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PRA of NPP Structures Under Earthquake Conditions — Structural Analysis

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ABSTRACT

This paper contains the second part of a seismic PRA, i.e. the structural reliability analysis. Based on the assumption of linear structural properties the structural analysis is carried out in the frequency range. The resulting failure probabilities of a sample structure, - a containment has been chosen for this purpose - revealed values which are considerable lower than the generally accepted in modern probability based codified design. In order to ensure a connection with deterministic design, conversion procedures to establish relations between power- and response spectra have been introduced as well. The results of the analysis provide pertinent information for a credible risk assessment for the entire NPP.

1. INTRODUCTION

The PRA of a NPP structure can be carried out in two steps, i.e. first the seismic risk analysis and second the structural reliability analysis. While the first part has been described in [1], this paper is concerned with the structural reliability analysis part. However, a strong interrelationship between both parts of the study - which is mainly due to the necessity of introducing simplified models - has to be taken into account. The most essential simplification concerning the structural analysis part is the mathematical description of the structure as a system of linear equations leading to an eigenvalue problem. The considerations of non-linear effects, for example of structural stiffness properties, or soil damping characteristics would require rather complex comprehensive probabilistic analysis. In a first step, such effects are taken into account by introducing "conservative" assumptions. For example considerable care has to be applied when modeling the damping parameter in context with an adequate treatment of the energy considerations (absorption, etc.).

2. THE PROBABILISTIC CONCEPT

For the mathematical formulation of the probabilistic concept as used here it is refered to section 2 of the previous paper [1]. There it has been stated that the properties of the loading due to earthquake excitation may be approximated by a weakly stationary, Gaussian process. Furthermore assuming a linear (structural) system, it is a wellknown fact that these properties are maintained for the response process as well. Hence the PDF of the load effect - consisting of the mean value of the dead load and the standard deviation of the earthquake loading - is fully determined. The latter may be obtained by integrating the PSD of the response, $G_S(\omega)$, i.e. by

$$\sigma_S = [\int_0^\infty G_S(\omega)d\omega]^{\frac{1}{2}} \tag{1}$$

Applying the modal analysis, the standard deviation may be expressed by the modal variances $\sigma^2_{S,ii}$ and covariances $\sigma^2_{S,ij}$, respectively:

$$\sigma_S = [\sum_{i=1}^{N} \Psi_i^2 L_i^2 \sigma^2_{S,ii} + \sum_{i=1}^{N} \sum_{\substack{j=1 \\ j\neq i}}^{N} \Psi_i \Psi_j L_i L_j \sigma^2_{S,ij}]^{\frac{1}{2}} \tag{2}$$

- where Ψ_i is the modal shape of the displacement and load effect respectively and L_i is the load participation factor (as defined by equ. (4)) - which, according to equ. (1) may be obtained by the auto-and cross-spectral densities $G_{S,ii}(\omega)$ and $G_{S,ij}(\omega)$. The cross spectral density for the modal shapes i and j is

$$G_{S,ij}(\omega) = H_i(\omega)G_A(\omega)H_j^*(\omega) \tag{3}$$

where H_i is the transfer function of the i-th mode, H_j^* the complex conjugate and G_A the PSD of the ground acceleration. The load participation factors are defined by

$$L_i = \frac{\phi_i^T M}{\phi_i^T M\phi_i} \tag{4}$$

where ϕ_i is the eigenvector of the mode shape and M the mass matrix. In this context it should be noted that for cases where the eigenfrequencies reveal significantly different values, the cross spectral densities are considerably smaller than the auto power spectral densities and consequently the second part of equ. (2) may be neglected [2].

3. CALCULATION OF FLOOR RESPONSE SPECTRA

For the purpose of the structural design procedure of various details of interest it is quite frequently of advantage to utilize constant load effects, i.e. deterministic models while stochastic considerations are particular suitable for an overall assessment of structures with respect to their optimum design, etc. [3].

