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# Probabilistic SSI analysis of unit 5/6, Kozloduy NPP

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ABSTRACT: This work is part of an on-going PSA project on the WWER-1000 units of NPP Kozloduy. In the paper are presented results from the probabilistic response analysis of the reactor building. The probabilistic seismic excitation definition and the probabilistic soil-structure interaction analysis are described. A comprehensive multiple time history analysis procedure is applied. The Latin Hypercube Sampling is used for preparation of the data base for the numerical computation. The soil stiffness and the radiation damping appear to be the key parameters of the reactor building response.

## 1 INTRODUCTION

The seismic probabilistic safety analysis is carried out in the frame of the overall project "NPP Kozloduy Unit 5&6, PSA - Level 1". It follows a general procedure for a probabilistic safety assessment for external events and the following steps are performed: seismic hazard at the site, plant systems' and structure's responses, fragilities of the safety related components, plant systems and accident sequences and consequences at nuclear unit.

The seismic response of the reactor building is assessed probabilistically and some results of the analysis are discussed hereafter. The applied analytical assessment methods lead to the development of an original procedure combining the requirements of the PSA and peculiarities of WWER-1000 structural design, to the probabilistic definition of seismic excitation taking into account the uncertainties and to the generation of probabilistic in-structure response spectra. As a result of the analyses a mean and variation of response for the essential structural elements are determined.

The problems of soil-structure-equipment modelling, of radiation and material damping modelling as well as of definition and investigation of the uncertainties are considered. Most of the analyses are based on the Latin Hypercube Experimental Design (LHCED) procedure.

## 2 SEISMIC HAZARD ASSESSMENT

The seismicity of Kozloduy region is well defined and studied. The earthquake distribution in the 320 km area around the NPP site is taken into consideration. The earthquake sources could be divided into three groups: local sources (zone with radius of 30 km and expected maximum magnitude Mmax,exp = 4.50), shallow depth sources (Mmax,exp = 8) and intermediate depth sources from Vrancea region generating long-period earthquakes (Mmax,exp = 7.8).

The probabilistic assessment of the seismic hazard is developed on the base of some assumptions: existence of a model of the potential seismogenic zones; the geometry of those zones, the frequency of earthquake realization as well as the maximum magnitude value of generated earthquakes and attenuation law are known. The random and model uncertainties are taken into account. Several alternatives are considered - two attenuation laws for the Vrancea seismic zone and two laws for the shallow depth sources; linear and quadratic regression curves for frequency of occurrence; two variants of maximum expected magnitude and variants of the space distribution of the seismic sources - fault model or diffuse seismicity. The computation is performed for 36 cases of different alternatives for 9 different periods of the response spectrum and damping 5% of the critical. As a result the mean value, standard deviation, median and geometric deviation (log-normal distribution) of the maximum acceleration, respectively of the spectral accelerations are determined. Hazard curves for the maximum acceleration at the site of Kozloduy are derived. The equal hazard acceleration response spectra for annual probability of exceedance of 0.001, 0.0001 and 0.00001 are computed. Figure 1 illustrates the spectra for the second hazard level.

