

SEISMIC ANALYSIS OF A ROCK-FILLED DAM WITH ASPHALTIC CONCRETE DIAPHRAGM

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ABSTRACT

The study of the dynamic response of rock-filled dams with impervious synthetic elements includes investigation of (i) displacements and inertial forces, (ii) potential reduction of the shear strength of the material subjected to cyclic loading, (iii) effects of the loss of shear strength on the stability of the structure and, (iv) permanent deformations which can cause serious damage. In this article are presented the results and the conclusions of the study of dynamic behaviour of the Knezhevo Dam in Probishtip, a rock-filled dam with an asphalt concrete core. This dam and reservoir is the main part of the multipurpose hydro system Zletovica in the north-east part of the Republic of Macedonia, which is under construction now. The height of the Knezhevo Dam over the rock is 80.5 m, the crest width is 10.0 m, and the upstream and downstream slopes are assumed 1.8. The dam location is situated between VIII and IX degree of MOS, and the dam site profile is composed of several families of schist. The aim of this research is to add to the understanding of the behaviour of rock-filled dams with asphalt concrete diaphragm under strong earthquakes and to contribute towards the establishment of a systematic procedure for dynamic analysis of this type of structures.

Keywords: seismic analysis, rock-filled dam, asphaltic concrete diaphragm

INTRODUCTION

Knezhevo Dam is the most important part of the hydro system (HS) Zletovica, which is a multi-purpose project that utilises the waters from the Zletovica River and its tributaries. Zletovica River is the right tributary of Bregalnica River and runs through the east driest region of the Republic of Macedonia, figure 1. This river represents a unique source of pure and healthy water for long-term solving of water economy problems of a wider region, particularly solving the water supply problem for the communes of Kratovo, Probishtip, Shtip, Karbinci, Lozovo and Sv.Nikole. The sub-catchment of Knezhevo Dam profile is 53.6 km², which is about 30.1 % of the total area of 178.2 km² - for the most downstream intake structure of the HS Zletovica. However, the average inflow for the Knezhevo Reservoir, according to the hydrological analysis is 0.88 m³/s, which is 44.4 % of the available water discharge of the HS Zletovica, where the average flow is assumed as 1.98 m³/s.

KNEZHEVO DAM: DATA RELEVANT FOR THE DYNAMIC ANALYSIS

Based on engineering-geological, geophysical and geotechnical investigations, carried out for the Knezhevo Dam site, chlorite-epidote, albite-chlorite-epidote and sericite-muscovite schists are defined. The boundaries of these families of schist are not clearly distinguished, and schists are degraded and disintegrated at the surface up to a depth of 4.0 m. The thickness of the alluvial sediments in the river bed is about 5.0 m, while on the sides the rock is covered with deluvial

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material, which is good for foundation. Seismic parameters for design of Knezhevo Dam have been defined based on performed seismological, seism-tectonic and engineering-seismological investigations of the terrain. According to the seismicity of the region, defined in these researches, the dam location is situated between VIII and IX degree of MCS scale. The earthquake accelerations of the Zletovica Project, which has been defined according to local conditions of seismicity, are as follow: (1) OBE = 0.26 g and (2) MCE = 0.36 g.

