# SEISMIC RISK ANALYSIS OF CONCRETE GRAVITY DAM CHAIRA, BULGARIA

## BULGARİSTAN'DA CHAIRA BETON AĞIRLIK BARAJININ SİSMİK RİSK ANALİZİ

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## ABSTRACT

It is very important to know the probability of seismically induced failure of critical structures like dams. For this probabilistic risk assessment (PRA) is performed for the Chaira concrete gravity dam. Hazard curves of the peak ground acceleration and equal hazard spectra are obtained, the treatment of uncertainties regarding the input data being paid special attention.

The probability of failure of the dam is evaluated as follows: statistical formulation of material properties and loads, evaluation of the statistics of the response, definition of failure criteria, evaluation of probability of failure. Important conclusions for the probability of failure of the dam are drawn.

## INTRODUCTION

In the last two decades the PRA is performed for critical structures such as nuclear power plants, large dams, etc. It is very important to know what is the probability of failure of large dams because of the potential flood zone of these structures, furthermore if they are in seismic active regions. To define this probability, seismic PRA is performed.

The first step is to investigate the seismic hazard of the structure site. Seismic hazard is usually defined by the probability distribution function of the peak value of chosen ground motion parameter (peak ground acceleration and/or peak response spectral amplitudes for a range of frequencies) in a defined time interval. The overall study methodology consists of reviewing the existing geological, tectonic and seismological information to formulate a model of seismic activity of the region and applying this model to assess earthquake ground motion in terms of probability. An important issue here is the treatment of uncertainties regarding the input data.

The second step is to evaluate the probability of earthquake induced failure of the structure under consideration. The probability of failure

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can be computed in function of the parameters that define the distribution functions Fr(x) and Fl(x). In general Fl(x) represents the distribution function of loads and Fr(x) is the distribution function of the resistance, which in the case of seismically induced failure is often called seismic fragility curve. The fragility curve represents the conditional probability of failure as a function of seismic level. The purpose of the fragility curve is to incorporate therein the uncertainty in evaluating the capacity. To determine the total probability of failure the fragility curve for each failure mode is weighted by the probability of exceedance of the seismic load.

The probabilistic seismic assessment analysis of Chaira concrete gravity dam has been performed. An original procedure is developed combining the requirements of the probabilistic safety analysis of critical structures in seismic regions and the peculiarities of the dam [1,2,3,4,5,7,8] with main steps as follows: statistical formulation of material properties and loads; evaluation of the statistics of the response; definition of failure criteria; evaluation of probability of failure. Some of the results of that analysis are shortly discussed in the paper.

## SEISMIC HAZARD ANALYSIS OF THE DAM SITE

The model of seismic activity of the regional surrounding of the site is defined on the bases of complex investigation of the existing geological, tectonic and seismological information. It incorporates the following main characteristics: the site is situated at a minimal distance of 10km from the tectonically active structures; the strongest macroseismic effects at the dam site could be caused by an earthquake of maximum magnitude $7.6 <= M_{max} <= 8.0$ at a minimal distance of 25-30 km from the site; earthquakes of magnitude $M = 5.5$ are considered as a diffuse seismicity. The ground motion attenuation relationships used for the models are based on the analysis of strong motion data records from earthquakes in Balkan region countries and Italy.

In treating the uncertainties associated with the seismic input for seismic hazard analysis a selection of the most influential parameters has been performed regarding the overall uncertainty. As a result 72 hazard curves are obtained. Fig.1 shows the resulting mean, median, 15th percentile and 85th percentile hazard curves obtained assuming log-normal distribution of the peak ground acceleration at a given annual probability of exceedance.

By analogy uniform hazard spectra for four hazard levels A, B, C and D (annual probability of exceedance 0.01, 0.001, 0.0001, 0.00001 respectively) at 5% damping are determined. In Fig.2 is presented the uniform hazard spectra (mean, median, 15th percentile and 85th percentile) for annual probability of exceedance 1e-4.