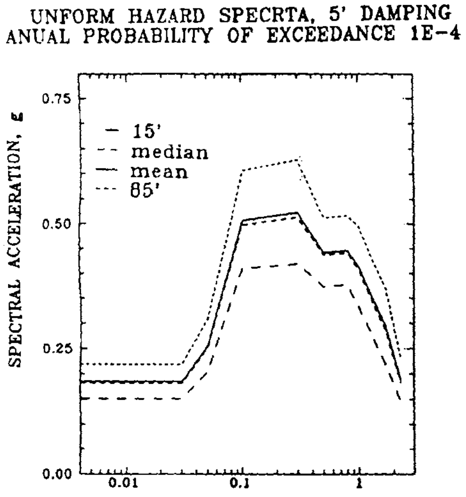

Fig. 1. Acceleration response spectra at free field

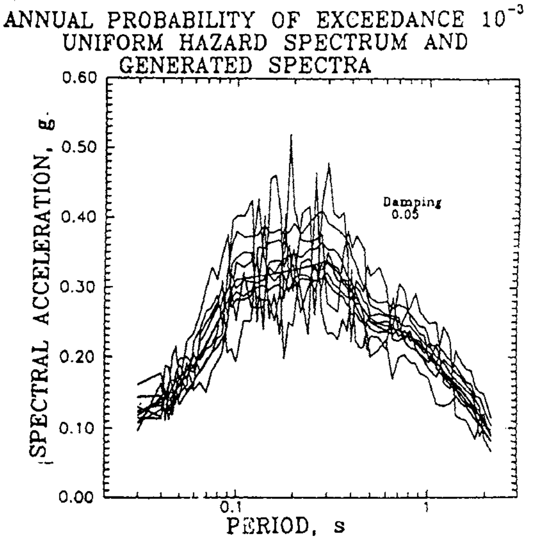

Fig. 2. Generated response spectra

## 3 COMPUTATIONAL ACCELEROGRAMS

For response analysis the seismic hazard is presented by a set of modified artificial accelerograms - 10 accelerograms (three components) for each level of annual probability of exceedance. In order to determine the necessary statistics for generation of the accelerograms 90 pre-selected records of real earthquakes divided in three groups corresponding to the seismic zones are analysed. The respective acceleration response spectra are statistically investigated. That information is needed for the generation of artificial accelerograms. Their three components are statistically independent and their relative intensity corresponds to that of the real accelerograms.

The time envelope of the generated accelerograms is assumed to be trapezoidal. The total duration varies between 15 and 60 s. The duration of the intensive part and that of the beginning are also varied. Using this information acceleration response spectra are generated. They match the respective equal hazard spectrum (Figure 2). Then accelerograms are generated and for each hazard level the mean value and the standard deviation of their spectra (10 spectra for each component) are computed. They match very well the mean value and the variation of the uniform hazard spectra.

All those accelerograms refer to the free field of the site. They are transferred to the foundation level by deconvolution procedure. A probabilistic model of the local geological stratum is compiled consisting of ten geological profiles generated by LHCED. For each level of hazard the mean and mean plus one standard deviation response spectra of the respective accelerograms at foundation level are computed. In such way the input seismic motion for each hazard level is presented by ten three component accelerograms taking into consideration the local soil conditions.

## 4 SOIL-STRUCTURE-EQUIPMENT MODEL

The structure of the reactor building of Unit 5 is a complicated spatial structural system consisting of four main parts - foundation block, containment shell, auxiliary building and inner reinforced concrete structure. Those four parts are joined by a thick reinforced concrete plate at level 13.2 m.

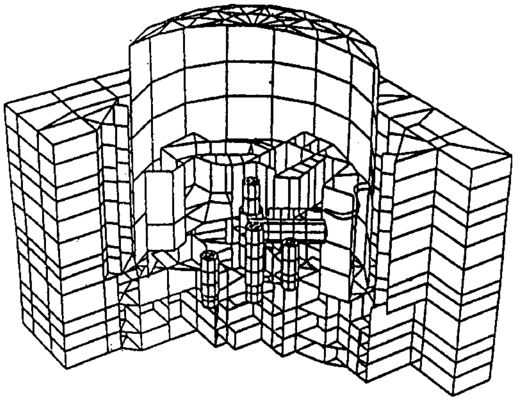

Fig. 3. Soil-structure-equipment model (cross section)

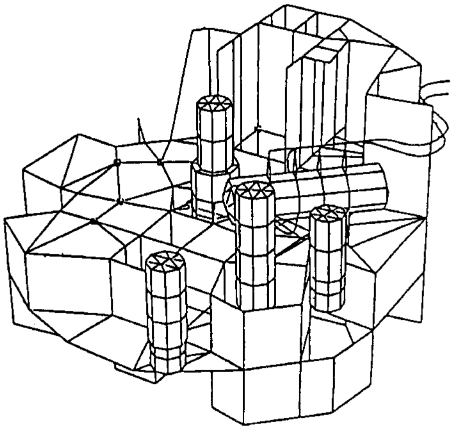

Fig. 4. Equipment model

The foundation block consists of a thick reinforced concrete plate with thickness of 2.40 m at elevation 7.00 m and two other plates with thickness of 0.60 m at elevations 0.00 m and 6.60 m. The plates are connected by numerous thick concrete walls. The containment shell is constructed by prestressed concrete. The auxiliary building is composed by reinforced concrete walls and plates situated around the shell and elevated above level 13.20 m. The inner concrete structure consists of many concrete walls and plates. It is situated in the containment shell. The main equipment is anchored in those elements.

A 3D finite element model for dynamic investigation of the soil-structure-equipment system is developed. A proper modelling of all bearing elements is aimed, taking into account all possible structural interactions. Because of the nature of the structure special attention is paid to the modelling of the separation joint between the auxiliary building and the containment shell.

The foundation structure is extremely rigid. It is modelled by 1430 plate elements. The containment is modelled also by shell elements. The ring support of the semi-spherical dom is modelled by beam elements. The internal concrete structure, placed in the containment above the elevation 13.2m is a complicated spatial system of walls and slabs. The modelling of that structure is very important for the equipment analyses. The auxiliary building is mounted also at elevation 13.2m. It surrounds the containment shell.

