SEISMIC RISK ASSESSMENT OF LARGE DAMS

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SUMMARY:
The large dam systems are high value infrastructure projects with significant importance. The large dams built in earthquake prone countries have greater demands on their structural safety. Inadequate seismic behaviour of dams could lead to severe consequences for the local communities. Therefore, the seismic safety assessment of large dam systems is an important engineering task. "Tzankov Kamak" dam is the first double curvature arch dam in Bulgaria. The complex shape of the dam, the site seismic environment, and the peculiarities of the local geological conditions (weakened and weathered zones in the foundation rock) require detailed investigation of the seismic behaviour of the dam and the foundation. The present paper describes the probabilistic evaluation of the seismically induced damages or failures of the dam wall and foundation. The parameters of the site specific seismic motion are based on results of PSHA. Failure and damage scenarios are derived by deterministic analyses for maximum credible earthquake (MCE) and evaluation of ultimate bearing capacity of the dam wall. In the probabilistic evaluation of the dynamic behaviour of the wall a Latin Hypercube (LHC) procedure is applied. Nonlinear dynamic analyses are used, including precise modelling of the contraction joints and base joints. Probabilistic assessment of damages and failure scenarios is performed and the scenarios with the highest probability of occurrence are presented.

Keywords: PSA, double arch dam, non-linear dynamic analysis, LHC

1. INTRODUCTION

This paper presents a methodology for assessments of the probability of occurrence of seismically induced damages and failures of large dams; it is applied to the specific case of seismic safety analysis of the arch dam "Tzankov Kamak" in Bulgaria. The methodology is a further development of former studies (Kostov et al., 1994, 1998, 2006). Basis for the analyses is the probabilistic seismic hazard estimations according to Cornell's (1986) approach; as a result, the probabilistic estimates for the seismic loading in terms of PGA and spectral accelerations are obtained including the characteristics of the maximum credible earthquake (MCE).

Preceding the seismic risk analysis several deterministic response and capacity analyses are performed as initial steps. The dam safety under seismic excitation corresponding to MCE is evaluated (Apostolov et al., 2010) in order to localize the critical zones of the dam as a first step. Non-linear time history analysis is used. The second step includes evaluation of the ultimate capacity of the dam wall (Andonov et al., 2010). The results of those analyses give several limiting (ultimate) values of the dam capacity, e.g. the crest displacements associated with total failure of the dam. A nonlinear push over analysis is applied to preliminary estimate the ultimate capacity values of the wall (Andonov, 2010). On the basis of these results, a probabilistic analysis of the dam is performed using LHC generations.

2. DESCRIPTION OF THE DAM STRUCTURE

"Tzankov Kamak" dam is the largest double-arch concrete dam in Bulgaria, with maximum height of 130,5 m and total crest length of 459,4 m. The widths of the base and the crest are respectively 8,8 m and 26,4 m. The dam consists of 17 separately erected 20-meter-wide cantilever blocks and abutment blocks, tangential to the crest's axis. The contraction joints between the vertical blocks are locked with

10-cm-thick shear key locks. A spillway is situated in the middle part of the crest. Five galleries are situated in the dam's body: the injection gallery following the base line and four horizontal inspection galleries on different levels of the dam. In Fig.2.1 are shown the general layout and a vertical cross section of the central block.