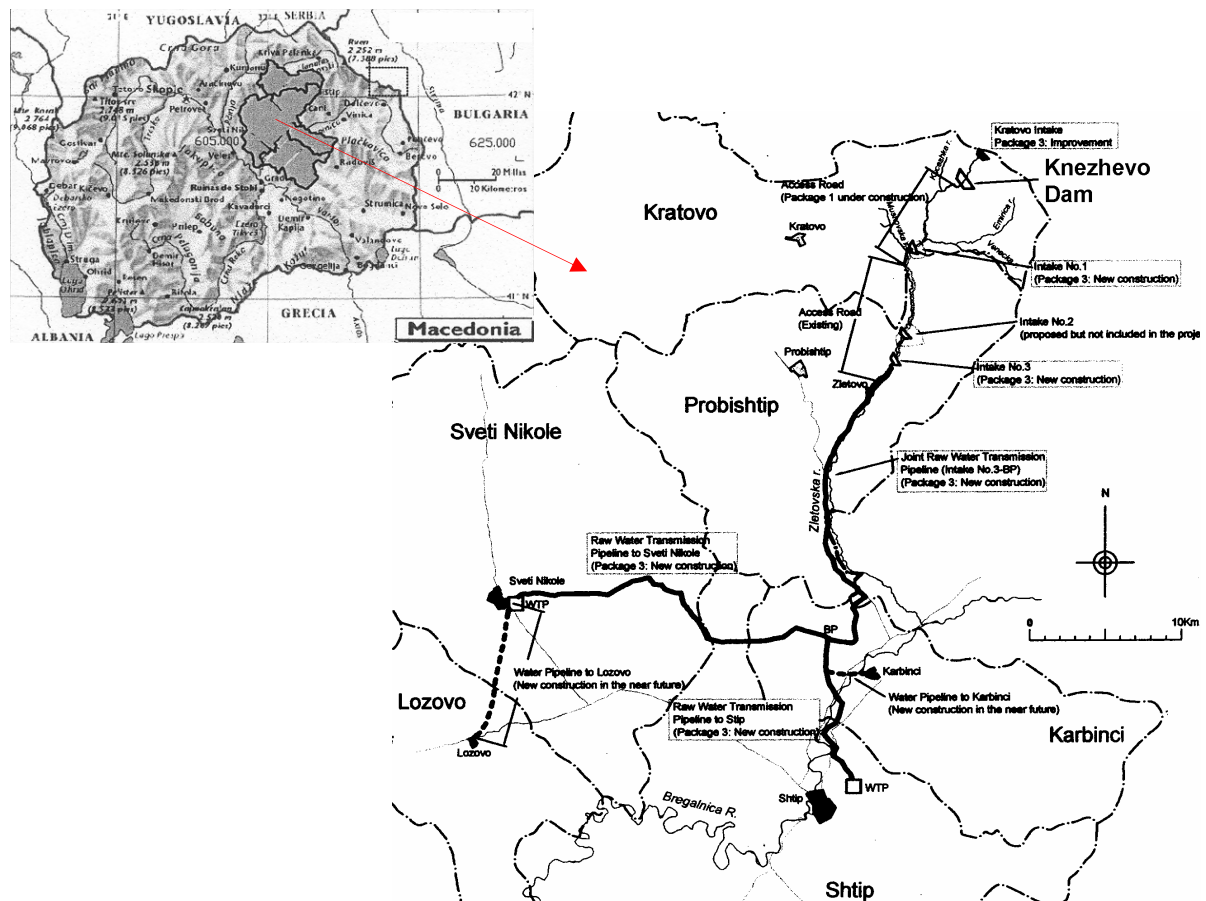


Figure 1. Republic of Macedonia and hydro system Zletovica

Knezhevo Dam is a rock-fill with internal asphalt core (bituminous concrete), type ACRD, figure 2. The upstream slope is protected by rip-rap, this layer being 1.5 m thick, and made of rock blocks. The dam water-tightness is insured by the vertical asphalt concrete core, which width is 60 cm. The aggregates used for the asphalt core will be provided by the andesite quarry. This core is separated from the shoulders with transitions, which are 1.5 m thick. The transitions will be made of crushed rock. Fine grained material may be added in the upstream transition in order to decrease the permeability, and therefore the seepage in case of opening of cracks in the core. Due to the excellent permeability characteristics of the foundation, there is no need for a grouting gallery. The contact of the internal membrane with the foundation is insured by a slab, (5.0 m width and 50 cm thick), which is founded and anchored on the bedrock. The consolidation grouting, as well as grout curtain, can be efficiently implemented after slab construction. The dam crest will have a total width of 10 m.

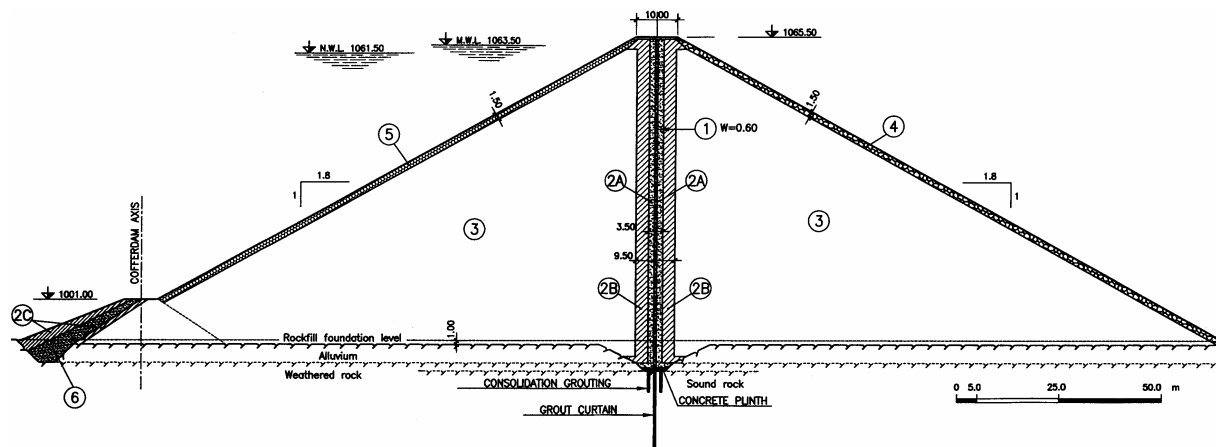


Figure 2. Typical cross section of dam Knezhevo. 1 - asphalt concrete core, 2A - fine transition $0 \div 60$ mm, 2B - coarse transition $0 \div 250$ mm, 3 - quarried rock $D_{max}=650$ mm, 4 - downstream protection, 5 - rip rap $500 \div 800$ mm