## RISK ELEMENTS

As a result of the seismic hazard analysis of the dam site with definition of the uncertainties, the probabilistic characteristics of the seismic loads are estimated. In order to assess the probability of failure of the dam structure those loads have to be transferred to response quantities: stresses (strains), displacements, accelerations,

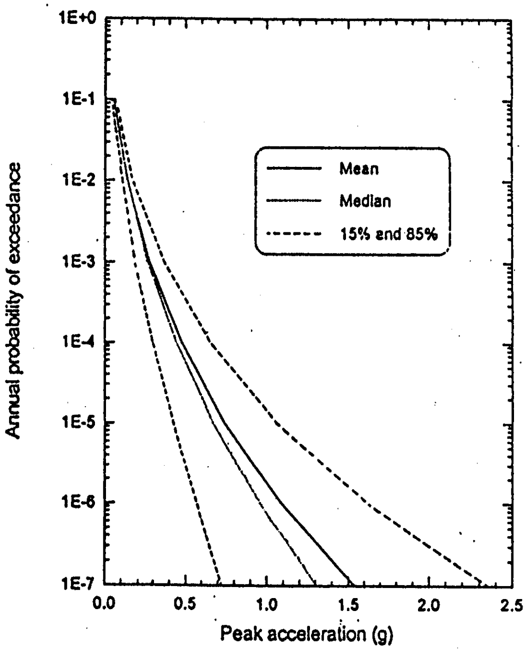

Figure 1. Seismic hazard curves

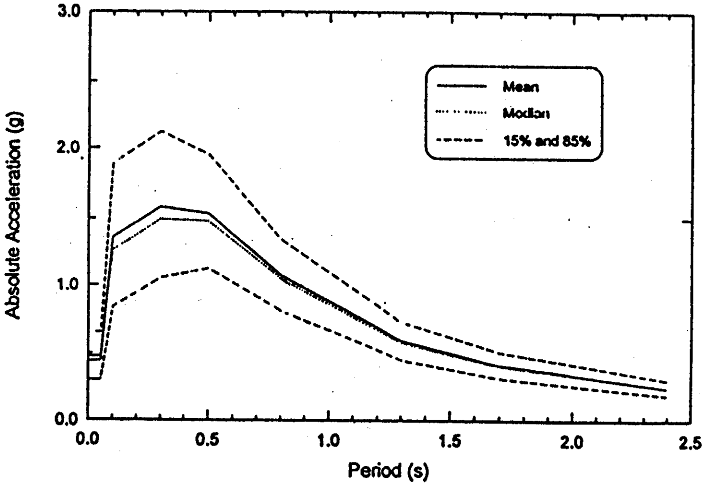

Figure 2. Equal hazard spectra, 5% damping, annual probability of exceedance 1e-4

etc. In order to obtain their statistical characteristics multiple deterministic analyses are performed each one with a set of input variables that conform their probability distributions. The distribution parameters of the output (response) variables are obtained by advanced Monte Carlo simulation techniques. Here the sample preparation is done according to the Latin Hypercube Experimental Design procedure [3], which is different from the traditional way of doing simulation analysis.

As mentioned above, for every single deterministic analysis there is a set of input variables needed (material characteristics and loads) which conform their probability distributions.

## STATISTICAL FORMULATION OF MATERIAL PROPERTIES AND LOADS OF CHAIRA DAM

All values in the statistical formulation of material properties for the different zones of the dam are assumed normally distributed. The zones of the dam are accepted as follows: surface concrete, core concrete, concrete in base level, contact concrete-bed-rock, bed-rock and alluvium. The mean static compressive strength of concrete in the different zones of the dam and coefficient of variation as well as the values for the static tension strength used in the analysis, are assumed on the basis of laboratory tests. The dynamic compressive strength of each zone is assumed 15% larger than the respective static one. The dynamic tensile strength and shear strength of the concrete are calculated. The static and dynamic modules are estimated on the basis of in-situ experiments. Also strengths at the contact "concrete-rock" are assumed in accordance to lab and field investigations. The thermal loads are represented in terms of ten sets of nodal temperature differences equally spaced throughout a year, obtained on the basis of statistical meteorological observations in the region of the dam. The values of the hydrostatic and the hydrodynamic loads are function of the water level in the lake which depends strongly on the selected scheme for usage of the power generation facilities. For the aim of the analysis an uniform distribution has been assumed for the water levels between 1225 m and 1260 m, i.e. minimum and maximum working water level of Chaira dam, respectively. The uplift forces and the water pressure on the grouting curtain are supposed perfectly correlated with the water level. Accordingly are defined the added masses and the function of the shape of the hydrodynamic pressure at the dam upstream side accounting for the hydrodynamic effects.

