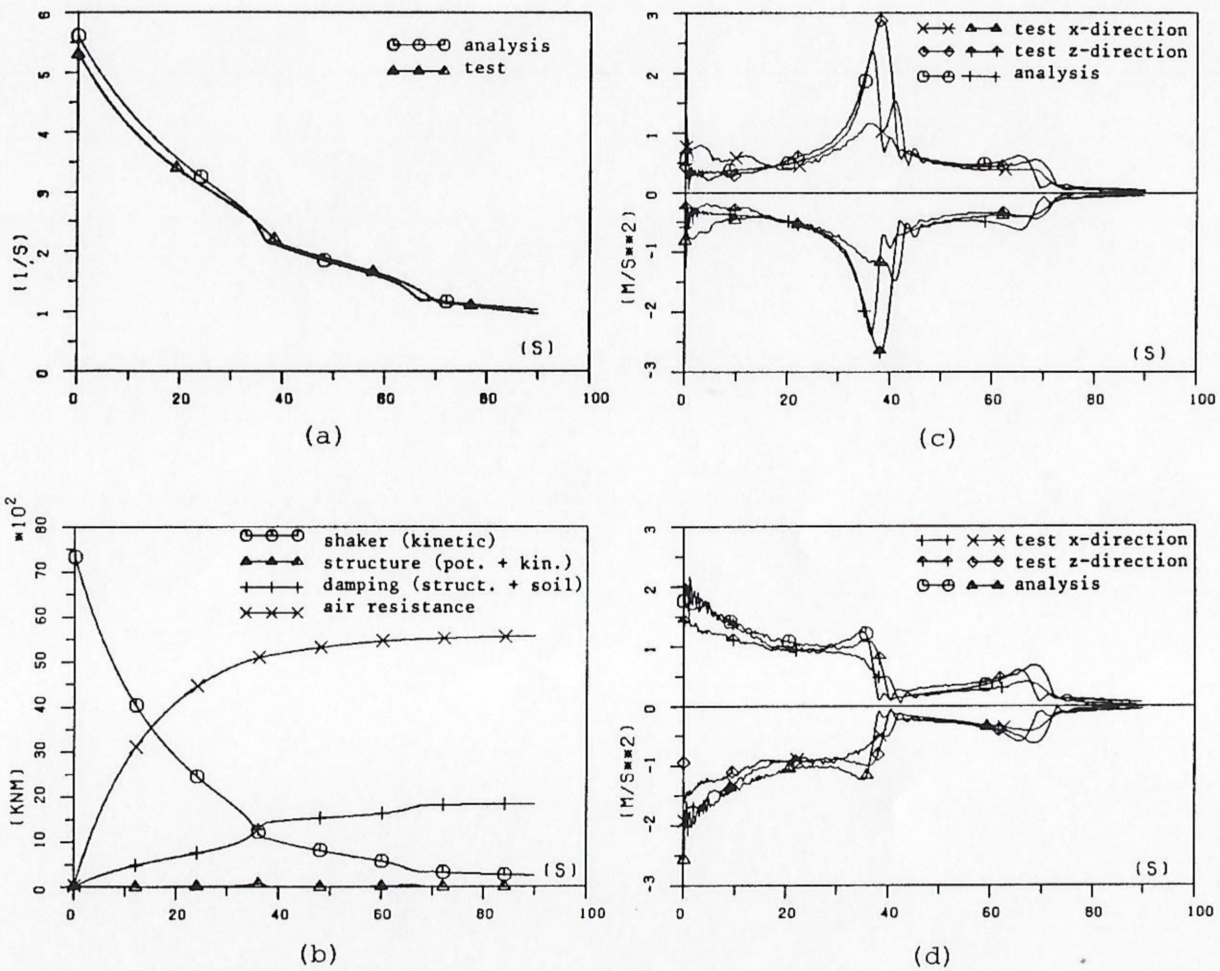


Figure 3. Shaker frequency (a), energy distribution and dissipation (b), peak accelerations at point A (c) and point B (d) versus time for test T40.36

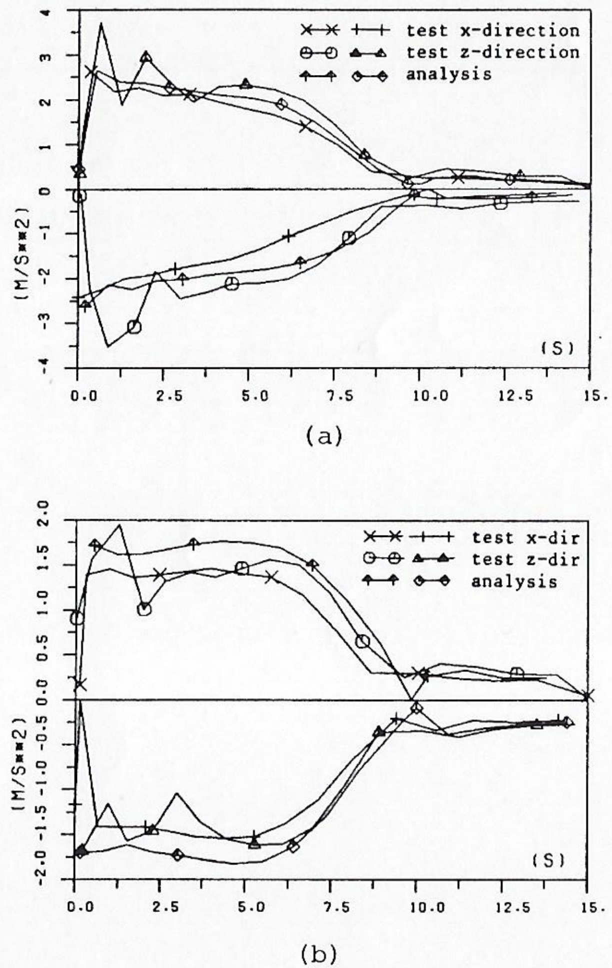


Figure 4. Peak accelerations at point A (a) and point B (b) versus time for test T40.13

