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Comparative study for methods to determine the seismic response of NPP structures

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ABSTRACT: There are many different important problem areas in evaluating the seismic response of structures. In this study the effort is concentrated on three of these areas.

The first task is the mathematical formulation of earthquake excitation. The random vibration theory is taken as the tool in this task.

The second area of interest in this study is the soil-structure interaction analysis. The approach of impedance functions is chosen and the focal point of interest is the significance of frequency dependent impedance functions.

The third area of interest is the methods to determine the structural response. The following three methods were tested: 1) The mode superposition time history method; 2) The complex frequency-response method; 3) The response spectrum method. The comparison was made with the aid of MSC/NASTRAN code. The three methods gave for outer containment building response results which were in good agreement with each other.

1 INTRODUCTION

This study investigates the response of structures to seismic excitation from the point of view of comparative presentations 1) for the random process nature of the earthquake excitation; 2) for the impedance function formulation of the soil-structure interaction phenomenon and 3) for the numerical methodological strategies for determining the response.

2 RANDOM VIBRATION APPROACH FOR EARTHQUAKE EXCITATION

The usual power spectrum expression for earthquake excitation is (Filtered With Noise) FWN expression. The uniformly modulated evolutionary excitation is obtained when the stationary FWN is multiplied by the time dependent modulation function. A more general evolutionary presentation is obtained when stationary FWN is multiplied by the time and frequency dependent modulation function. In this chapter of the study we investigate the response of the example structure given in Figure 1 for uniformly modulated FWN excitation and also for evolutionary excitation with time and frequency dependent modulation function.

The structural response is determined in terms of quadratic mean or variance for the response processes. The variance functions are calculated for the horizontal response of points C and F which correspond to second and fifth degree of freedom of the finite element model of the structure. The variance function is calculated in 81 points over the time interval of 20 seconds. Only the contribution of three lowest modes has been taken into account in the response calculations. The response calculations were carried out with the aid of STOCAL-program (Wung, 1989). The variance functions for DOF5 and DOF2 are depicted in Figure 2 for uniformly modulated FWN excitation. The variance functions of DOF5 and DOF2 were calculated also for WN excitation modulated with time and frequency dependent modulation function. The general appearance of these variance functions is the same as those in Figure 2.

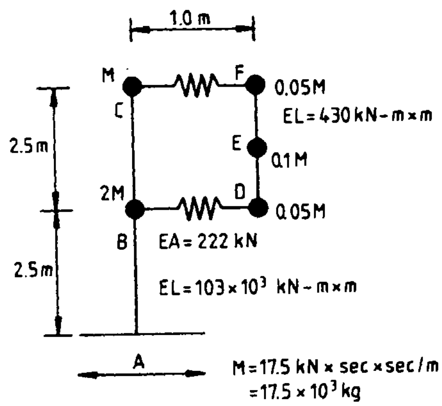

Figure 1. The example structure investigated for random vibration excitation

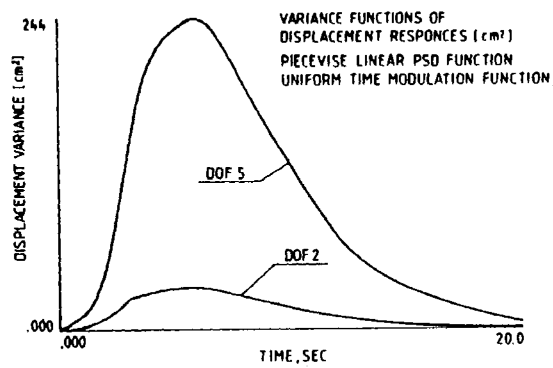

Figure 2. Variance functions of displacement responses of the example structure

3 SOIL-STRUCTURE INTERACTION (SSI) FOR THE EARTHQUAKE EXCITATION

The soil-structure interaction effect will be illustrated with the aid of an application example which describes the seismic response of the reactor building of the nuclear power plant. The foundation slab of the reactor building is embedded in the sandy soil and the seismic excitation is determined as the motion of the free-field. The soil properties are defined by following parameters: G = shear modulus = 450 MPa; Poisson ratio = 0.4; weight density = 21.2 kN/m^3; embedment depth = 12.2 m; radius of base = 22.9 m; depth to bedrock = 305 m and soil damping ratio = 10%. The geometry of the structural model of the reactor building is depicted in Figure 3.