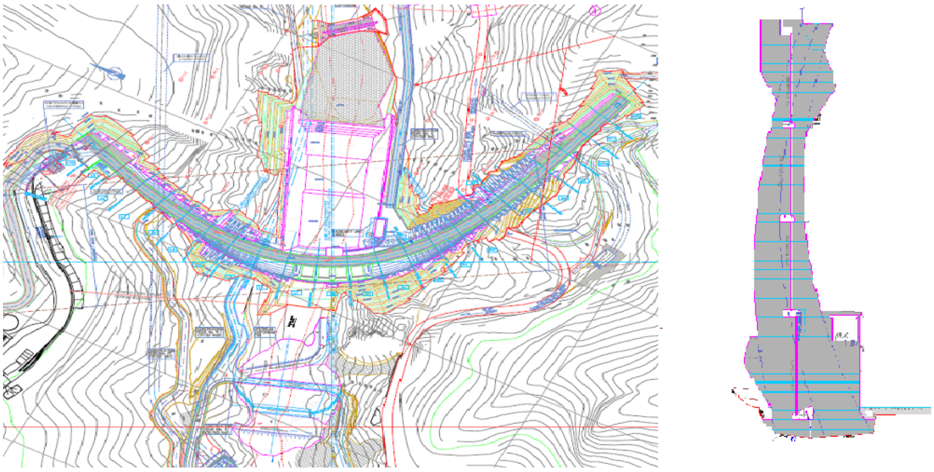

Figure 2.1 General view and vertical cross section of Tzankov Kamak dam

3. FINITE ELEMENT MODEL AND ANALYSES

The numerical model of the double-arch dam includes the dam's body and the surrounding rock foundation, the implemented excavations, the variations of the rock layers in foundation depth and the concrete plug-ins substituting the weakened and weathered rocks. The boundary conditions are applied of all lateral sides of the rock foundation model. Parts of the dam model with the geological profile underneath, as well as the base plane model are shown in Fig.3.2. The FE model consists of 8 layers of elements in direction of the dam's width. The average solid element size of the dam wall model is 3 m. The rock foundation is modeled with the following boundary extends (FERC, 1999): 1.) one dam's height in the direction perpendicular to the stream and in vertical direction; 2.) two dam's heights in the direction parallel to the stream;

The nonlinear dynamic analyses are carried out using the general-purpose computer code SOLVIA (SOLVIA, 2003). A nonlinear concrete constitutive model is used for the dam body, the contraction joints, and the base joint. The uniaxial stress-strain curve of the concrete constitutive model that is implemented in SOLVIA is shown in Fig. 3.1. The rock foundation is analyzed as massless elastic media.

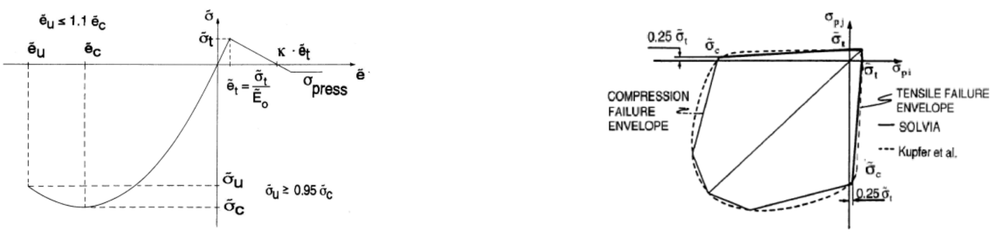

Figure 3.1 Uniaxial stress-strain curve and failure envelope for concrete, used in SOLVIA

The analysts' experience and the observed post-earthquake damages indicate as possible critical zones the connections between the separate dam blocks and those between the dam and its foundation, respectively, mostly due to their lower tensile strength compared to that of mass concrete (USACE, 1994; FERC, 1999; Fell, R. et al., 2005). Despite the decreased tensile strength of the contraction joints and their possible opening, the shear transfer capabilities of the wall must be kept intact in order to guarantee the overall stability and arch behavior of the structure. For this reason, shear keys are provided for the contraction joints. Also, a rough foundation is provided for the base surface in order to increase the shear bearing capabilities of the base joint.

The contraction joints are modeled by a thin layer of solid finite elements between each adjacent block. To ensure accumulation of the arch strains in these inter-block spaces, the material with decreased tensile strength is used for the contraction joints. The shear stress transfer capabilities of the joint elements, even after cracking, are ensured by the shear coefficient in the concrete material model. The dynamic tensile strength of the contraction joints is assumed 40% of the value for wall concrete. In the modeling of the base joint, a similar procedure is applied and its dynamic tensile strength is assumed to be 55% of the mass concrete value (FERC, 1999; USACE, 1994).

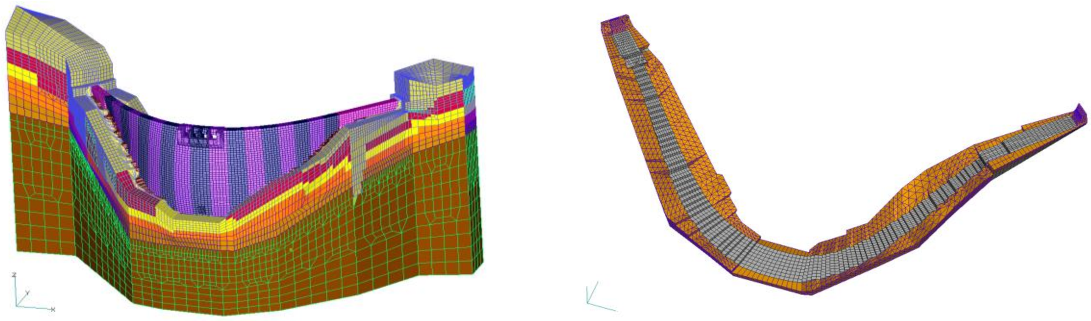

Figure 3.2 FE dam model with the rock foundation and model of the base joint

Two types of concrete with design grade C20/25 differing in their water permeability are used for dam construction. As the mechanical properties of both concretes are practically equal, only one concrete material is used in the analyses. The material properties used are derived from the results of in-situ and laboratory tests, whereby for the MCE case the 85% confidence level is accepted for all used values. The dynamic tensile strength of concrete is assumed to be 50% higher than the static one (ICOLD, 1983; USACE, 2007; FERC, 1999). Rayleigh damping is used for the concrete material and approximately 7% of damping ratio is assumed for the frequency range of interest. The values of the concrete material properties used in the analyses are shown in Table 3.1.

