Seismic Analysis of Special Emergency Buildings for the NPP Beznau, Switzerland
Part I: Design Analysis

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Part II: Review

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INTRODUCTION

The retrofitting project for the NPP Beznau includes the construction adjacent to the reactor building of two special buildings NANO 1P and 2P housing the emergency mechanical, electrical and piping safety equipment. A dynamic analysis for the earthquake design of these two buildings has been carried out by the project engineers of the firm NOK. Subsequently, this analysis was checked by the Swiss Regulatory Authority and their consulting engineers. This paper is divided into two parts, Part I describing the method and results of the dynamic analysis and Part II the assessment of the seismic analysis.

PART I: DESIGN

SITE AND STRUCTURE

The site of the NPP Beznau is located in the valley of the Aare about 15 Km from the border with W. Germany. The soil consists of a layer, 10 to 13 m thick, of dense alluvial gravel overlying claystone.

The two NANO buildings, comprising monolithic reinforced concrete structures, were erected beside the reactor buildings as shown in Fig. 1. They exhibit differences, especially with regard to their foundations, and due to the required space and the limited space available they are founded at different levels.

CONCEPT AND PRELIMINARY INVESTIGATIONS

The concept for the seismic analysis involved an approximate in preference to a more exact approach to determine the seismic effects. It makes use of relatively simple soil-structure interaction models, which together with additional simplifying assumptions, could be checked by further supplementary investigations. The uncertainties in the analysis were covered by modifying the results appropriately.

Preliminary investigations were carried out to study the influence of adjacent buildings, the effect of wall and slab vibrations and the local differences in the soil profiles for NANO 1P and 2P.

MAIN ANALYSIS

Modelling the Buildings

The NANO buildings were modelled using 3-D linear-elastic beam elements and lumped masses. As may be seen from Fig. 2 the model consists of 7 beam elements and 6 lumped masses. The floors are modelled by rigid elements connecting the mass nodes to the centres of the beams. The auxiliary nodes representing the ends of the corresponding floors are also shown.

Soil Model

The soil was modelled with springs and dampers for each of the 3 displacement and 3 rotational degrees of freedom. The frequency-dependent spring and damping constants, with the exception of the torsional constants, were determined using the program PLUSH (Romo-Organista, et al). A modification of the program PLUSH allowed determination of the transfer functions. The stiffness and damping values for the torsional degrees of freedom were derived from the theory for a layered half space independent of frequency (Christiano).

Using symmetrical finite element models analyses were carried out separately for excitation in the orthogonal X, Y and Z directions with the program PLUSH and a range of frequency 0.1 to 8.0 Hz. The soil was modelled to a depth of 115.00 m with 26 layers. The soil properties (strain-dependent shear modulus and damping, Poisson's ratio and density) had been obtained for the earlier investigations for average soil profiles. In accordance with the conservative design philosophy embedment of the buildings was neglected. For the foundation very stiff massless elements were used. The foundation widths corresponded to the actual sizes. The curves from the PLUSH analysis were modified according to the analytical results for rectangular foundations (Pais & Kausel). Fig. 3 shows the frequency- dependent impedance functions with the model in the X-Z plane for NANO 1P. To obtain the spring and dampings costants for the main analysis, the modal analysis were carried out iteratively, whereby the eigenfrequencies obtained were compared to the estimated frequencies and if they differed the spring constant was adjusted. For example, in the Y-direction the values for 3.9 Hz were used (Fig. 3).

Seismic Input

The seismic input consisted of artificially generated acceleration time histories of length 30 s and a time interval of 0.01 s, which corresponded to the US NRC design spectra. They were scaled for SSE giving maximum values in the X, Y and Z directions of 0.21 g, 0.20 g and 0.15 g respectively and were introduced at the foundation nodes. The SSE values were extracted from the results of free field calculations at the level of the foundation of NANO 2P. For NANO 1P the rock surface is at a higher level, so that the maximum accelerations would be lower. Therefore, use of the same time histories for both NANO buildings is conservative.