MATERIAL PARAMETERS

The geotechnical parameters of the materials, adopted for the analysis of the dam stability are presented in table 1. These parameters are: volume weights, strength parameters and linear elastic parameters of local materials - representative for static loading, modulus K representative for dynamic loading with Equivalent Linear (EL) models.

Table 1. Geo-mechanical parameters

symbol	unit	(1) filter dacite	(2) shoulder schist	(3) foundation alluvium	comment
φ	$^{\circ}$	43.0	39.0	35.0	angle of internal friction
e		0.30	0.25	0.20	void ratio
γ_{sat}	kN/m^3	23.5	23.0	22.3	saturated unit weight
K_0		0.32	0.37	0.43	$K_0 = 1 - \sin \varphi$, at-rest earth pressure coefficient
ν		0.24	0.27	0.30	$\nu = K_0 / (1 + K_0)$, Poisson coefficient
E	kN/m^2	30,000	26,000	27,000	static modulus of elasticity (Young)
K	kPa	13,450	14,746	16,170	$K = 5000 \cdot (2.17 - e)^2 / (1 + e)$, dynamic modulus for shear modulus G_{max}
n		0.23	0.20	0.17	$n = e / (1 + e)$, void
γ_{sp}	kN/m^3	27.6	26.3	24.8	$\gamma_{sp} = (\gamma_{sat} - \gamma_w n) / (1 - n)$, specific unit weight
γ_{dry}	kN/m^3	21.2	21.0	20.7	$\gamma_{dry} = \gamma_{sp} (1 - n)$, dry unit weight
ω_{sat}		0.11	0.09	0.08	$\omega_{sat} = e \gamma_w / \gamma_{sp}$, saturated wetness
ω		0.06	0.05	0.06	$\omega < \omega_{sat}$, natural wetness
γ	kN/m^3	22.5	22.1	21.9	$\gamma = \gamma_{dry} (1 + \omega)$, natural unit weight

The dynamic material properties used in the non-linear or equivalent linear analysis (ELA) are presented on figures 3 and 4. The central diaphragm of asphalt concrete is 0.6 m width. The geometric parameters of the diaphragm are: momentum of inertia $I = b * h^3 / 12 = 1.0 * 0.6^3 / 12 = 0.018 \text{ m}^4$ and cross section $F = b * h = 1.0 * 0.6 = 0.6 \text{ m}^2$. The adopted material characteristics of asphalt concrete are as follow: $E = 11.7 * 10^6 \text{ kPa}$, $\nu = 0.3$, $G = 4.5 * 10^6 \text{ kPa}$.

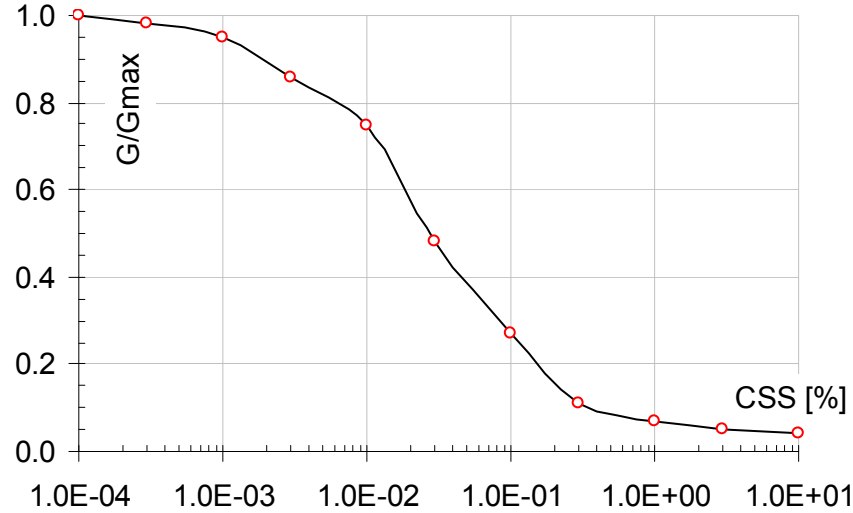


Figure 3. Reduction of the shear modulus as a function of the shear strains, $(G/ G_{\max}) = f_1(\gamma)$