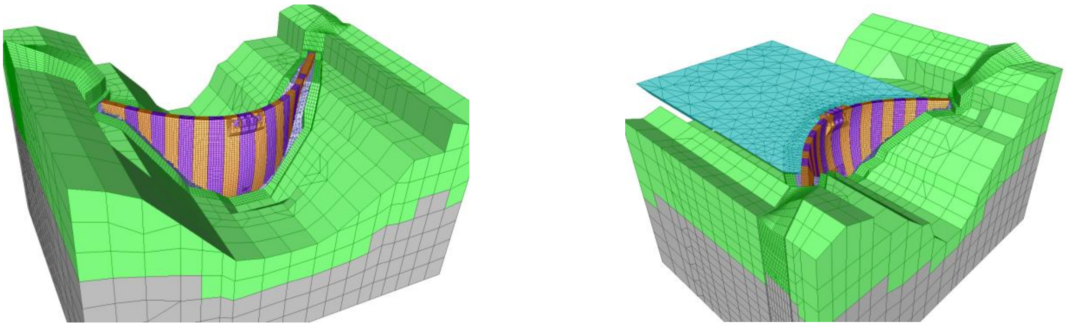

Figure 3.3 Complex FE model of the system "dam- rock foundation-water reservoir

Table 3.1 Concrete material properties

CONCRETE C20/25				
Material property		**Units**	**Static value**	**Dynamic value**
Elastic modulus	E	MPa	28 500	34 000
Compressive strength	σ_c	MPa	39.6	45.6
Tensile strength	σ_t	MPa	3.38	5.5
Poisson’s ratio	ν	-	0.20	0.20
Density	γ	kg/m3	2 380	2 380
Thermal coefficient	α	1/C	0.00001	0.00001
Damping ratio		%		7

The rock foundation is represented by seven types of rocks. Their elastic modules vary from 12000 MPa to 72000 MPa, and Poisson’s ratio varies between 0.24 and 0.27. The damping ratio of the rock foundation used in the dynamic analysis is assumed to be 10% (FERC, 1999).

4. SEQUENCE OF ANALYSES

The loads on the dam structure, used in the MCE analysis are: dead loads, temperature loads, load associated with the average annual water level in the reservoir (uplift, hydrostatic and hydrodynamic loading) and seismic impact. The PGA of the MCE is 0.42 g. The input seismic motion is presented as three dimensional artificial acceleration time histories, generated to be compatible with the respective site specific Uniform Hazard Response Spectrum (UHRS) evaluated in the Probabilistic Seismic Hazard Assessment (PSHA) of the dam site. The vertical PGA is 67% of the horizontal.

The sequence of performance of the analysis follows the subsequent application of self-weight and hydrostatic pressure, and thermal analysis, leading to the stress-strain condition of the dam, which is considered as the initial condition for seismic analysis.

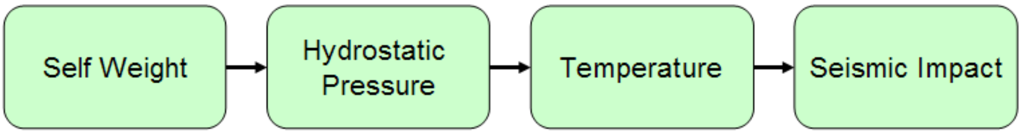

Figure 4.1 Sequence of analysis

The hydrostatic pressure is applied as element pressure, normal to the outer face of all upstream dam elements under the assumed water level. The uplift pressure between the upstream side and the grouting curtain is assumed to be equal to the hydrostatic pressure and applied at the dam's bottom surface. After the grouting curtain the uplift pressure is neglected.

The hydrodynamic water pressure is modeled as added masses calculated using a modified Westergaard method, which takes into account the type of structure vibration. The calculated added masses are applied to all nodes on the upstream side of the dam under the water level. The total sum of applied masses reaches about 70% of the wall mass. An alternative approach to consider the hydrodynamic pressure, namely FE modeling of the water reservoir, is used to verify the calculated added masses, too (Fig.3.3). For this reason a complex dam-reservoir model is created using fluid elements that extends horizontally up to three times the dam's height (FERC. 1999). Close results were achieved by both approaches.

