PROBABILISTIC SOIL-STRUCTURE INTERACTION ANALYSIS

ZEMİN-YAPI ETKİLEŞİMİNİN PROBABİLİSTİK ANALİZİ

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ABSTRACT

The probabilistic risk analysis (PRA) of critical structures for seismic excitation is a difficult task but very important especially for existing buildings. The soil-structure interaction analysis in probabilistic aspect is a part of the PRA. An original procedure is developed combining the peculiarities of WWER 1000 reactor building and the requirements of the probabilistic safety analysis. On the base of uniform hazard spectra the free-field seismic motion in form of acceleration time histories is generated. The probabilistic definition of the seismic input motion at the structure foundation level is obtained taking into consideration the modifying effect of the local soil conditions. A 3D finite element model of the structure including the soil stiffness and damping elements is developed. Mean response and variation of response parameters are determined. The Monte Carlo simulation technique and Latin Hypercube Experimental Design procedure are applied. The soil-structure interaction effect is analyzed in probabilistic format.

INTRODUCTION

In the last two decades the probabilistic risk assessment (PRA) is performed for critical structures such as nuclear plants, large dams, etc. The important question is what is the potential accident risk for critical structures near high population concentration. To answer this question a probabilistic risk analysis should be applied. Seismic PRA has a purpose to estimate seismic risk. It gives information on seismic capacity in a probabilistic format. In the seismic PRA except the seismic risk the frequency of core melt is given as well as lowest capacity elements are identified and component fragility parameters are estimated. The goal of PRA is to develope distribution of in-structure response spectra and structural forces for selected earthquake levels.

A probabilistic risk analysis is under way for the main buildings of Units 5/6

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of NPP Kozloduy. The first phase of investigations is to assess probabilistically the seismic response of the buildings. For this purpose an original procedure that has to match both requirements of the PRA and peculiarities of WWER 1000 structural design is developed [1,3,4,5,8,9,10]. Probabilistic definition of seismic excitation is achieved taking into consideration the uncertainties. Probabilistic in-structure response spectra are generated and mean values and variations of response for the important structural elements are determined.Then fragility analysis is performed. The Monte Carlo simulation technique is used. For reducing the amount of computational work the Latin Hypercube Sampling Procedure is applied. Computar codes SHAKE, SIMQKE, STARDYNE, SMACS are used.

First step of the analysis is the seismic hazard assessment and the definition of the excitation levels. The earthquake distribution in the 320 km area around the site is taken into consideration. The available tectonic and seismological studies and the respective results are used. The results of the seismic hazard study are hazard curves giving the peak acceleration values vs annual probability of exceedance and uniform hazard spectra at 5% damping for different levels of probability. For both types of curves mean and standard deviation values are given as well as median and geometric deviation (log-normal distribution) are assessed. The uniform hazard spectra represent the free-field ground motion at the site.

The second step is the fragility analysis. This analysis needs probabilistic determination of the response of soil, structure and different components. The soil-structure interaction (SSI) is an important part of this study. The SSI elements are: foundation input motion, foundation impedances, structural model. The foundation input motion should be determined by the free-field motion taking into consideration the local soil profile with characteristics at low strain and strain compatible properties.

The probabilistic soil-structure interaction analysis as a part of the PRA discussed above is performed shortly in the paper giving some of the results necessary for the future study of the problem.

COMPUTATIONAL ACCELEROGRAMS

For the sake of further response analysis the seismic excitation is assumed to be presented by means of accelerograms. For lack of sufficient number of real accelerograms recorded at the site the accelerograms should be generated. The generation process is performed on the base of the uniform hazard spectra. Because of the limitation of time and the costs for analysis the number of accelerograms is selected to be ten for each level of annual probability of excidence. The levels of E-3, E-4 and E-5 are chosen to be representative for the hazard of the site. It is assumed that they will give the maximum contribution to the probability of failure and probability of a core melt. The artificial accelerograms shoud have all peculiarities of real records. The selection of real accelerograms recorded mostly in other countries [2,7] is done on the base of the criterion for similarity of different parameters (magnitude, focal depth, epicentral distance, etc.).