The seismic load is the most important for the seismic risk analysis. It is represented by means of the equal hazard acceleration response spectra and the corresponding accelerograms. Knowing the mean values of those spectra and their standard deviations (Fig.2) and assuming lognormal distribution of the spectral values ten spectra for each level are generated to match the prescribed statistics. In Fig.3 are shown the mean equal hazard response spectrum and the Latin Hypercube acceleration response spectra generation for level C. The acceleration response spectra for vertical motion are generated accordingly using a mean ratio between horizontal and vertical acceleration of 0.5 and coefficient of variation 20%. For each generated couple of acceleration response spectra (horizontal and vertical) acceleration time histories for each hazard level A, B, C and D are generated to match the response spectrum.

## STATISTICS OF RESPONSE

For the dynamic and static investigations the soil medium and the concrete dam body are modelled by 2D finite elements. Plane strain condition is assumed. The soil-structure model has the following dimensions: total length of 220m, total height of 170m. Four cross sections are investigated in details. Their disposition is presented in Figure 4 (part of the model). In addition a 3D finite element model is investigated in order to make comparison and estimation of response factors to correct the 2D estimates.

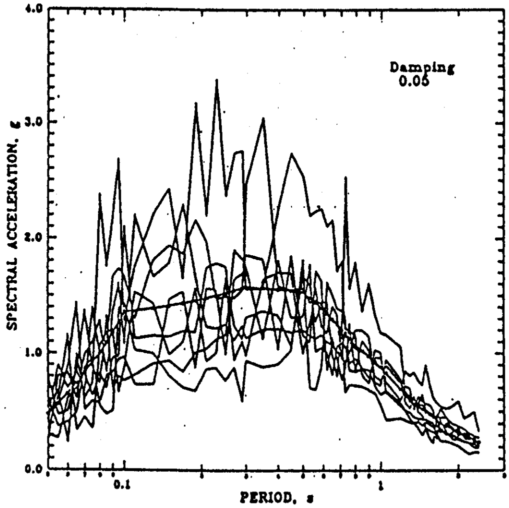

Figure 3. LHC Acceleration response spectra generation and mean equal hazard response spectrum for level C

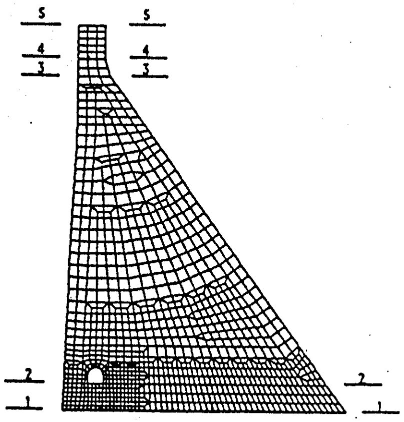

Figure 4. Part of the 2D FE soil-structure model

The analysis is performed by the computer program NISA [6] in 40 runs - the procedure is repeated 10 times (for the 10 different input sets) for each of the four hazard levels. The results in each hazard level are processed independently.

Detailed analyses are performed for the four sections shown in Fig.4. For those sections the maximum tensile Syy stresses in the nodes of the upstream side are supposed to be the main contributor to the risk. All other response values from the dynamic time history analysis are taken at the time of max Syy. Figure 5 represents a snapshot of the Syy stresses in the 3D model for level C from the dynamic analysis. After the maximum response is determined in each of the investigated sections a correction (on the base of comparison of 2D and 3D model results) is performed by a response factor assumed 0.3 for sections 1 and 2 and 1.0 for sections 3 and 4. This is performed for all four levels of seismic excitation and then a statistical analysis of response is done. It is assumed that the response quantities follow a normal distribution. Commulative distribution functions (CDF) of Syy for each node of the investigated sections and for each seismic level are computed and compared with the