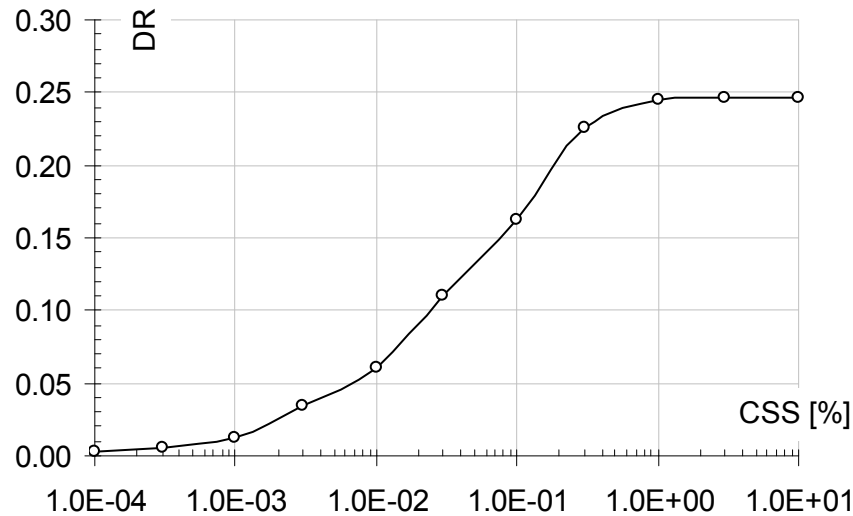


Figure 4. Damping ratio as a function of the shear strains, $DR = f_2(\gamma)$

MATHEMATICAL MODEL OF THE DAM AND INITIAL STATE OF STRESSES

The adopted cross section of the Knezhevo Dam for this dynamic analysis, which is based on some simplifications and discrete approximation with mesh of finite elements, is presented on figure 5. The static analysis of state of stresses for initial conditions at full reservoir was performed by using the programme Geo-Slope SIGMA (Geo-Slope SIGMA/W v5, 2001.), a geotechnical computer package based on finite elements method. The stress state for full reservoir is shown in figures 6 and 7.

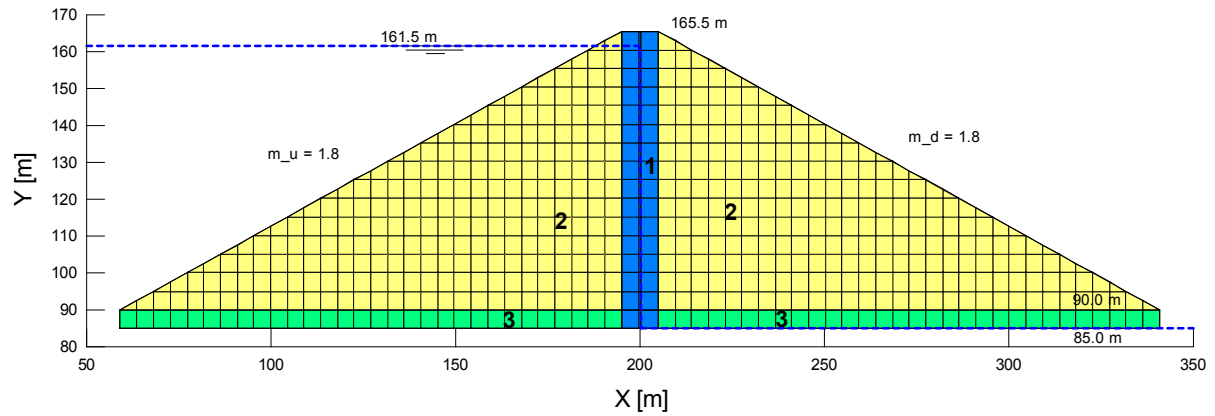


Figure 5. Mathematical model of Knezhevo Dam, for state of full reservoir and mesh of finite elements, Nodes = 621, Elements = 572