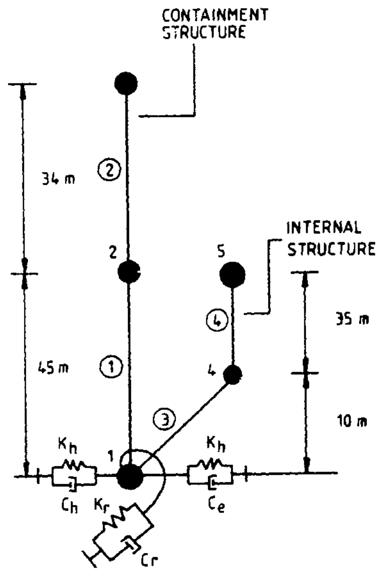

Figure 3. The soil-structure interaction model.

The free-field acceleration time history consistent with the USNRC Reg. Guide 1.60 (USNRC, 1973) spectrum is the loading of the model. The SSI-analysis takes place in the frequency domain and the acceleration time history is Fourier transformed before the analysis. The results of the SSI are the acceleration time histories and in-structure response spectra in the nodal points specified by the user. The node where the response is plotted is node 5 in this example. For this node the horizontal response spectra with 5% damping ratio is evaluated. The in-structure response of the SSI-analysis was developed using the CARES-program (Xu, 1990). There are four different SSI-models in CARES, namely, Kausel, Veletsos and Beredugo-Novak models and frequency independent soil stiffness and damping constants. The results for Beredugo-Novak model and frequency independent model are depicted in the same figure in order to facilitate the comparison of the results. In Figure 4 the horizontal response spectra are plotted for the node 5. The response spectra values are given as fractions of gravitational acceleration (g). The zero period acceleration of the input acceleration time history was 0.2 g. It can be observed from the Figure 4 that the use of frequency dependent impedance functions reduces the response. When the results for all four SSI-models are developed it can be seen that the results of Veletsos model are very near to the results of frequency independent model and that the Beredugo-Novak model gives about 20% lower results than the frequency independent model. The lowest results are obtained from Kausel model. It gives spectral values that are 34% lower than values of the frequency independent model.

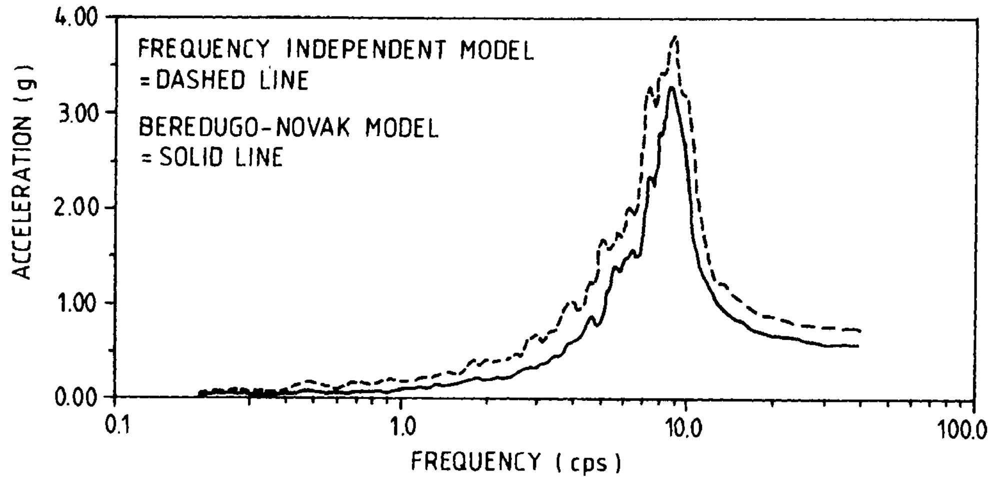

Figure 4. Spectral acceleration in node 5
Solid line = frequency independent model
Dashed line = Beredugo-Novak model

4 THE STRUCTURAL RESPONSE DETERMINATION METHODOLOGIES

The seismic analysis is usually carried out for three mutually perpendicular components of motion. Two of these components are horizontal and one is vertical. The directions of the motion are usually assumed to coincide with the principal axes of the building. The response of the multidegree of freedom system when exited by seismic load can be expressed with the aid of the following differential equation.