In practical application, the so-called Floor Response Spectra (FRS) are utilized for the design of internal structures and substructures, respectively. Procedures for approximation of relation between power- and response spectra are available. These methods are generally based on extreme value (peak factor) consideration (see for example [4,5]). Consequently the FRS may be directly derived from the PSD without carrying out two seperate - i.e. one deterministic and one stochastic - analyses, which would be required otherwise.

4. CASE STUDY

4.1 Structural Analysis

The case study as carried out here is based on the results as described in section 3 of [1]. In order to be able to assess the effects of the various approximations for load - as well as structural analyses the overall PRA has to be carried out in several steps. The modal analysis is most advantageously applied to assess the dynamical behavior of the particular structure under consideration. This is mainly due to the transparent relationship between load and load effects (see equ. (1) - (4)). Load participation factors are a first but fundamental information on the dynamical sensitivity of the structure. Hence a conservative approach to define the loading function is feasible or, - for cases where a definite description of the loading function is available - the structure may be optimized for minimum load effects. In the present study (see also Fig. 7 in [1]) a conservative load spectrum has been chosen. The dynamic properties of the sample structure - a containment - are listed in Table 1. The dominance of the first two mode shapes of the loading, which may be concluded from the modal variances, is a direct consequence of the choice of the load spectrum.

4.2 Failure Probability of the Structure

A cross section located in the transition between the spherical dome and the cylindrical shell of the reinforced concrete shielding is analyzed as an example (see No.1 of Fig.1). Due to the additional design against aircraft impact a disproportional large wall thickness of 2.0 m is required. The bending reinforcement, however, is relatively small, which leads to the fact that the failure of the bending reinforcement is the expected failure mechanism. Hence, the random variable of significance is the axial force of the reinforcement, which is modeled by a normal distribution with a coefficient of variation of 0.055 and a mean value of 602 N/mm^2.

The conditional failure probability, given the earthquake event E is shown in Fig.2, for a vertical and horizontal cross section respectively. For a design life of 40 years the failure probability $F(t) = 1 - L(t)$ is shown in Table 2 for two different load cases.

4.3 Floor Response Spectra

Utilizing the procedure suggested in [4] the FRS have been calculated from the PSD for the absolute values of the structural accelerations. The results for a particular cross section (No. 2) are shown in Fig. 3. The FRS are scaled to a maximum horizontal ground acceleration of 4.2 m/s^2. In order to include the entire expected motion the conversion has been based on a duration of 30 sec. The expected strong motion duration for the region under consideration is about 15 - 20 sec. The most essential approximation in the conversion of PSD into response spectra is the assumption of the weakly stationary Gaussian response properties. It has already pointed out that in some cases this assumption may not necessarily reflect reality [6].

5. CONCLUSIONS

The PRA concept for NPP structures under earthquake excitation as suggested in this paper and in [1] proved to be quite useful for practical application. The interaction between the seismic risk- and structural reliability analysis parts has been pointed out quite clearly. Since the structural analysis is based on the assumption of linear behavior, the application of random vibration techniques prove to be quite useful. If, however, nonlinear effects have to be included, either equivalent linearization techniques, time history analyses, etc. have to be applied. The failure probabilities of critical cross sections of the structure under consideration are well below those values which are generally accepted in codified design. The procedure as suggested here forms a sound basis for a rational i.e. optimum design of NPP structures. Moreover it provides part of the information needed for a credible risk assessment of an entire NPP.

MODES	NATURAL FREQUENCY [Hz]	DAMPING [%]	LOAD PARTICIPATION FACTOR	MODAL VARIANCE $10^{-6}[m^2]$
1	1.06	15	331	30.334
2	1.94	15	435	8.860
3	5.03	7.5	181	63
4	5.73	2.4	3	51
5	8.78	7.1	66	5

Table 1:
Dynamic Properties of NPP-Structures Under Earthquake Excitation

CROSS SECTION	EXCITATION	
	According Fig. 5 of [1] *	According Fig. 5 of [1] **
horizontal	$< 1.0 \cdot 10^{-10}$	$< 1.0 \cdot 10^{-10}$
vertical	$8.6 \cdot 10^{-6}$	$3.9 \cdot 10^{-5}$
earthquake occurrence rate ν events per year	28.7++	11.9+