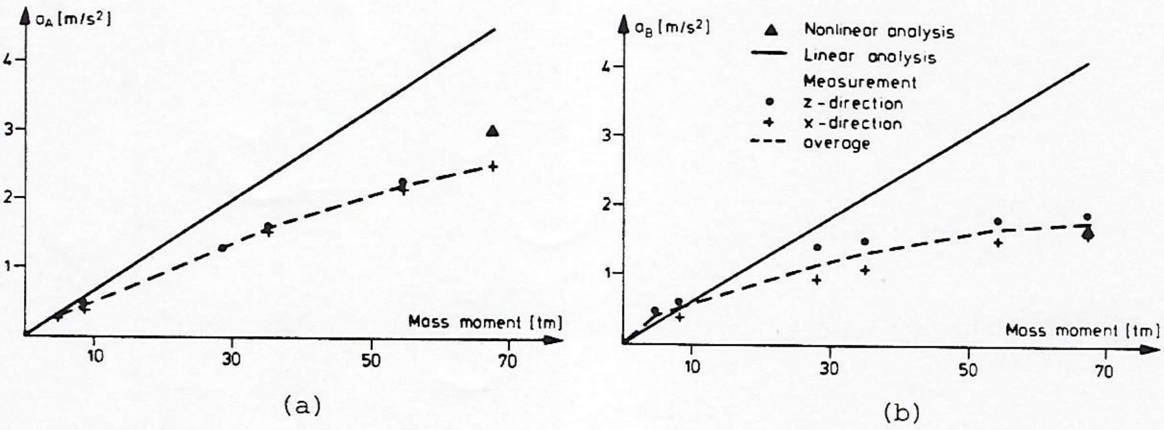


Figure 5. Maximum horizontal accelerations in the fundamental mode versus mass moment of the shaker; external structure, point A (a); internal structure, point B (b)

In order to assess the structural safety, several critical parts of the structure were investigated. It turned out that the maximum allowable loading of the structure is limited by the strength of the external structure. Vertical tensile forces caused by the global overturning moment some distance below ground level are critical. Only a minimum reinforcement, which is sufficient for wind loads, exists in the external cylindrical wall of the structure. The actual strength of the external structure is therefore limited by the tensile strength of the concrete. In order to determine the allowable tensile stress, splitting tests were performed on concrete samples taken from the external wall at and away from horizontal construction joints.

Analysis of the ringroom ceiling and other parts of the foundation structure showed that some local failures could be expected before the external structure would fail. These local failures, however, would not reduce the global structural safety significantly and could therefore be accepted. Another limitation of the allowable load seemed to be a possible sliding of the steel containment in the supporting concrete calotte. Sliding effects were studied using a three-dimensional rigid-plastic element between the internal structure and the calotte. It was found that sliding would increase energy dissipation strongly and decrease accelerations, while displacements due to sliding would stay in an acceptable range.

5 TEST RESULTS

Some results of test T40.36 are shown in Figure 3 together with predicted response values. The frequency of the shaker and the horizontal accelerations of the internal and external structure are well predicted by the analytical model. (Analysis results were obtained prior to the tests.) The values measured in the x- and z-directions reflect the somewhat different properties of the structure in the horizontal directions, which were neglected by the analytical model. If the unbalanced mass moment is increased, while nonlinearities in the structural response and in soil-structure interaction are neglected, the computed accelerations in the first mode increase approximately linearly with the unbalanced mass moment. The test results, however, show a significant nonlinearity (Figure 5). It is essentially caused by nonlinear soil-structure-interaction effects due to the development of a gap between the structure and the laterally surrounding soil and due to nonlinear behaviour of the soil around the foundation. This leads to a decrease of the eigenfrequency and an increase of damping. Similar numerical results for a test with a large unbalanced mass moment were obtained (after the test) with an appropriately modified soil-structure interaction model (see Figure 4 for test T40.13).

After the test T40.13 cracks were observed in the soil around the structure. It was also noted that a gap between the external wall and the soil had developed. Measured strains in the external structure showed that in addition to the global response local effects are important. E.g. a small foundation of a crane gantry next to the external wall of the reactor building restrained the movement of the wall locally and caused considerable bending moments in the wall. However, no failure was observed in the wall. In the ringroom ceiling and the calotte several cracks appeared, as expected. A sliding of the steel containment was not observed.

6 SUMMARY AND CONCLUSIONS

Tests with a heavy shaker were performed at the decomissioned HDR reactor building. The global response of the building was well predicted by the computation methods used. This holds for accelerations, displacements and also for internal forces in relatively uniform structural members. The prediction of local strains was less accurate, because the actual structure is very complex and the stiffness of cracked walls and floors is hard to assess.

The building withstood very strong shaking, similar to that of a severe earthquake, though no horizontal loads but those of wind had been considered in its design. In the last test, a state presumably close to failure was reached. Strongly nonlinear soil-structure interaction effects were observed.

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