$$[M][\{\ddot{X}\}+\{\ddot{U}_g\}]+[C]\{\dot{X}\}+[K]\{X\} = \{0\} \tag{1}$$

where
$[M]$ = the mass matrix (nxn)
$[C]$ = the damping matrix (nxn)
$[K]$ = the stiffness matrix (nxn)
$\{X\}$ = the relative displacement vector (nx1)
$\{\dot{X}\}$ = the relative velocity vector (nx1)
$\{\ddot{X}\}$ = the relative acceleration vector (nx1)
$\{\ddot{U}_g\}$ = the ground acceleration vector

Equation (1) can be solved using the time history method based on the mode superposition. In mode superposition method the linear equation system (1) is decoupled using the transform to the so called generalized coordinates. The decoupled equations are then solved independently of each other.

In the response spectrum method generalized response in each mode is determined from the expression

$$Y_j(\max) = \Gamma_j(S_{aj}/\omega_j^2) \tag{2}$$

where S_{aj} is the spectral acceleration corresponding to the frequency ω_j and Γ_j is the modal participation factor for mode j (ASCE, 1986). The maximum displacement of node i relative to base due to mode j is obtained by multiplying expression (2) with the ith component of mode shape vector j. There are many different methods to combine the contributions of the individual nodes most notably SRSS method and CQC method (Wung, 1989).

When the complex frequency-response method is used, the response time history, R(t), may be expressed as:

$$R(t) = \frac{1}{2\pi}\int_{-\infty}^{\infty} R(\omega)\exp(i\omega t)d\omega \tag{3}$$

where $R(\omega)$ is the response in the frequency domain and is given by

$$R(\omega) \text{ is } T(\omega)\cdot\ddot{U}_g(\omega) \tag{4}$$

where $T(\omega)$ = transfer function for the structure at circular frequency ω

$$\ddot{U}_g(\omega) = \int_{-\infty}^{\infty}\ddot{U}_g(t)\exp(-i\omega t)dt$$

There exists many methods to calculate the transfer functions. Most notably of them are described in ASCE standard (ASCE, 1986).

As an application example we investigate the seismic response of VVER-91 type reactor building and its outer containment. The response was determined using three above mentioned methods. The profiles of the horizontal acceleration determined as described above are given in Figure 5.

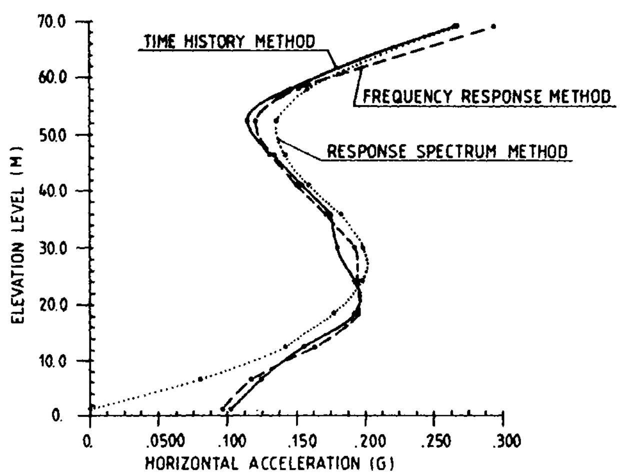

Figure 5. Seismic response of VVER-91 outer containment
Solid line = Time history method
Dashed line = Complex frequency-response method
Dotted line = Response spectrum method

5 CONCLUSION

Based on the results described above the following conclusions seem to be warranted. The random vibration theory especially the advanced evolutionary spectrum presentation for earthquake excitation yields a very effective apparatus for developing statistical measures of structural response. The soil structure interaction models give results which vary in a quite vide range. It appears that the frequency dependence of impedance functions should be taken into account whenever possible. The experimental validation of the impedance function models by means of comparison of the numerical and experimental results would be most welcome.

In judging of the methods for determination of the structural response the time history method, the complex frequency response method and the response spectrum method seem to give very similar results. In its current form the response spectrum method is not capable to correctly describe the acceleration at the base of the structure.

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