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SOIL DAMPING EFFECTS ON VIBRATIONS OF NONCIRCULAR FOUNDATIONS

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SUMMARY

Presented in this paper is a study on the effects of limiting of modal damping values for a nuclear power plant switchgear building as example for a building with an arbitrarily shaped foundation. It is shown that time domain computations in connection with limiting of modal damping values may lead to unrealistic results in comparison with frequency domain computations using realistic impedance functions of soil. The use of approximate radiation damping coefficients in modal analysis without limiting of modal damping values leads to good approximations of frequency domain computations and is sufficient for engineering purposes.

INTRODUCTION

In modal analysis computations for the seismic response evaluation of buildings, equivalent frequency independent radiation damping coefficients must be introduced in order to obtain composite modal damping values, which account for both viscous and hysteretic damping components of soil. The limitation of modal damping values in the current German practice of seismic analysis of nuclear power plants to $D = 0.15$ for horizontal and rotational modes of vibration and $D = 0.30$ for vertical modes of vibration, which mainly account for uncertainties in the soil damping coefficients, often leads to unrealistic results, which has been shown by Stangenberg et al. (1987) for buildings with circular foundations. These limitations mainly account for the above mentioned simplifications and uncertainties in the radiation damping coefficients and have to be followed, if no more detailed analyses are carried out.

In this study, the seismic responses of a building with a noncircular foundation resting on an elastic halfspace have been evaluated for soft and hard soil conditions by different methods: (1) time domain computations using modal analysis and different damping assumptions; (2) frequency domain computations using realistic dynamic-stiffness coefficients of the soil.

In case of frequency domain analyses, the dynamic-stiffness coefficients of the soil are combined with the stiffness matrix of the structure. The dynamic-stiffness coefficients of the soil are computed using the algorithms published by Luco and Apsel (1983) for evaluating the Green's functions for a layered half-space, where the frequency-domain response is expressed in terms of semi-infinite integrals with respect to wavenumber, which are solved numerically for a wide range of frequencies.

STRUCTURAL MODEL AND SOIL DATA

The building is modelled by a vertical beam with lumped masses and distributed stiffnesses, which represent the overall mass and stiffness distribution, see Fig. 1. Springs and dashpots attached to the foundation mat (in case of modal analysis) incorporate soil-structure interaction.

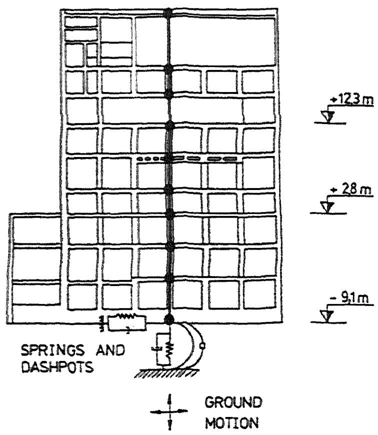

Fig. 1   Structural model

In case of frequency domain analysis, the dynamic-stiffness matrix of the soil is combined with the dynamic-stiffness matrix of the structure to get the total structure-soil system. For the soil, which is assumed to be a homogeneous halfspace, 4 different sets of parameters have been considered, see Table 1.

Soil	Weight Density (kN/m^3)	Hysteretic Damping (%)	Poisson's Ratio	Shear Wave Velocity (m/s)
1 Medium Sand	18	10	0,47	236
2 Dense Sand	20	8	0,45	387
3 Compact Soil	22	5	0,43	564
4 Rock	23,5	1	0,33	1459

Table 1   Soil parameters

TIME DOMAIN ANALYSIS

Three statistically independent artificial acceleration histories compatible with the ground response spectrum (German KTA-spectrum) are generated. The analysis in case of time domain computations is the modal analysis. The spring and (radiation) damping constants are evaluated according to halfspace theory, see e.g. Richart et al. (1970). The total damping is taken approximately as sum of radiation and material damping according to the current practice in modal analysis. In a case study, 2 different assumptions concerning the damping parameters are investigated:

Case 1 Radiation damping constants according to Richart et al. in connection with limiting of modal damping values to D = 0,15 for horizontal and rotational modes

Case 2 Introduction of the following radiation damping coefficients (1) and no limiting of modal damping values:

$$D = \frac{a_o}{2}\left\{\frac{\mu_1}{\gamma_1}\,\frac{\mu_1 a_o^2}{1+\frac{\mu_1^2}{\gamma_1^2}a_o^2}+\gamma_o\right\} \tag{1}$$

where $a_o = \omega b/v_s$ is the dimensionless frequency with b = characteristic length of the building (here half building width), v_s = shear wave velocity and ω = building mode with the dominant displacement in the respective translational or rotational component of motion. The dimensionless coefficients γ_o, γ_1, and μ_1 are taken according to Wolf and Somaini (1986). They depend only on Poisson´s ratio and on the ratio b/l of foundation width/length.