The soil is represented by springs and dashpots. The spring constants corresponding to the soil stiffness characteristics (six components) and the respective damping characteristics are estimated using different methods - the semi-empirical method of the weightless spring, the method of the elastic half space and the method of the impedance matrix for a stratified half space. The values of the foundation stiffness characteristics computed by the different methods are practically identical.

The modelling and computation of the soil-structure-equipment system is performed in three stages. First stage concerns the elaboration of the 3-D model of the structure only (fixed in the base). In the second stage 3-D model of the soil-structure system is developed adding springs and dashpots to the base mat. Finally the complex spatial model of the soil-structure-equipment system is created. A cross section of the model is shown in Figure 3. Only the main equipment (reactor, steam-generator, pressuriser, emergency cooling water tank, pumps, turbines, pipes, etc) participates in the model. Because of the double symmetry only 1/4 of the primary circuit is included as flexible elements. The influence of the other equipment is taken into consideration by their masses lumped at different points of the main structure. The model of the main equipment is shown in Figure 4.

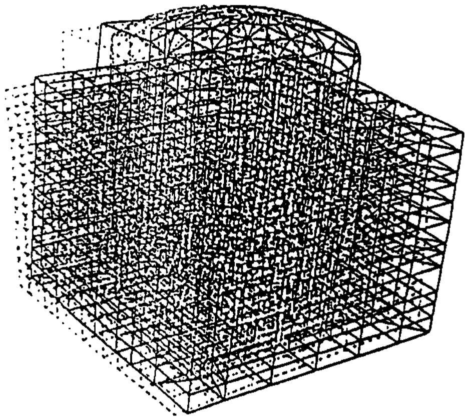

Fig. 5. Second mode of vibration
$T_2=0.528$ s

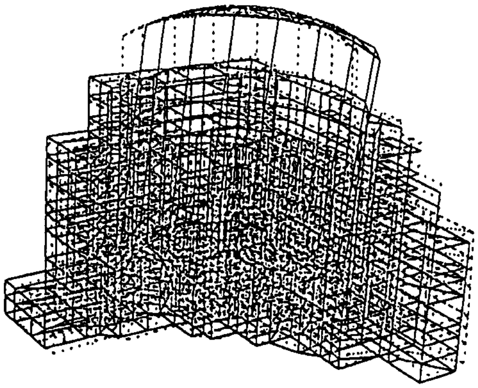

Fig. 6. Fifth mode of vibration
$T_5=0.234$ s

The damping in the model is computed according to the composite damping rule. In the structure are used 4%, 5% and 7% of the critical damping respectively for the three hazard levels with 50% variation and in the soil the damping in vertical direction is assumed to be 60%, 70% and 80% (variation 50%), in horizontal direction - 60% of the vertical, for rocking - 50% and for torsion - 30% of the vertical damping.

The modal analysis is performed using 255 natural modes up to the frequency of 25 Hz. Variation of 30% of the natural frequencies is applied. The second and the fifth modes of vibration of the soil-structure-equipment system are shown in Figures 5 and 6.

## 5 PROBABILISTIC RESPONSE ANALYSES

All computations are performed according to LHCED procedure. The response is computed for all ten three-components accelerograms involving the above mentioned variations and for three levels of hazard. Then a statistical analysis is made for each hazard level and mean values and standard deviations of responses are determined. Three components of the acceleration response spectra at different places (important for the structure and the equipment) are computed. In Figure 7 are shown the spectra for the first hazard level at the mid point of the dome. For various locations the cumulative log-normal distribution of the maximum acceleration is determined. In Figure 8 is given the cumulative distribution of the acceleration of the three components for hazard level 0.001 at the same nodal point. The maximum displacements of different points of the model are computed also.

The efforts in some of the plane elements of the model (the main bearing structural elements) as well as the efforts in the beam elements (the main piping elements) are statistically treated. The mean and mean plus one standard deviation values of axial and shear internal forces and of bending and torque moments are determined.