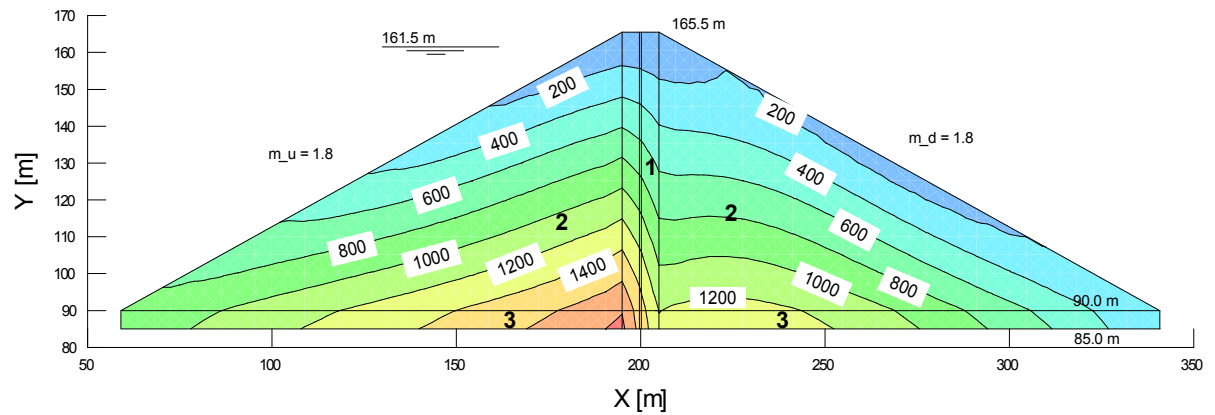


Figure 6. Maximum total stress σ_1 [kPa], initial state of stresses

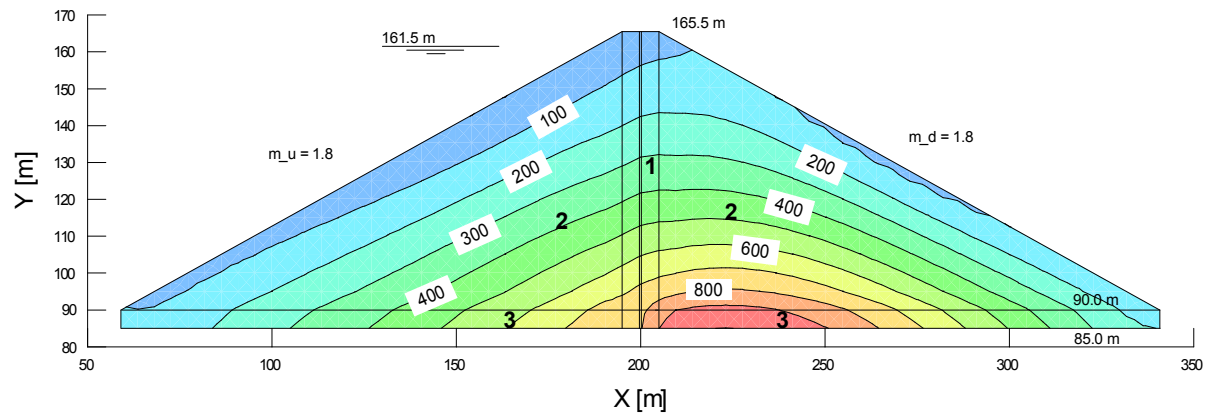


Figure 7. Mean effective stress σ'_m [kPa], initial state of stresses

DYNAMIC ANALYSIS OF THE KNEZHEVO DAM

According to Euro Code 8, the 2-D analysis in time domain should be conducted with strong motion excitation composed of two components - one horizontal and one vertical. The assessment of the seismic stability of the dam is performed with two types of earthquakes: OBE – Operating Basis Earthquake and MCE – Maximum Credible Earthquake, according the newest information about terminology of seismic aspects of dams from ICOLD (Wieland M., 2003.). The return periods for these two earthquakes are adopted with respect to the size and the importance of the structure. In the case of the 80.5 m high rock-filled dam Knezhevo, the return periods are adopted as $T_1 = 200$ years

and $T_2 = 10,000$ years, for OBE and MCE, respectively. The accelerograms for the earthquakes used in this dynamic analysis are real or recorded which amplitudes are scaled to match the expected PGA, and synthetic earthquakes. In this paper, in the following text is presented only the response of the dam subjected by MCE synthetic earthquake.