A transient temperature analysis is performed based on the annual temperature curves for the air and the various water layers in depth.

The seismic input motion is described by statistically-independent artificial accelerograms compatible with the median uniform hazard acceleration response spectrum. One of the generated time histories and its compatibility with the target spectrum are shown on Figure 4.2.

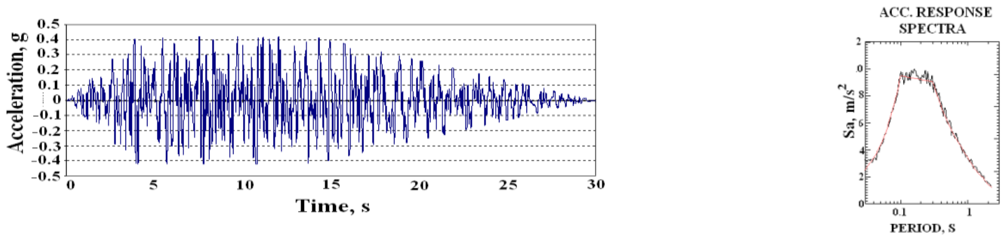

Figure 4.2. Seismic level – MCE. Generated accelerogram. Compatibility with the target spectrum

5. FAILURE SCENARIOS

The main results and the critical zones established after the MCE analysis and the evaluation of the ultimate seismic capacity of the dam are presented below.

5.1. Contraction joints

The status of all contraction joints in the moment of their maximum opening during the MCE excitation is presented in Figure 5.1. If the joint opening exceeds the shear key thickness this could lead to eventual loss of the arch effect and simple cantilever behaviour of the blocks, possibly resulting in a total dam collapse. The maximum values of the contraction joint openings are less than the shear key thickness – 10cm, therefore the arch action of all blocks of the dams is guaranteed in case of MCE.

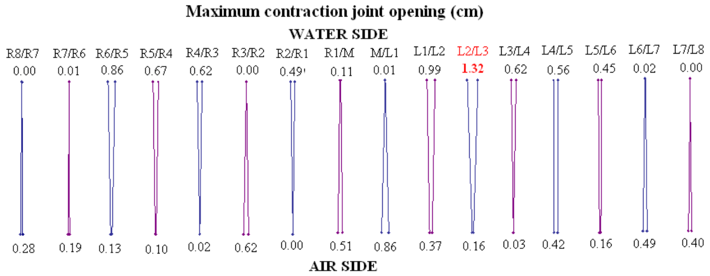

Figure 5.1. Maximum opening for each contraction joint of the dam

5.2. Base joint

Continuous cracks, parallel to the base line appear at the base joint of the dam mainly on the upstream face. Maximum crack opening is nearly 4 mm located again in the middle part of the right side of the dam. The cracks in the base joint penetrate the base plane up to 30% of the base width (Fig.5.2-left), which is the range of the grouting curtain. The overall stability of the dam is guaranteed, but the base joint cracking could eventually lead to water infiltration through the grouting curtain.

The sliding stability of the dam is verified by a comparison between the averaged shear capacity and the averaged shear stresses in the base plane (Fig.5.2-right). The min. base shear safety factor is 3.12.

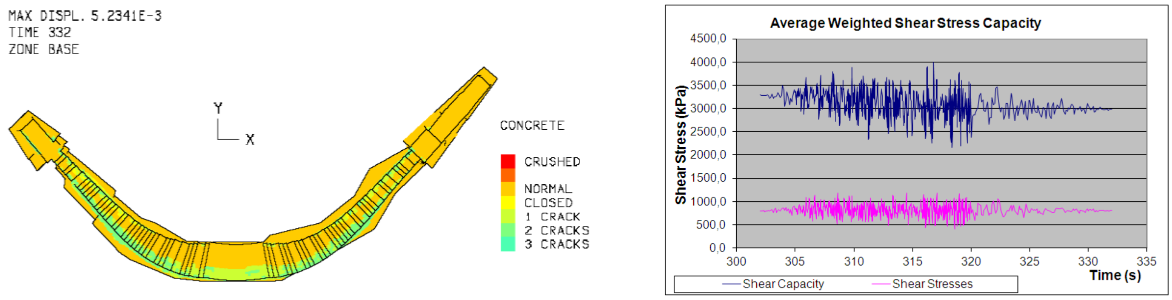

Figure 5.2 Base joint damages and shear stress capacity in the base joint

5.3. Downstream face

A critical zone that needs special attention is the downstream face of the dam, where in a large area in the middle part horizontal cracks appear (Fig.5.3). Eventual failure mechanism connected with this critical zone is a complete collapse of the upper part of the dam. Under MCE excitation the cracks in the middle part of downstream face are mostly superficial; however, deeper cracks are concentrated at the horizontal sections at the level of the galleries (Fig. 5.3-left), due to the decreased sections and the stress concentration. At the lateral blocks, the possible crack's penetration is deeper, eventually reaching the injection gallery (Fig. 5.3-left).