ACCELERATION RESPONSE SPECTRA
ANNUAL PROBABILITY OF EXCEEDANCE $10^{-3}$
DAMPING: 0.02; 0.05; 0.10
COMPONENTS L, T, V AT NODAL POINT 2028

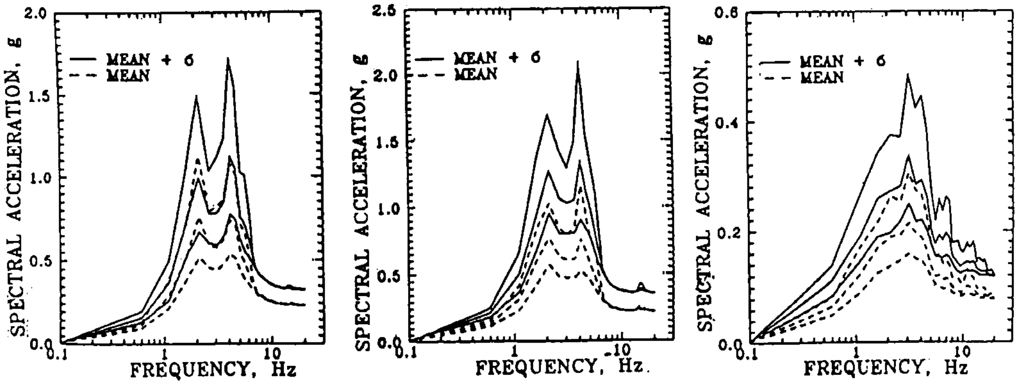

Fig. 7. Response spectra at the middle of the roof shell

CUMULATIVE DISTRIBUTION OF PEAK ACCELERATION
ANNUAL PROBABILITY OF EXCEEDANCE $10^{-4}$
(a,b,c) L, T, V COMPONENTS AT NODAL POINT 2028

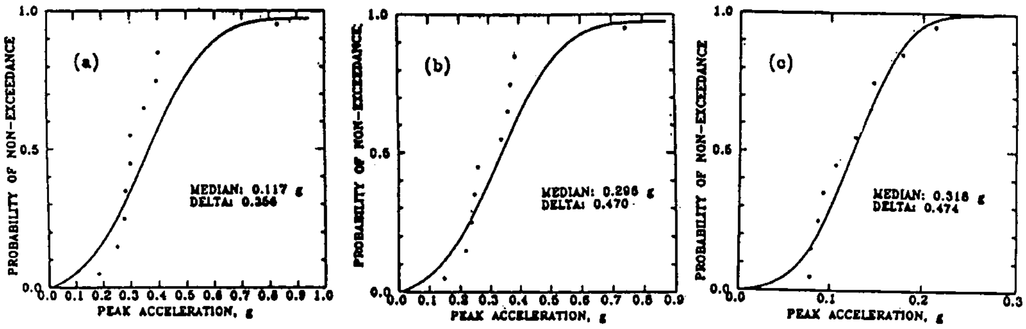

Fig. 8. Cumulative distribution

A probabilistic capacity analysis is performed also. The fragility curves are obtained taking into account the design strength characteristics of structural materials. The analytical procedure of Newmark for determining the acceleration at failure is applied. The distribution of this acceleration is obtained assuming log-normal distribution of the random variables. For the most important elements of the structure and equipment the following parameters are computed: the median value of the respective fragility parameter, the random uncertainty Vr, the model uncertainty Vu and the HCLPF (High Confidence Low Probability Failure). The fragility curves for shear failure of the containment shell at elevation 13.2 m are plotted in Figure 9.

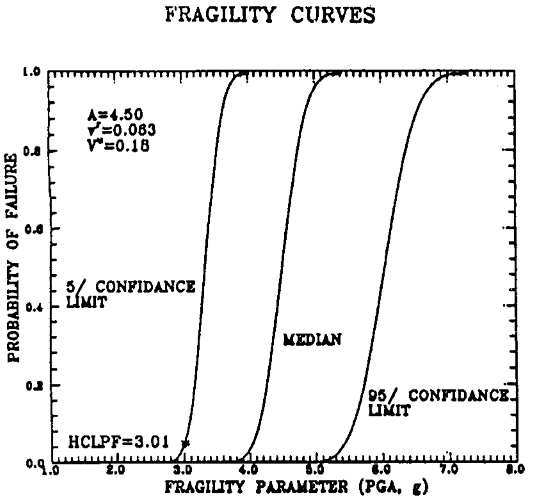

Fig. 9. Shear failure of containment at level 13.2

## 6 CONCLUSION

The probabilistic response analysis of WWER-1000, Kozloduy NPP leads to the following conclusions:

1. The reactor building is very rigid structure and the seismic response is primarily due to the soil-structure interaction.
2. The important seismic response is caused by rocking vibrations.
3. The equivalent modal damping for vertical vibration is larger than 70%. This is an important parameter which control the overall seismic behaviour of the structure.
4. There is an essential vertical acceleration response component in the auxiliary building due to strong rocking effects.
5. The variation of the seismic response is primarily due to the variation of the seismic excitation.

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