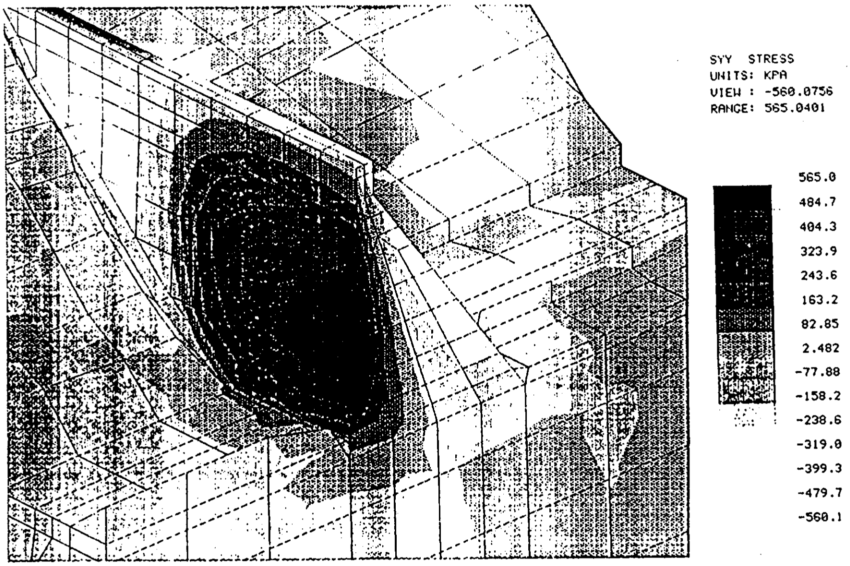

Figure 5. Snapshot of the Syy stresses in the 3D model for level C

theoretical normal CDF. The comparison shows that the generated response allows an assessment of the statistical character of the investigated quantities and that they fit very well the normal CDF. Figure 6 illustrates this for level D at node 831, corner at upstream side and base joint. The asterisks in the figure represent the generated values and the solid line shows the theoretical normal CDF. The mean values and the standard deviation of the generated response quantities are estimated. Figure 7 and 8 show the mean value and deviation of Syy and Sxy, respectively, for section 1.