Seismic Analysis

A modal analysis in the time domain was carried out separately for both buildings using the program STRUDL. The modal damping was taken as a weighted average of the material damping of the structure and the geometrical damping in the soil following (Roesset et al).

RESULTS AND DESIGN

For the main calculations neither the interaction between the NANO and the neighbouring buildings nor the variation between soft and stiff soil profiles were taken into consideration. Thus these influences were incorporated by modifying the main calculations as follows:

- Dynamic bending moments and shear forces for design purposes were taken from the SSE and OBE calculations and increased by 10 %.

- To determine the relative displacements in NANO 1P and 2P the extreme values at the mass points and the boundaries for each story were found and increased by 30 %.

- To obtain the spectra for a whole story, the response spectra were derived from the time histories at the mass point and the four corner points of the story and for each orthogonal direction the envelopes were determined. All floor spectra in the range 0.1 to 30 Hz were increased by 10 %. To account for structure-structure interaction the vertical spectra were further increased by 50 % at 18 Hz. Finally the spectra were smoothed and the widths of the peaks modified at the eigenfrequencies by $\mp$ 30 %. As a typical result the response spectra for SSE in the X-direction are shown in Fig. 5 for the height 344.00 m above mean sea level.

CONCLUSIONS

The concept outlined above enabled uncertainties in the assumptions to be covered while keeping the costs of analysis within reasonable limits. A further advantage of this method is that the results of the seismic analysis are not sensitive to minor design changes which inevitably occur in the planning phase.

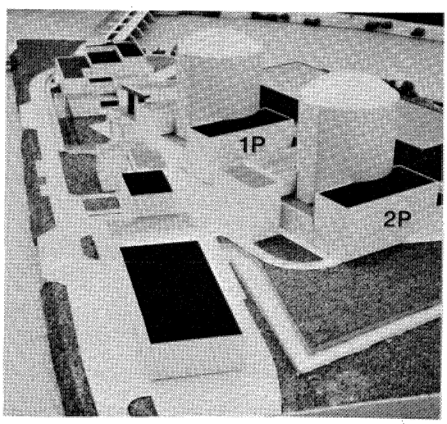

Fig. 1: Site of the two NANO Buildings

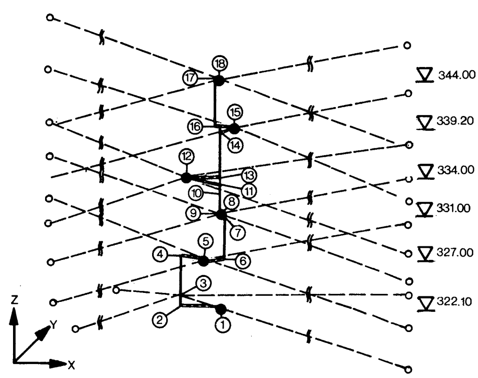

Fig. 2: Model of structure for dynamic analysis

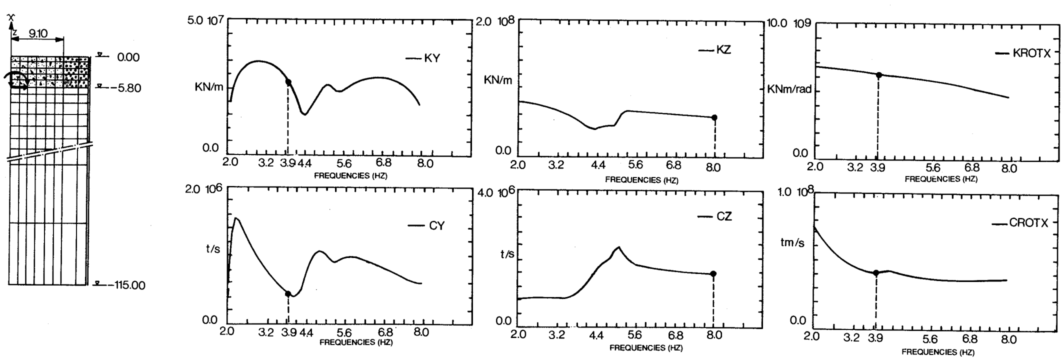

Fig. 3 a) Model of soil layers
b) Frequency-dependent soil spring and damping constants