++ lower boundary of magnitude = 3.0 (max.accel. ≥ 20 cm/s^2)

\+ lower boundary of magnitude = 4.0 (max.accel. ≥ 50 cm/s^2)

* attenuation law E

** increased seismic activities in regions III and IV

Table 2:
Failure Probability $F_T(t)$ of Cross Section Under Consideration for a Design Life of 40 years

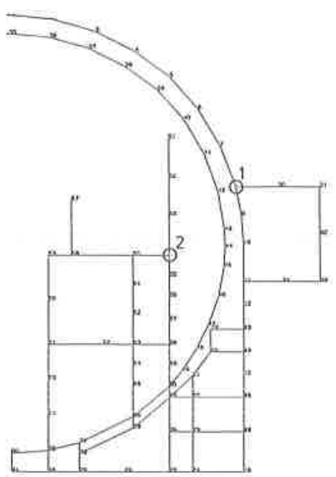

Fig.1: Sample Structure Containment

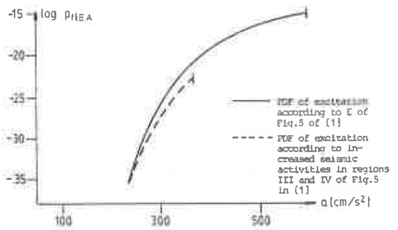

Fig.2a: Conditional Failure Probabilities of the Horizontal Cross Section No. 1

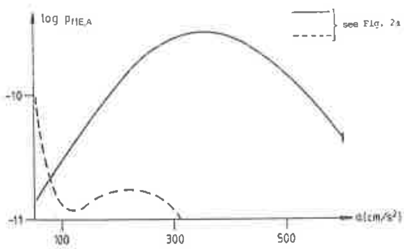

Fig.2b: Conditional Failure Probabilities in the Vertical Cross Section No. 1

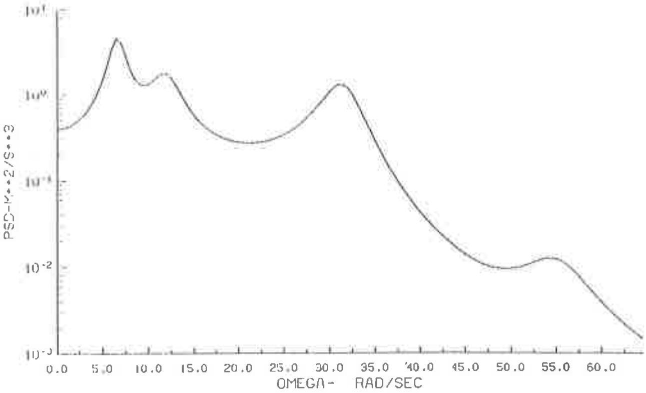

Fig.3a:
Power Spectrum of Absolute Acceleration in Horizontal Direction - Cross Section No. 2

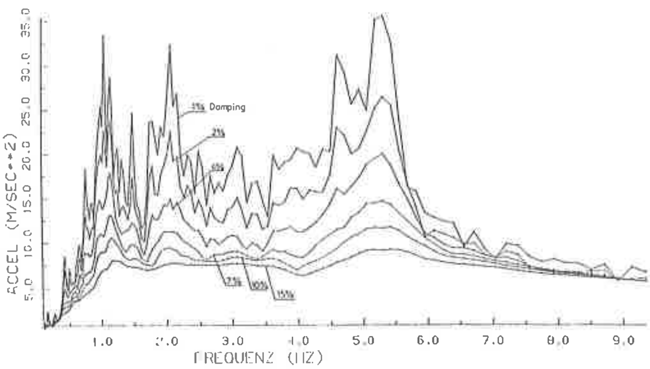

Fig.3b:
Response Spectrum, Horizontal Components - Cross Section No. 2

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