Statistical analysis is performed over 90 pre-selected accelerogram components (60 horizontal and 30 vertical) divided in three groups corresponding to the distribution of seismic sources around the site under investigation - near field sources (epicentral distance R up to 30 km, maximum magnitude M_{max} = 4,5), intermediate distance sources at shallow depth (R = 80 to 220 km and M_{max} = 8) and long distance sources in Vrancea region at intermediate depth (R = 240 to 330 km and M_{max} = 7.8). All records have been done at places with geology similar to the local geology. For each group 10 three components accelerograms are processed - the mean and mean + 1 sigma values of their acceleration response spectra are determined. The horizontal components are given in Fig. 1. In the third group the Vrancea 1977 earthquake records are not included because there are only two records and the N-S component recorded in Bucharest is extraordinary with very long predominant period. This peculiarity can be seen in the last sub-figure (Fig.1).

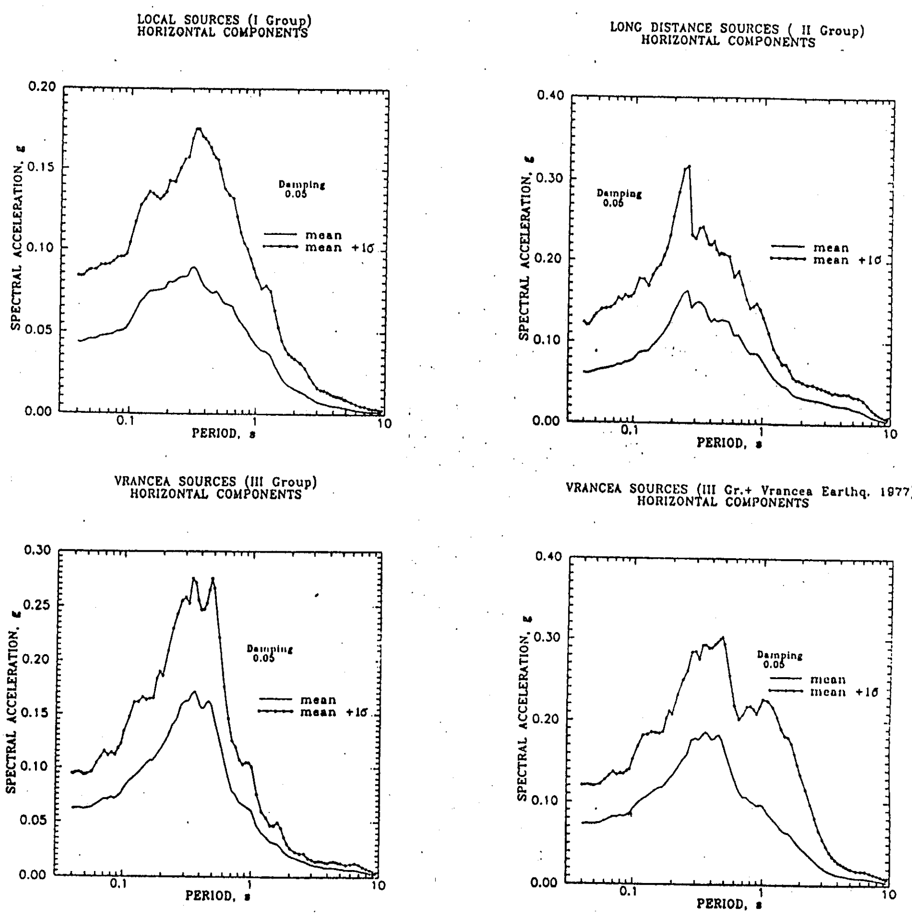


Figure 1. Acceleration response spectra of real accelerograms

In the response anlysis the Vrancea 1977 earthquake records are used separately. The analysis of the mean spectra for all groups shows similarity in the frequency content of the horizontal and vertical components.

For each group of real records the ratios of the maximum acceleration values of the two horizontal components as well as of the vertical and horizontal components are determined (mean values and standard deviation). On the whole the first ratio (mean value) is about 1 and the second - about 0.5.