5.4. Compressive stresses and critical zones

The max, compressive stress of the dam reached during the MCE analysis does not exceed 25MPa and is sufficiently lower than the concrete compressive strength and can not affect the dam safety. The overall stability of the dam "Tzankov kamak" is guaranteed during MCE. The critical zones of the dam are: base joint, contraction joints, and the horizontally cracked area at the downstream side of the dam. The expected damages in these zones do not jeopardize the overall stability of the wall. Despite the eventual cracks in the base joint, the shear capacity of the dam wall remains sufficient. The connection between the heavy spillway and the wall is another critical zone of the dam structure where intensive horizontal cracking could occur and special attention has to be paid during a post earthquake inspection.

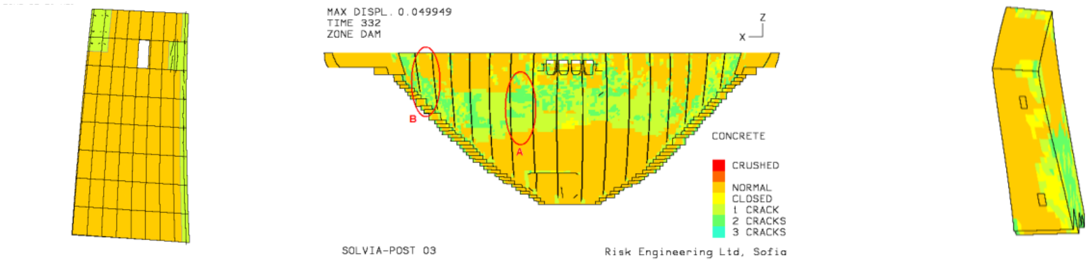

Figure 5.3 Cracked zones at the downstream dam side and section cuts at two blocks

5.5. Ultimate capacity and failure mode

Fig. 5.4 presents the failure modes of the structure in upstream and downstream directions obtained as a result of non-linear analyses with monotonically increasing unidirectional lateral loading proportional to the spatial distribution of displacements. The displacement distribution is calculated using spectral analysis based on the first 10 eigenmodes.

The failure modes in the two directions are with similar mechanisms, but are different in location and effect on the structural safety. The failure mode in both directions consists generally in exceeding of the bearing capacity along a horizontal failure plane. With increasing the lateral loading the intensity of cracking at the opposite of load direction wall face increases correspondingly. The depth of the cracking zone increases, too, and thus the compressed zone of the cross sections is continuously reduced. The stress redistribution leads to increasing of the compressive and the shear stresses in the compressed zone. The failure is formed when the stress redistribution provokes compressive or shear stresses above the ultimate limit strength. The obtained by push-over results show that the failure mode in both directions is formed due to the exceeding of the shear strength.

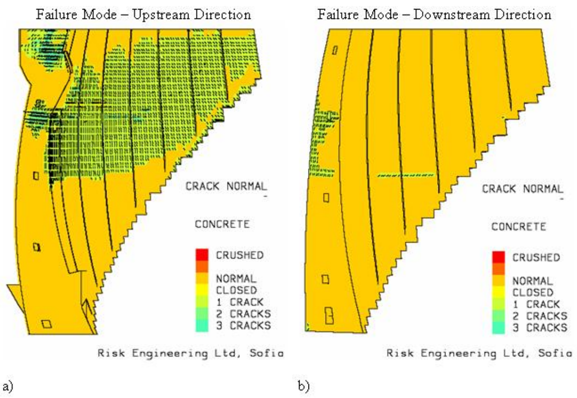

Figure 5.4. Failure modes in a) upstream and b) downstream directions obtained through nonlinear static analysis

The radial crest displacement envelope, showing the ultimate limit displacements of the crest before the total failure of the dam is shown in Fig. 5.5.