The damping coefficients (1) are in good agreement with the approximate halfspace formulae according to Richart et al. (1970) for 1-dof-systems with equivalent circular foundations. But for multi-dof-systems, this must not hold, because there may be coupling effects of horizontal and rocking modes and coupling effects of soil-structure modes with higher structure modes. The latter effect especially has more influence in case of rock foundations. E.g., for soil 4 of Table 1, the dimensionless coefficients acc. to Wolf and Somaini (1986), Table III, are :

Horizontal: $\gamma_0 = 1.06$, $\mu_1 = 0$
Rocking: $\gamma_0 = 0$, $\gamma_1 = 0.45$, $\mu_1 = 0.34$

Inserting these coefficients and the fundamental building frequency 5.44 cps in eq.(1) leads to the radiation damping ratios D = 0.19 for horizontal and D = 0.03 for rocking motions.

FREQUENCY DOMAIN ANALYSIS

The equilibrium equations for harmonic motion of a rigid foundation of frequency is conveniently formulated as

$$\left[K_s(\omega)\right]\cdot\left\{u_o(\omega)\right\} = \left\{P(\omega)\right\} \tag{2}$$

with the 6 x 6 impedance matrix $[K_s(\omega)]$ and the 6-component vectors $\{u_o(\omega)\}$ of the displacements and $\{P(\omega)\}$ of the applied load amplitude, respectively. After coupling with the mass and stiffness matrix of the superstructure, the equations of motion in the frequency domain can be solved by the complex response method.

The main difficulty is the evaluation of the impedance matrix $[K_s(\omega)]$, which contains the dynamic stiffness coefficients of the six rigidbody degrees of freedom of the arbitrarily shaped foundation.

The foundation area S is divided into n rectangular subregions S_i, c.f. Fig. 2. Assuming that the traction in subregion S_j can be considered constant, the average displacement in subregion S_i can be expressed according to Wong and Luco (1976) as

$$\{u_i\} = [G_{ij}]\ \{P_j\} \tag{3}$$

in which G_{ij} is the 3 x 3-matrix of Green's functions

$$[G_{ij}] = \int_{S_i}\int_{S_j} G(\bar{x}-\bar{\xi})\, dS_i(\bar{x})\, dS_j(\bar{\xi}) \tag{4}$$

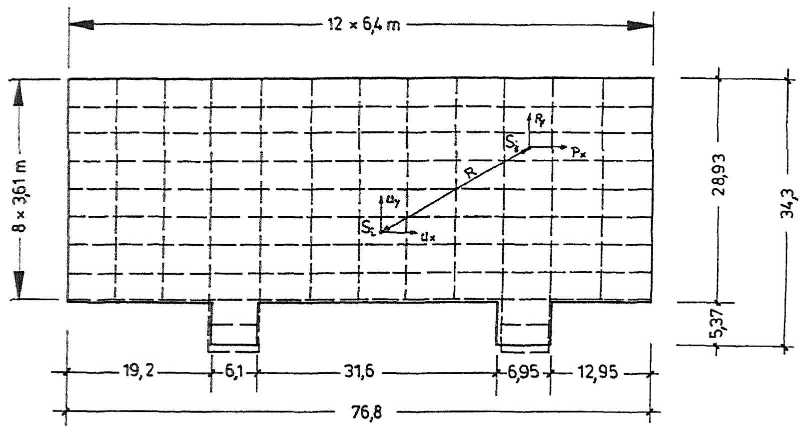

Fig. 2 Illustration of discretization of arbitrarily-shaped foundation

The elements of the Green's functions matrix $[G_{ij}]$ are evaluated by numerical integration of semi-infinite integrals with respect to wavenumber using the algorithms given by Apsel (1979) and Luco and Apsel (1983). The matrices $[G_{ij}]$ for all subregions are assembled to the 3n x 3n matrix $[G]$, and together with the 3n displacement and load vectors $\{u\}$ and $\{P\}$, respectively, the equation of motion is