Using the uniform hazard spectra for the chosen three levels of hazard for each level 10 spectra are generated. For the given mean and standard deviation values generation of log-normal distributed numbers are obtained applying the Latin Hypercube Experimental Design (LHCED) procedure. Some of the generated spectra and the uniform hazard spectrum for annual probability of exceedance 0.001 are shown in Fig.2a. The generated spectra (10 for each level) are used for generation of accelerograms - for each spectrum three independent accelerograms are determined. They are corrected with coefficients obtained by LHCED procedure. The acceleration response spectra of those accelerograms are computed and their mean values and the respective standard deviation values match the mean values and the variation of the uniform hazard spectra. The comparison between generated and target mean spectrum and the corresponding standard deviation for hazard level E-4 is shown in Fig.2b.

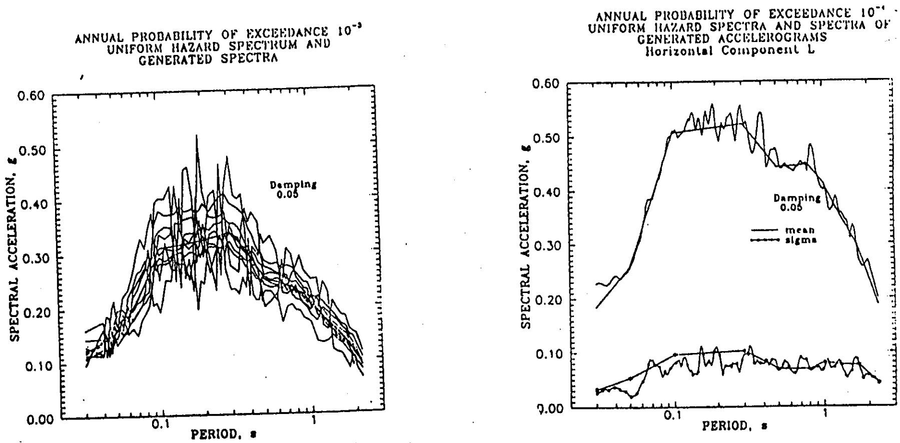


Figure 2. Free-field acceleration response spectra (generated ground motion)

The generated accelerograms are obtained at the free-field surface. They have to be transferred to the foundation of the structure and used as an input motion in the structural response analysis. The modifying effect of local ground conditions on the seismic motion should be taken into consideration. For this purpose a probabilistic model of the soil strata is compiled. The geometric model consists of linear, homogeneouse, horizontally stratified soil layers overlying a homogeneouse half-space. The characteristics of the layers (S-wave velocity, density, Poisson's ratio, damping coefficient, etc.) in the free-field profile and those under the structure foundation plate are different only in upper 13 meters - the structure is founded at