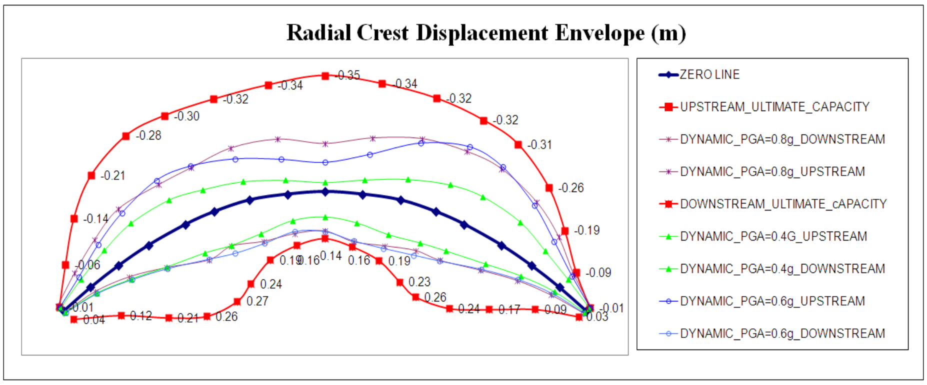

Figure 5.5 The radial crest displacement

5.6 Damage and failure scenarios

The performed analysis of MCE (annual probability of exceedance 10^{-4} and PGA 0.42g) and the static push-over analyses outlined the critical dam zones. Based on the results of the deterministic analyses the most critical failure and damage scenarios are defined. According to their influence on the structure safety the defined critical scenarios are divided in two groups -damage scenarios and failure scenarios.

- damage scenarios usually represent damages and failures in local zones of the dam or rock foundation and they can not affect seriously the global dam safety. The damages may cause problems of increased water seepage through the wall and wall foundation and subsequently a need of repair work. However, the overall structural integrity and safety should not be scientifically affected and severe accident should not be expected.
- failure scenarios that are characterized by sever damages or total structural collapse and may lead to total loss of arch action, heavy cracking or loss of global stability of the dam body and/or abutments.

The damage scenarios based on all mentioned critical zones can be summarized as: 1.) horizontal cracks at the upstream face of the dam; 2.) base joint opening; 3.) exceedance of concrete compressive strength in limited areas; 4.) damages in the rock foundation due to exceedance of rock base compressive strength; 5.) contraction joint opening, etc.

The estimated failure scenarios are: 1.) sliding of the dam structure at the contact between the dam and the rock foundation; 2.) collapse of the left abutment, due to exceedance of compressive strength; 3.) sliding of the abutment at the contact between the dam and the rock foundation; 4.) dam collapse due to total failure mechanisms in upstream and downstream direction.

6. SEISMIC RISK ASSESSMENT

The aim of the seismic risk assessment is to evaluate the probability of failure of the dam for the expected lifetime of the structure due to seismic initiating events, to determine the most critical scenarios of failure and most critical sections in the structure. The probability of failure of the dam can be obtained from the annual frequency of failure, βE, determined by (Borges, 1971):

$$\beta E = \int [d[\beta (x)]/dx]\, P(f^3x)\, dx \qquad (1)$$

where βE is the annual frequency of dam seismic failure, β (x) is the annual frequency of exceedance of load level x, $P(f^3x)$ is the conditional probability of dam failure at a seismic load level x. The function P is known as a fragility function. The problem requires assessment of the seismic hazard β(x) and the fragility $P(f^3x)$.

The probabilistic seismic excitation β(x) is described by hazard curves obtained from PSHA. The fragility curve for each damage scenario assessed to be critical for the investigated structure is evaluated. Finally, each fragility curve is convolved with the seismic hazard curve to estimate the annual frequency of realization of each scenario. The global risk for a seismically induced dam failure is presented by the scenario with the highest frequency of occurrence.

6.1. Seismic hazard analysis and statistical definition of the seismic input

The PSHA procedure proposed by Cornell is used to assess the parameters of the probabilistic seismic excitation. The random and epistemic are considered by a set of hypothesis forming the branches of the logic tree. The ground motion attenuation laws used for the models are based on the analysis of strong motion data records from the European database (Ambraseys et al., 2001).

From the discrete distributions of frequency of exceedance for various levels of maximum acceleration the mean, median, 15th-percentile and 85th-percentile hazard curves and UHRS for six hazard levels with annual probability of exceedance 0.01, 0.00211, 0.001, 0.0001, 0.00001 and 0.000001, respectively, are obtained.

For each hazard level the seismic loading is presented by a set of acceleration response spectra and the corresponding acceleration time histories. Those spectra are generated on the base of the statistics of the (UHRS). The vertical components are obtained by scaling of the horizontal ones with random numbers with mean value of 0.67 and standard deviation of 0.3.