$$\{u\} = [G] \{P\} \tag{5}$$

For a rigid foundation, the generalized displacement vector $\{u_o\}$ is defined by

$$\{u\} = [\alpha]\{u_o\} \tag{6}$$

where $[\alpha]$ is the 3n x 6 matrix connecting the average motion of each subregion with the rigid-body degrees of freedom of the foundation. After inserting the vector $\{u\}$ in eq. (5), the load vector $\{P\}$ can be determined by solving the system of equations (5). The impedance matrix $[K]$ then follows from equation (2).

The dynamic stiffness coefficients for horizontal and rocking degrees of freedom of the foundation of Fig.2 for motions in lateral direction are plotted in Fig.3 . They are normalized to $G \cdot b$ (horizontal) and $G \cdot b^3$ (rocking) where b = foundation width.

RESULTS

From the various results, only a choice of selected results can be given in this paper. Table 2 summarizes internal forces due to horizontal earthquake (shear forces Q, bending moments M) at the inner structure +12,3 m and +2,8 m as well as base shear and base moment. All values are mean values from the different history results.

The performed parameter investigations show, that the modal analysis case 2, i.e. radiation damping coefficients according to eq. (1) in connection with no limiting of modal damping values, leads to a good approximation of frequency domain computations with the realistic impedance functions of soil whereas the computations with limiting of modal damping values (case 1), are either too conservative or, in special cases, may be even not conservative, see Table 2.

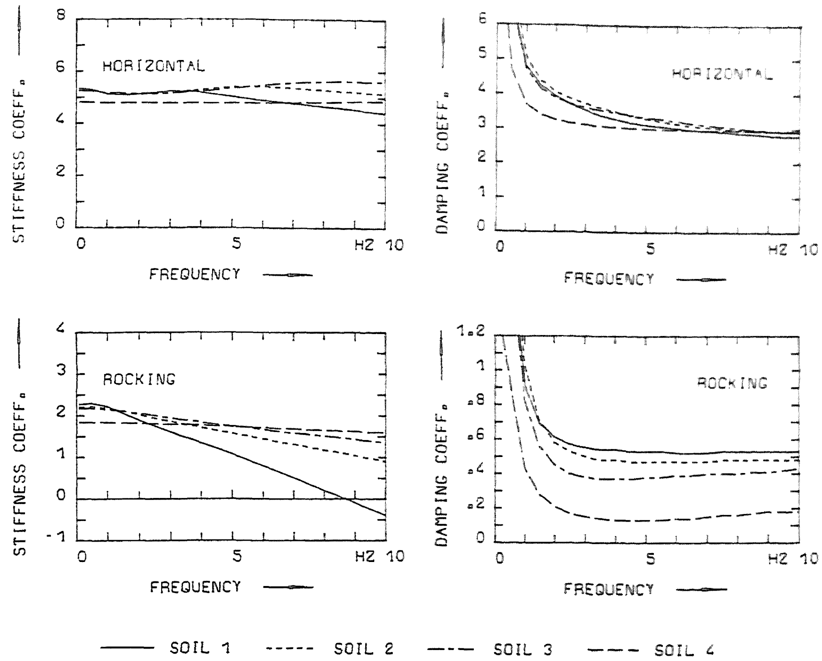

Fig. 3   Dynamic-stiffness coefficients

	Soil 1		Soil 2		Soil 3		Soil 4	
	case 1	case 2	case 1	case 2	case 1	case 2	case 1	case 2
Q +12,3 m	1,549	1,019	1,404	1,083	1,109	1,079	0,905	1,054
M +12,3 m	1,417	0,921	1,330	1,000	1,026	0,961	0,890	1,028
Q + 2,8 m	1,431	0,990	1,276	1,012	1,059	1,036	0,880	1,018
M + 2,8 m	1,432	0,935	1,309	1,005	1,025	0,996	0,875	1,015
Q - 9,1 m	1,213	0,937	1,204	1,011	1,056	1,034	0,934	1,013
M - 9,1 m	1,451	0,963	1,270	0,998	1,036	1,011	0,874	1,008
Mean Value	1,416	0,961	1,299	1,018	1,052	1,020	0,893	1,023

Table 2   Normalized Internal Forces (frequency domain analysis = 1,0)

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