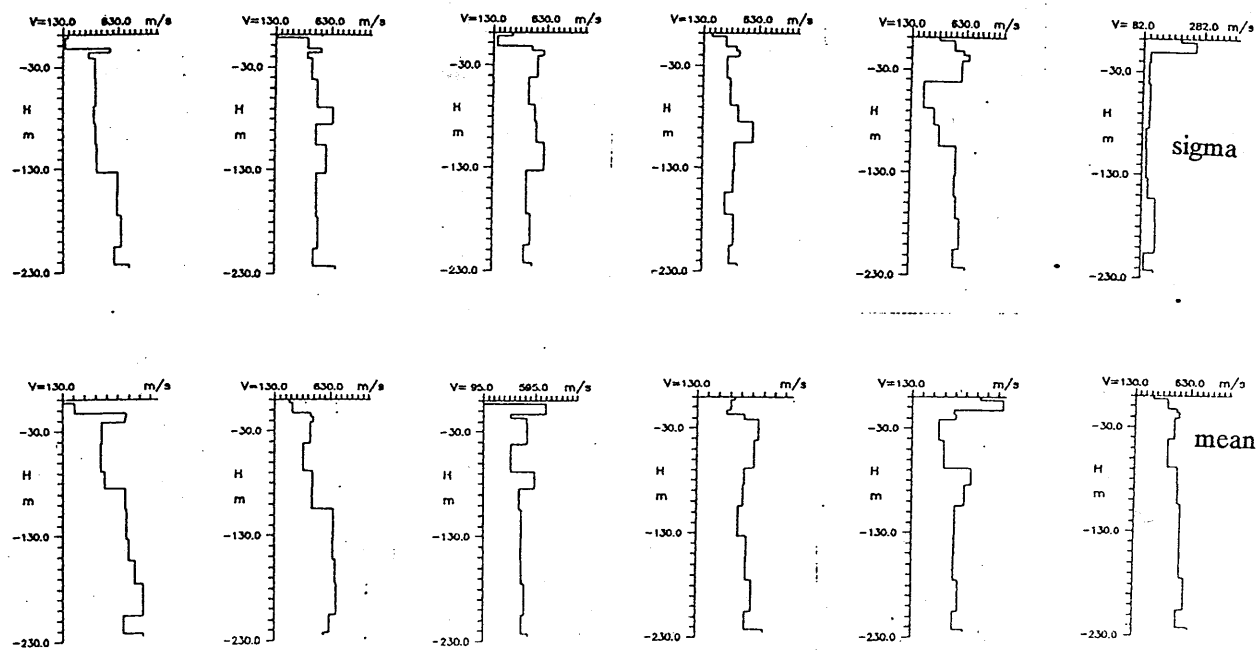


Figure 3. Generated velocity profiles of the soil model

depth of 7 m under the free surface over a specially treated layers up to -13 m. All characteristics are experimentally determined (mean values and limits of variation) at low strains . Strain compatible properties (the variation of G-modulus and damping depending on the strains developed during the excitation process) are determined also experimentally for six soil types. Those relations are used applying the equivalent linear method for solving the deconvolution problem (transfer the free-field motion to the foundation level). After a statistical processing of soil characteristics and applying LHCED procedure ten geological profiles are generated. The S-wave velocity variation in depth for the generated soil profiles are shown in Fig. 3. The last subfigures in first and second lines refer to standard deviation and mean values.

The deconvolution is performed over those 10 soil profiles with all generated free-field accelerograms for each level of hazard. In such way 90 accelerograms at foundation level are obtained (for each hazard level 10 three components accelerograms). Their acceleration response spectra are computed and also mean and mean + 1 standard deviation spectra are determined. In Fig. 4 the results for annual probability of exceedance 10^{-4} are shown - three components. For other two levels of hazard only horizontal components are given in Fig. 5. Comparing those spectra with the free-field spectra the amplification effect of upper soil layers can be seen. This effect is more considerable in low period range (about 0.1 s).

SOIL-STRUCTURE INTERACTION MODEL

The soil-structure model consists of 3D finite element model of the superstructure and ground model is represented by elastic springs and dashpots. All bearing elements are modelled by plate, shell and beam elements. Special attention is paid to the modelling of connections or separation joints between the different part of the structure (it is very complicated). The main part of the equipment is also included in the model.

The spring-dashpot representation of the soil is assessed applying three methods: the semi-empirical method of the weightless springs, the analytical method based on the elastic half space theory and the method of viscoelastic stratified half space. According to the first two methods the springs and dashpots are frequency independent. In the last method the representation of the soil is done by the impedance matrix which is frequency dependent and can be easily applied for calculation in frequency domain. For the sake of modal analysis performed for determining the structure response the characteristics obtained by first methods are used. The results from the third method are used only for comparison.

The semi-empirical method of the weightless springs [6] (Barkan, 1962, Savinov, 1972, Prakash, 1986) is based on the assessment of the vertical soil-foundation stiffness Kz. The stiffness in horizontal directions Kx and Ky as well as the rocking stiffnesses Kxx, Kyy and Kzz (torsion) are determined as a part of the vertical stiffness. The constant Kz depends on the coefficient of elastic uniform compression of the soil Cz and the contact area between the soil and the foundation plate. The relations between all constants are determined for mashine foundations and are given in the literature. This method is not assessing the damping constant. An empirical formula taken from Russian code is used but the experience proved that the values are very low.

The second method based on the half space theory gives relations of the constants outlined for a rigid circular disk laying on the surface of a visco-elastic half space. For square foundations equivalent radiuses are determined different for translational, rocking, and torsional motion. The results for the spring constants obtained by the two methods are very similar. The difference is significant for the damping constants. In the response analysis the values obtained by the second method is applied.

In the above method the embedment is not taken into consideration. A possible modification of the horizontal stiffness of the foundations could be done using the prescription of the American ATC3 provisions.