6.2. Statistical formulation of the material properties and loads

6.2.1. Strength and elastic properties of the materials

The material property statistics is estimated by processing of available construction and foundation data. For each type of material the mean value and the variation coefficient of the material characteristics are determined.

6.2.2. Thermal loads

The thermal distributions in the dam bodies are obtained by transient heat transfer analysis performed for a period of one year. The statistical data used are based on the meteorological and hydrological observations in the region and the dam reservoir. As a result the stresses due to thermal loading are calculated for each node of the structure. A set of 10 temperature loadings with equal probability of realization is generated to be used in the probabilistic analysis.

6.2.3. Hydrostatic, hydrodynamic and filtration pressure

As “Tzankov kamak” dam is recently constructed a uniform distribution is assumed for the water levels around the mean normal operation level. The hydrostatic loads and the hydrodynamic pressure are assumed to be perfectly correlated with the water levels. The effects of the hydrodynamic pressure are considered by added masses lumped at the upstream wall face.

6.3. Assessment of the response statistics

The nonlinear deterministic analyses are carried out by the computer code SOLVIA. The computational procedure is based on an advanced Monte Carlo method (Latin Hypercube) for simulation (Imam et al., 1981). The main steps of the computation are as follows: preparation of input variable samples by LHC procedure, computation of the dam seismic response parameters, evaluation of the critical zones and the maximum values of the control parameters, and, finally, evaluation of the statistics of the results. The procedure is applied for each of the seismic levels. For statistical processing of response, a normal distribution of response parameters is assumed.

6.4. Conditional probability of failure and fragility curve generation

The probability of failure for an investigated scenario is computed under the assumptions that the load and the resistance are distributed in a log-normal fashion. The conditional probability of failure is computed using the distribution function of the resistance and the density function of the loading. For each failure scenario the conditional probabilities of failure calculated and respectively the fragility curve for the particular scenario is developed.

6.5. Seismic risk assessment

The risk for seismically induced damage or failure expressed as annual probability of occurrence of the most critical scenarios is calculated by integration of the seismic hazard curves together with the specific fragility curves. The LHC procedure is applied for the integration in order to take into account the uncertainties in the hazard assessment and in the conditional probability of failure. Samples of size 10 are used. Table 6.1 presents the values of the estimated seismic risk, expressed as annual probability of occurrence. Secondary risk for facilities and inhabited areas is not considered.

Table 6.1. Seismic risk associated with defined scenarios

№	DAMAGE SCENARIOS	CONFIDENCE LEVEL		
		15%	50%	85%
1	Damages on the downstream side of the wall	4.89E-5	8.96E-5	1.64E-4
2	Damages of the grouting curtain	4.89E-5	9.25E-5	1.76E-4
3	Damages due to the exhausting of the concrete comp. strength	1.11 E-5	2.48E-5	5.57E-5
4	Damages due to the contraction joints opening	9.33E-7	1.85E-6	3.7E-6
	FAILURE SCENARIO	CONFIDENCE LEVEL		
5	Sliding of the wall at the contact “concrete – rock”	2.54E-7	3.32E-7	4.32E-7

7	Sliding of the abutment at the contact “concrete – rock”	2.31E-6	4.64E-6	9.32E-6
8	Deep sliding of the abutment	2.54 E-7	3.32E-7	4.33E-7
9	Global failure of the construction toward the upstream side	4.43E-7	1.74 E-6	6.83E-6
10	Global failure of the construction toward the downstream side	9.33E-7	1.85E-6	3.7E-6

From the analysis of the calculated seismic risk associated with the defined failure or damage scenarios the following conclusions should be drawn:

- The calculated risk for occurrence of the defined failure scenarios (85% confidence) is in the range of 10^{-7} to 10^{-6}. The estimated seismic risk should be assessed as low (i.e. acceptable). In the same range are the failure risks for facilities with very high secondary risk.
- The most probable failure scenario is shallow sliding of the left abutment.. The probability of occurrence of this scenario is about 5E-6 with a 50% confidence level.
- The applied methodology of the seismic risk evaluation is based on the independent assessment of each scenario. The cumulative risk for seismically induced failure of the wall is in the order of 8.9E-6 and should be assessed as acceptable.
- The damage scenarios are with higher probability of occurrence. They are associated mainly with exhausting of the strength capacity of the dam materials and local damages. These damages are repairable and the initial strength and functional condition of the construction could be restored.
- The most probable damage scenarios are damages of the upper 1/3 of the downstream side and damages of the grouting curtain at the contact between the wall and the rock base. The probabilities of occurrence of these scenarios are equal – about 2E-4 with 85% confidence. If the bearing capacity of the dam is assumed constant in time, the risk for realization of these scenarios in a period of 475 years is 4.2% with a 50% confidence and about 9% with a 85% confidence, respectively.
- The analyzed damage scenarios are related mainly to exhausting of the strength capacity of the used materials. The seismic risk is calculated for the present strength condition and in the analyses the real strength parameters are used. This fact implies some conservatism as the concrete strength shall increase in time and the values of the seismic risk shall decrease in the future, respectively.