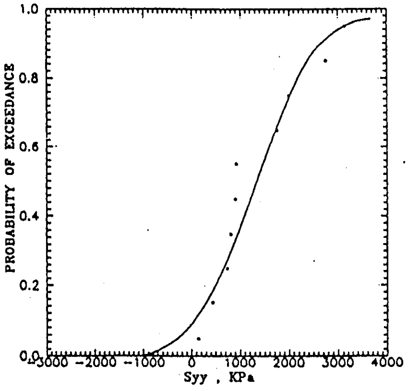

Figure 6. Commulative distribution function of Syy, N.P.831, level D

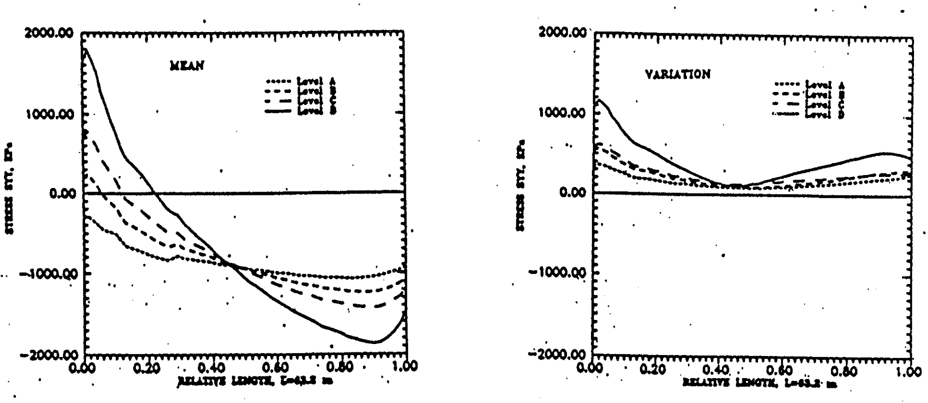

Figure 7. Mean Syy and variation of Syy, section 1

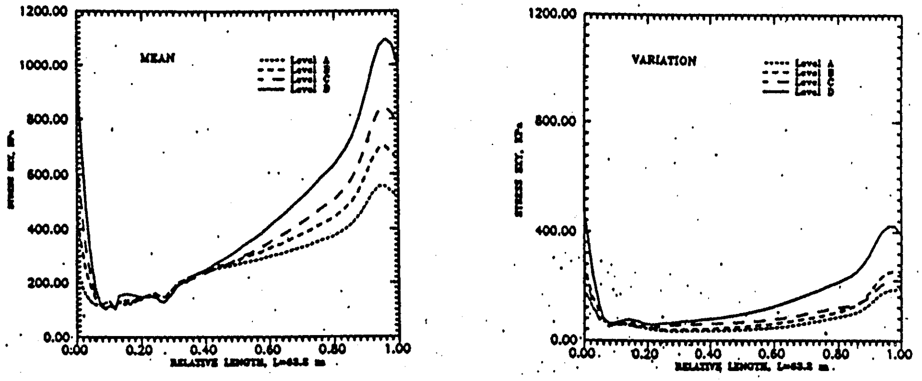

Figure 8. Mean Sxy and variation of Sxy, section 1

There are two major assumptions in the performed analysis which were justified. The first one is the number of the generations. Within each excitation level ten generations were used. To justify this test computation was performed for level C with 30 generations and the results from the 10 and 30 runs generations were compared. The same procedure was used for sample preparation. The comparison shows that the selected number of generations is sufficient for assessing the statistics of response. This is illustrated in Figure 9. The second assumption concerns the values of the response factor. To justify the values of 0.3 for sections 1 and 2 and of 1.0 for sections 3 and 4, a comparison was made between the results from 2D and 3D analyses. The spatial effects which can be accounted for in the 3D model lead to significant reduction of the stresses in the base zone of the dam. It is proved that the results from the 2D model using the accepted response factors are in good agreement with the corresponding results from the 3D model.

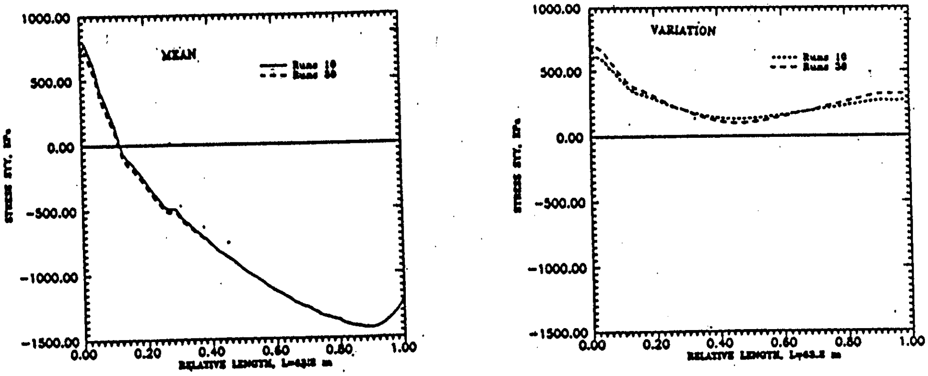

Figure 9. Mean Syy and variation of Syy, section 1, level C

## ASSESSMENT OF THE PROBABILITY OF FAILURE

Two failure scenarios are analysed for the Chaira concrete gravity dam: failure from overstressing of the concrete and failure from sliding of the entire dam body along the base joint.

For the first scenario the four previously described sections of the Chaira dam were investigated for failure due to tension.

Because the stress estimation is performed on the base of FE computation, stresses are known in each node of the FE mesh. In each node of the considered sections the probability of exceedance of the tensile strength is computed. In the base section instead of tensile strength, the cohesion between rock and concrete is considered. All finite elements in a section are supposed to resist as a parallel system. The probability of failure of the section in tension is calculated as the maximum of the probability of exceedance of the tensile strength in the nodes, within 0.2 of the section length, from the water side. That implies full correlation of the rupture in adjacent nodes, i.e. if there is a rupture in one node, all other nodes will be damaged also. This assumption should be assessed as a very conservative one. The computed in this way probability of failure is assessed as an upper bound.

An alternative is to average the tensile stresses in a small zone, i.e. to suppose certain correlation between the performance in adjacent elements. The zone where an average tensile stress is computed varies for each section. This alternative is supposed to give a lower bound of the failure probability. In general that alternative is more realistic.