Dynamic characteristics of synthetic earthquakes

A brief comment on the seismic hazard is presented in the following text. For the seismic safety evaluation of the Knezhevo Dam, with 80.5 m high over rock, an earthquake with a return period of 10,000 years is considered as a maximum credible earthquake (MCE). According to the seismological map of The Republic of Macedonia, the Knezhevo Dam is located in a zone with expected MCE intensities of VIII degrees (MKS-64). According to the USGS scales, intensity of VIII corresponds to an earthquake with magnitude $M = 6.9$. The research of the relations between the magnitude and the peak ground acceleration (PGA) in the region, which includes the Republic of Macedonia, (Manić M., 1998.), shows that the PGA values may be assumed as 0.36 g for MCE. The elastic response spectrum of the input ground motion at the rock level is taken according to Macedonian normative, (Paskalov T., Zelenović V., 1986.,) figure 8. Category of foundation for S_I (rock and solid foundation with $V_s > 800$ m/s), is appropriate for characteristics of dam site of Knezhevo Dam. From Euro Code 8, the vertical component a_v is taken as 2/3 of the horizontal component a_h . Therefore, for peak ground acceleration (PGA) in horizontal direction of $PGA_x = 0.36$ g, correspond the value in vertical direction of $PGA_y = 0.24$ g. The diagram of intensity for generation of real earthquake for duration of 25 sec, according to Euro Code 8, the strong motion duration for $a_h = 0.36$ g of MCE, rounded on ± 5 sec, is presented on figure 9. The artificial spectrum compatible accelerograms (figure 10), which are required for the nonlinear dynamic analysis in time domain, is generated with the computer programme SIMQKE (SIMQKE 1997).

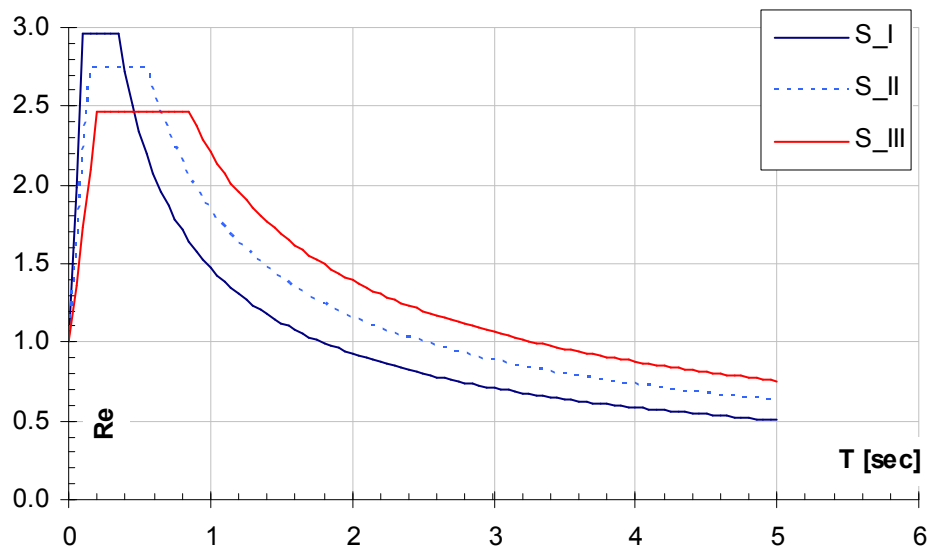


Figure 8. Design spectra according to Macedonian norms

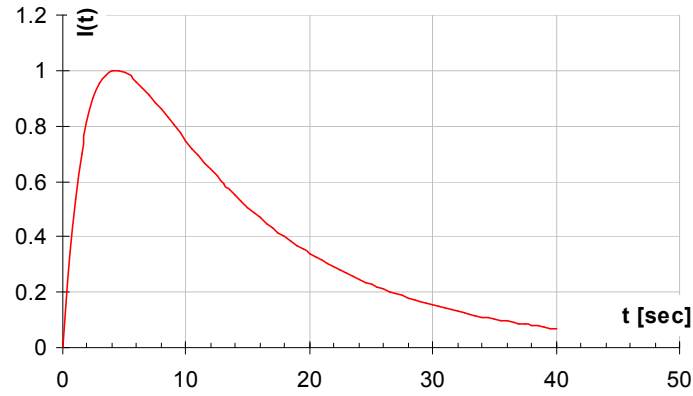


Figure 9. Exponential intensity for generation of real earthquake for duration of 25 sec