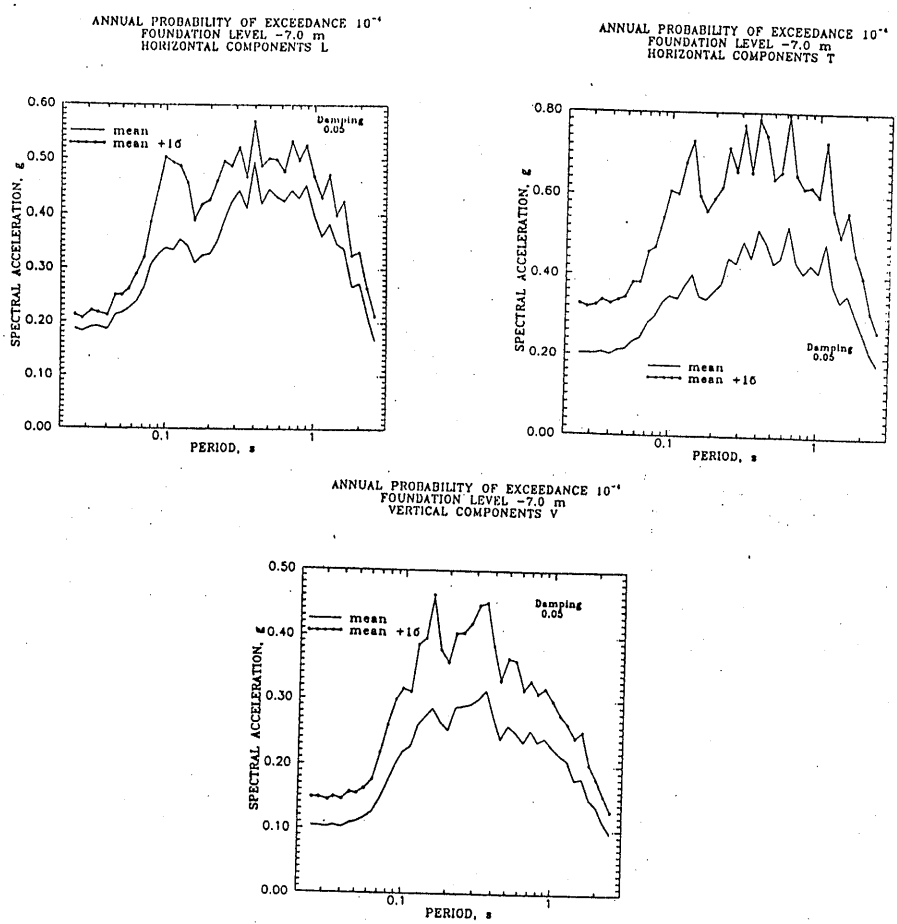


Figure 4. Acceleration response spectra of input ground motion (3 components)

Applying the third method the compliance and impedance functions are determined for different directions of motion. The results are close to that obtained by the second method.

SOIL-STRUCTURE INTERACTION ANALYSIS

Using the estimated foundation stiffness the "fixed base" model of the superstructure is improved adding springs and dashpots to the base mat foundation of the structure. A modal analysis is performed. The first two natural modes of vibration of the structure are shown in Fig. 6. Those modes could be called soil-structure interaction modes. Because of the significant structural stiffness the rocking effect is clearly expressed.

For the dynamic analysis of the main building 255 natural modes are used up to the frequency of 25 Hz. Time domain modal integration is performed repeatedly for all 10 three components accelerograms and the three levels of hazard.

For the response analysis the damping parameters are varied - damping in the structure and in the soil. Variation of 50% is assumed both for the structure and the soil. The values are different for the three levels of hazard - in the structure 4%, 5% and 7% of the critical damping are used respectively and in the soil for vertical vibration 60%, 70% and 80%. For horizontal vibration the damping in the soil is assumed to be 60% of the vertical one, 50% for rocking and 30% for torsion. The damping in the model is computed according to the composite damping rule. The natural periods obtained by this procedure are considered as mean values and additional variation of 30% is undertaken. All computations are conducted according to LHCED procedure, i.e. the response is computed for all ten 3-component accelerograms under the above described variation and for 3 levels of hazard. Then a statistical analysis is made for each of hazard levels and mean values