For the purpose of assessment of the second scenario an integration over $S_{xy}$ in the base joint is performed. The resistance is computed according to the assumed rock properties at the contact rock-concrete. It is essential to mention that the sliding force, respectively the sliding resistance is determined for the moment of time when the stress $S_{yy}$ /tension/ is maximum. Usually at that moment there is a big vertical response, i.e. the vertical seismic acceleration is acting in opposite direction to gravity. Two different alternatives are considered also in that case. The first one is when the cohesion in the base joint is negligible. This is supposed to be a very conservative case. The second one: the cohesion is 1.5 times the respective static value (mean value of the static cohesion is 1.8 MPa). The first assumption is believed to give an upper estimate of the probability of failure, the second one should represent the respective lower bound.

In order to define the total probability of failure first the conditional probability of failure is assessed for each seismic level. The probability of failure (exceedance of the tensile strength) at each node of the finite element mesh within the investigated section is computed under the assumptions that stresses and resistance (strength) are normally distributed. The conditional probability of exceedance of the tensile strength in the investigated sections 1 to 4 are evaluated. All computations are done for each seismic level independently. The calculated values could be treated as discrete values of the fragility curve for tensile failure of each section. The maximum values of the probability of exceedance of the tensile strength are supposed to be the upper bound of the probability of tension failure in the respective section. As described above an alternative (lower bound) is computed by averaging the tensile stress in small zones.

For the second scenario of failure the conditional probability of sliding along the base joint is also evaluated. The sliding is computed for each seismic level. It is assumed that sliding will occur when the total force $S_{XY}$' (integral of $S_{XY}$) in the base joint will exceed the sliding resistance $R_{XY}$.

In the evaluation of the sliding resistance it is assumed that the ultimate stress is $S_a=C_d+0.8S_{yy}$, along the length of that part of the base joint which is under compression. $C_d$ is the cohesion. Two values are used for the cohesion: 0.0 and 1.8MPa. The Safety Coefficient for sliding is computed as ratio of mean sliding resistance to mean sliding force, i.e. that is a central factor of safety (conditional).

As already described the total probability of failure is computed weighting the conditional probability of failure by the probability of exceedance of each seismic level, for the upper and lower bounds of the failure probability.

## CONCLUSION

On the base of a seismic hazard analysis of the Chaira dam site a comprehensive seismic risk evaluation of the dam structure is performed. As a result the following conclusions could be drawn:

- The seismic resistance of the Chaira dam should be assessed generally as high although it is constructed at a site with a very high seismicity.
- The values for the probability of major failure of the dam for a period of 50 years are assessed as acceptable.
- The highest probability of failure is observed in the crest zone and in the zone of the main drainage gallery.

Some recommendations were made for increasing the safety of the dam in the zones with the highest probability of failure.

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## REFERENCES

1.Cornell,C.(1968), Engineering Seismic Risk Analysis, BSSA, V.5
2.Franzini,J.,McCann,M.,Shah, H.(1984), Application of Probabilistic Risk Analysis to the Safety of Dams, 8 World Conference on EE, V.7, San Francisco
3.Iman,R.L., Conover,W.J.,(1981), Latin Hypercube Sampling Program, Short Course on Sensitivity Analysis Techniques, Texas Technical University
4.Kostov,M., Samardjieva,E., Shanov, S.,(1992), Seismic Analysis of Belmeken-Chaira,V.IV,Part I - Seismic Hazard and Seismic Design Parameters,Sofia,(in Bulgarian)
5.Laboratory and Field Investigations for Chaira Dam,(1975), Energoproect,(in Bulgarian)
6.NISA II, Computer Code, User Manuals,(1992), Engineering Mechanics Research Corporation, Michigan
7.Toro,G. and McGuire,R.,(1988), Probabilistic Seismic Hazard Analysis. EPRI Methodology, Proc. 2nd Symposium on Current Issues Related to NPP Structures, Equipment and Piping, Hilton at Disnneyworld,FL
8.U.S. Nuclear Regulatory Commission Review Plan, (1983), Probabilistic Risk Assessment Procedure Guide, Report NUREG/CR-2300