PART II: REVIEW

FIRST ASSESSMENT OF THE DESIGN ANALYSIS

The licensing authority and its consultant for structural design (reviewer) judged the analysis method as a sound and well reasoned approach. It has the advantage, that the key step in the analysis can be handled very efficiently with a relatively simple structure-soil-model. Modelling uncertainties are taken into account by suitable modifications of the analysis results. However, because the way of establishing the spring and damping parameters for the soil model was a new proposal of the designer, some features of the soil model had to be discussed and verified in detail.

The dynamic soil stiffness computed with the 2-D finite-element model was found to be sensitively dependent on two essential modelling decisions: The equivalent **basemat width** representing the three-dimensional real structure-soil-system and the **depth of the soil profile**. Based on parameter studies with axisymmetric finite-element models the reviewer recommended modifications to the designer's original assumptions. Namely, the profile-depth had to be increased to 109 m below the foundation level which is about 6 - 10 times the equivalent basemat radius.

Finally, the reduction of the frequency dependent dynamic soil stiffness to representative **frequency independent springs and dampers** was a questionable simplification which was required by the applied analysis method. The reviewer decided to judge the influence of frequency dependence by a comparative review analysis with a frequency dependent approach.

REVIEW ANALYSIS

For the review analysis, the substructure method in the frequency domain, as described by Wolf (1985) was applied. The concept of the method is illustrated in figure (4). It is characterized by the substructuring of the complete structure- soil-system in the two subsystems soil with massless rigid foundation and structure with basemat. The method combines the ability of accurately modelling the frequency dependent dynamic soil stiffness with the advantage of an efficient representation of the dynamic behaviour of the building structure by using the modal superposition technique. The main features are briefly summarized in the following.

Subsystem 1: Soil with massless rigid foundation

The finite-element-model for this subsystem is axisymmetric. Transmitting boundaries as in the widely used FLUSH-code (Lysmer et al, 1975) represent the horizontal energy radiation. The motion of the rigid foundation is described by three translational and three rotational degrees of freedom. The solution of the subsystems's equations of motion in the frequency domain provides the frequency dependent dynamic foundation stiffness matrix, which relates the forces to the six displacement components.

Subsystem 2: Building structure with basemat

This subsystem is described with the fixed-base eigenfrequencies and mode shapes obtained from the modal analysis with a 3-D stick-model of the structure. The reviewed model parameters of the design analysis were used for this step. In this analysis the first five modes were found to be sufficient to describe the dynamic behaviour of the building.

Complete System (Mode Synthesis)

The two subsystems are coupled by the six displacement components at the foundation. The complete system results as the synthesis of the subsystems. For this part of the analysis, a new computer code named MOSYME (for Mode Synthesis Method) fitting into the available software library for dynamic structural problems was developed, tested and verified.

The frequency-dependent dynamic stiffness matrix of the complete system is obtained by adding the dynamic stiffness matrices of the subsystems. The excitation of the system is defined by three translational components of design acceleration time-histories in the free field at foundation level. These time-histories, transformed to the frequency domain (Fast Fourier Transform) and multiplied with the system's mass matrix, define the external forces acting on the system induced by the safe shutdown earthquake.

Then the equations of motion are solved for the complete system, taking advantage of the transformation to modal coordinates and the superposition of modal displacements. The equations of the complete system are reduced to the six degrees of freedom of the foundation. The structure's displacements and accelerations are obtained as linear functions of the foundation displacements. Finally, the inverse Fourier transform provides the acceleration time histories from which the floor response spectra are calculated.