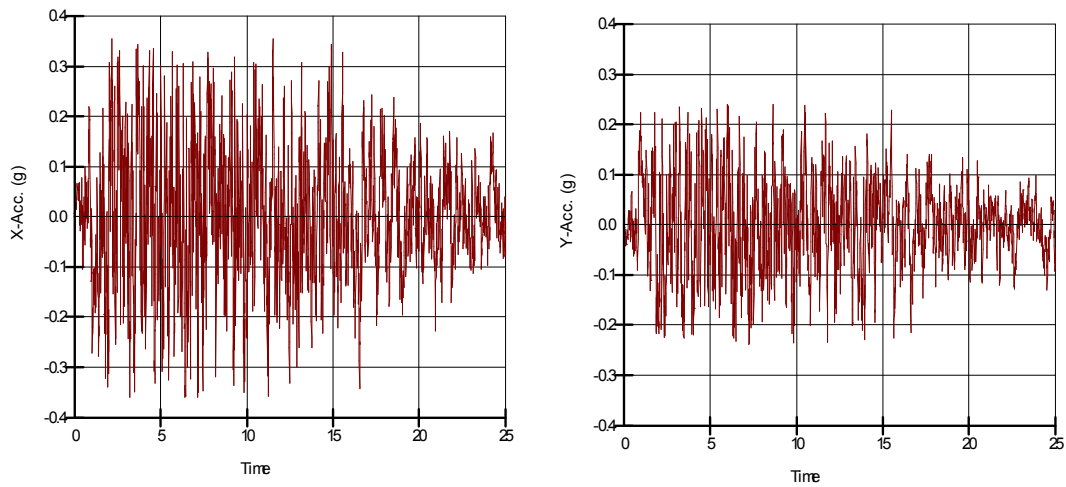


Figure 10. Input ground motions (synthetic earthquake), horizontal component of $PGA_x = 0.36$ g and vertical component of $PGA_y = 0.24$ g

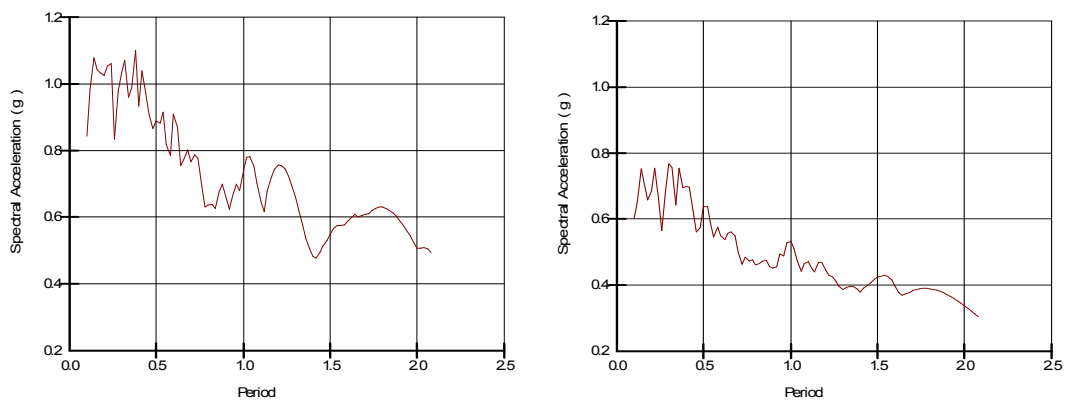


Figure 11. Spectral accelerations of horizontal and vertical component of synthetic earthquake, for damping ratio of $DR = 0.05$

Response of the dam subjected to MCE in time domain

The dynamic analysis in time domain was performed by using QUAKE, a geotechnical FE package for seismic analysis of structures made of local materials. The material model used in the analyses is non-linear or equivalent linear. In the equivalent linear analysis, the irregular seismic motion is transformed into equivalent cyclic motion (Geo-Slope QUAKE/W v5, 2001). The dynamic amplification factor, figure 12, $DAF = PCA / PGA = 0.91/0.36 = 2.52$, where PCA is Peak Crest Acceleration, and PGA is Peak Ground Acceleration. From the amplitude spectra of the response (figure 13), obtained in the dynamic analysis of dam in time domain, by using synthetic MCE

excitation, it was established that the most severe seismic action would be an earthquake with predominant periods in the range $T = 0.8 \div 1.0$ s, which is regular response for this embankment dam (Matsumoto N., ..., 2005.).

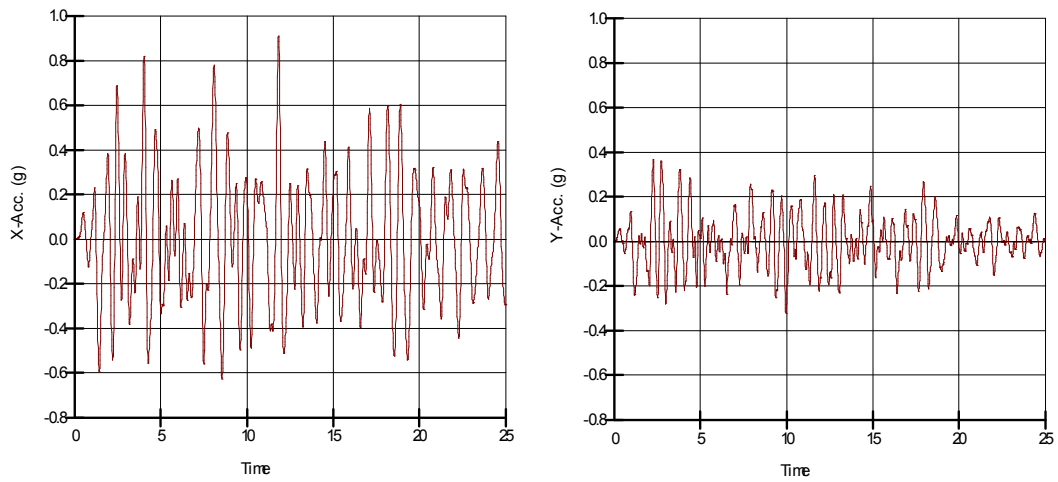


Figure 12. Absolute dam crest accelerations [g], horizontal and vertical, Peak Crest Accelerations: $PCA_x = 0.91$ g, $PCA_y = 0.37$ g