6.6. Sensitivity analysis

To access the influence of the uncertainties of the input data on the seismic risk and to estimate the variation of the values assessed, a sensitivity analysis is performed. The influence of the uncertainties of material strength characteristics, of seismic hazard parameters, etc., is studied.

The influence of the model uncertainties on the global seismic risk can be assessed by varying of the number of the LHC combination of system parameter used in the statistical processing of the results. Ten runs are assessed to be enough to obtain confident results.

The influence of the uncertainties of material strength parameters and seismic input is assessed for central values of the seismic risk (50% probability of exceedance). The sensitivity analysis for uncertainties of seismic hazard and strength parameters is performed for the following cases:

- 30% decrease of the strength parameters of the concrete and base rock;
- 30% increase of the mean values of the hazard acceleration curves;
- Simultaneous strength reduction and increase of seismic hazard - as an unfavorable combination.

The results of the sensitivity analysis are shown in Table 6.2.

The sensitivity analysis shows that the risk of occurrence of failure even for the most unfavorable combination remains in the range of E-05. Such values of seismic risk are typical for critical structures with high secondary risk and are acceptable for the dam evaluated.

The risk of seismically induced damages at the downstream side and in the grouting curtain is relatively high, i.e. about 1.3E-3. It means that for a 100-year period risk of damages is about 12%. This value is calculated applying the most conservative assumptions and its realization depends strongly on the concrete tensile strength that usually increases with the time. A 10% increase shall reduce the seismic risk by 70%. Based on these arguments we have the belief that the risk for the occurrence of these scenarios for 100-year period even at the most conservative assumptions shall be sufficiently lower than 5% and should be assessed as acceptable.

Table 6.2. Seismic risk for the strength parameters' simultaneous reduction and increase of the seismic input

№	SCENARIOS	SEISMIC RISK			
		mean	+30% excitation	-30% strength	combined
1	Damages on the downstream side of the wall	8.96E-5	2.45E-4	5.14E-4	1.35E-3
2	Damages of the grouting curtain	9.25E-5	2.70E-4	5.18E-4	1.29E-3
3	Damages due to the exhausting of the concrete compressive strength	2.48E-5	8.36E-5	1.48E-4	4.42E-4
4	Damages due to the contraction joints opening	1.85E-6	5.63E-5	1.34E-4	4.26E-4
5	Sliding of the wall at the contact "concrete – rock"	3.32E-7	1.90E-6	1.49E-6	4.10E-6
6	Sliding of the abutment at the contact "concrete – rock"	4.64E-6	9.32E-6	2.87E-5	9.26E-5
7	Deep sliding of the abutment	3.32E-7	3.05E-6	9.9E-6	4.27E-5
8	Global failure of the construction toward the upstream side	1.74 E-6	8.86E-6	3.45E-5	9.85E-5
9	Global failure of the construction toward the downstream side	1.85E-6	7.53E-6	1.32E-5	4.52E-5

CONCLUSIONS

The seismic safety of "Tzankov kamak" arch dam under MCE excitation is demonstrated. The dam wall keeps its integrity and restricts the uncontrolled flooding from the water reservoir. The probability of occurrence of the scenarios associated with heavy damages and total failure of the dam does not exceed 10^{-6} with an 85% confidence level. This range of the failure probabilities is typical for hazardous industries and is acceptable for facilities like the "Tzankov kamak" dam. Even using the conservative assumption of increase of seismic loads and a simultaneous reduction of the material strength, the probability of failure remains in the range lower that 10^{-4} and is acceptable.

The damage scenarios at the downstream dam face and grouting curtain due to exhausting of the tensile strength of material may occur with relatively high probability. The values of the seismic risk are about 4.2% for a 475-year period. These probabilities should be considered as overestimated for the strength characteristics of the concrete shall increase with time.

A probable damage scenario is the opening of the vertical contraction joints. This scenario is most probable in the upper part of the wall. The risk for dangerous opening of the vertical contraction joints able to influence the dam safety is small but the joint opening shall require extensive repair work.

The applied methods of analyses demonstrate the efficiency of the LHC for propagating data uncertainties in complicated multidimensional problems. Although it requires significant computational efforts, the results achieved are supporting strongly the engineering decisions made.

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