Figure 5 shows as a typical example the computed horizontal floor response spectra at the top of the structure model. The spectrum of the review analysis (solid line) is compared with the corresponding response-spectrum of the design analysis (dotted line) and with the smoothed and peak-broadened design spectrum (dashed line). It can be seen, that the design spectrum envelops the computed response spectra not only from the design analysis but also from the review analysis. Slight differences in the frequencies of the dominant modes (rigid-body-rocking) are evident. They can be explained as a consequence of the different approach to modelling the soil.

The comparison of the results of the reviewer's analysis in the frequency domain with the design analysis verifies the use of frequency-independent springs and dampers for this applications in the way they were determined by the designer.

CONCLUSIONS

The substructure method in the frequency domain was introduced as an efficient numerical tool for the review of soil-structure-interaction analyses. The comparison of the analysis results with the results of the design analysis provided reliable arguments for a positive assessment of the design analysis by the licensing authority. It was found for this seismic analysis that a complete consideration of the frequency dependence of the dynamic soil stiffness is not absolutely necessary, as long as the frequency-independent soil model parameters are determined by a reasonable approximation.

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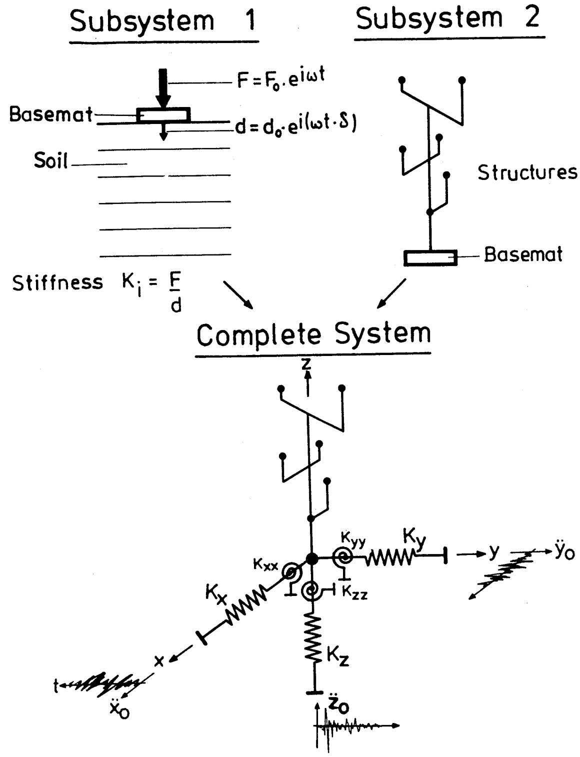

Fig. 4: Substructures and synthesis used for the review analysis.

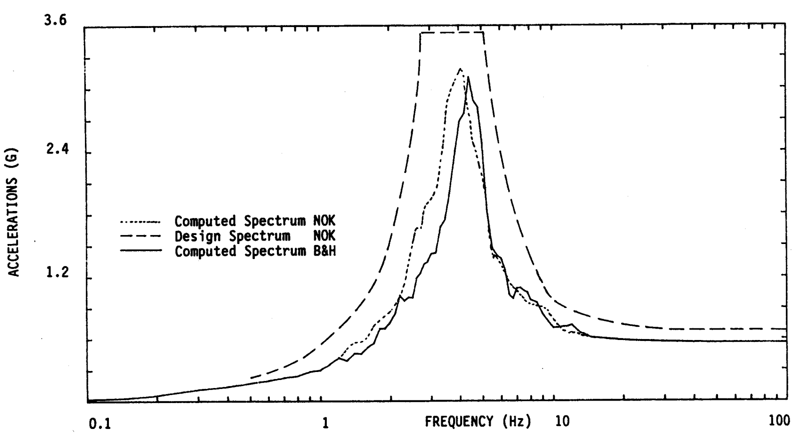

Fig. 5: Comparison of typical floor response spectra from the design analysis (NOK) and the review analysis (B&H): horizontal response to 3-D-SSE-excitation, top of building, damping 5 %.