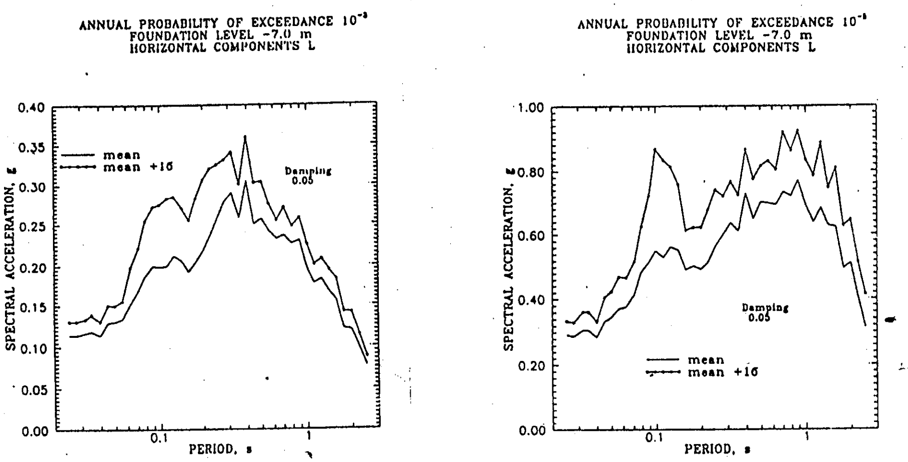
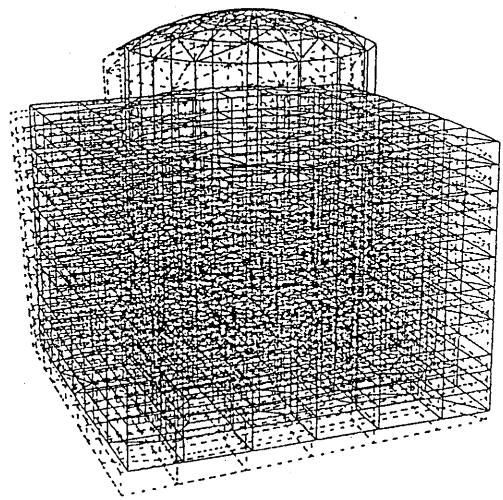
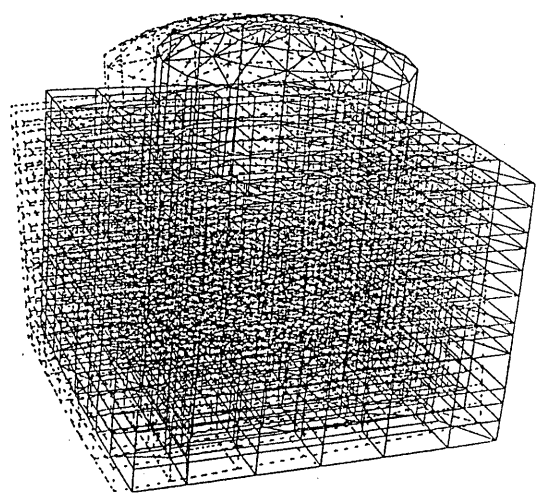


Figure 5. Horizontal components of acceleration response spectra of input motion



First mode of vibration, soil-structure model, T1=0.514s.



Second mode of vibration, soil-structure model, T2=0.512s.

Figure 6. Natural modes of vibration, soil-structure model

and standard deviations of responses are determined. In such way the mean and mean + 1 standard deviation acceleration response spectra (three components) are computed for various locations. The cumulative lognormal distribution fit for the maximum floor accelerations at different places are performed.

CONCLUSION

The probabilistic soil-structure interaction analysis is a part of the seismic response analysis for PRA. and a part of a general PSA procedure for seismic events. There are no existing seismic PRA performed for such kind of reactors and data from earthquake experience nor data from full scale dynamic experiments are not available. This is the reason a numerical simulation procedure for probabilistic response analysis to be adopted.

The main results achieved in the probabilistic soil-structure analysis are: detailed investigation of the seismic danger for the site; detailed investigation of the local soil conditions; assessment of seismic input motion expressed in 3-component acceleration time histories for different level of annual probability of exceedance; creation of a comprehensive 3D finite element model of the reactor structure including the soil effect. Final conclusions regarding qualitative assessment of the risk will be done after the finishing of the fragility analysis.

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