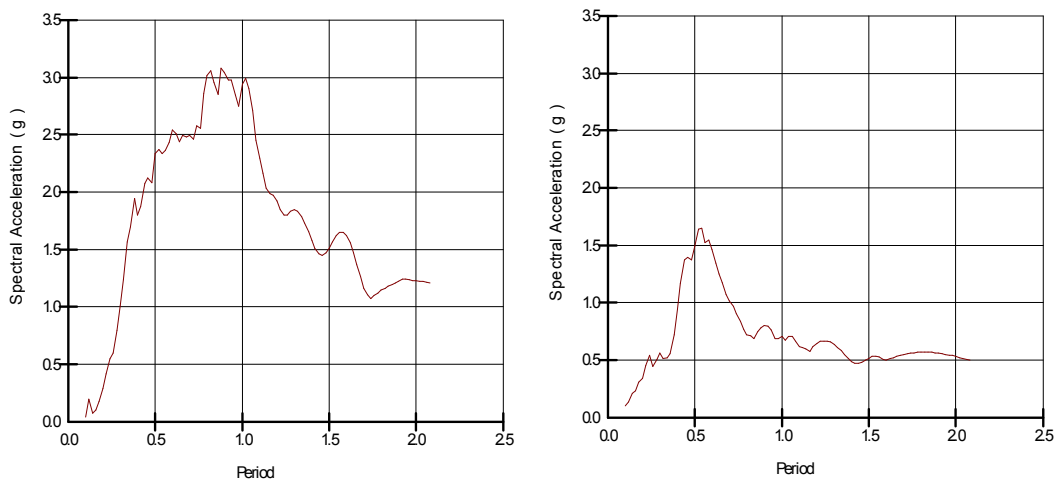


Figure 13. Response spectra of absolute accelerations for DR 0.5 at the dam crest, horizontal and vertical

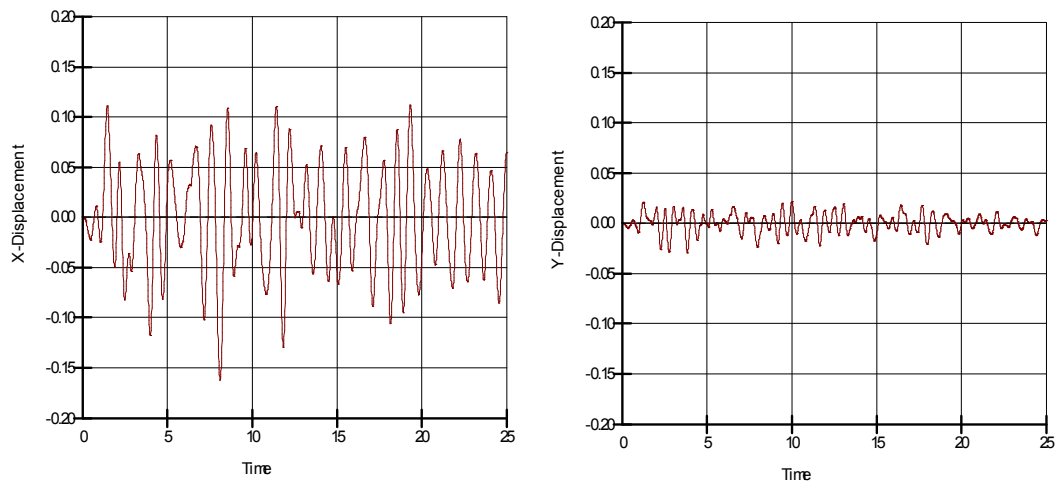


Figure 14. Relative dam crest displacements [m], horizontal and vertical

Permanent displacements

Embankment dams suffer damage in earthquakes due to: (1) slope instability that leads to partial or total collapse of the dam and (2) settlements, which reduce the dam height and may lead to overflow and erosion of the downstream slope, often with catastrophic consequences. The mechanism of settlements of the crest is a result of the additional compaction of the material (not compacted sufficiently during the construction), caused by the dynamic loading of the earthquake (Kramer S.L., 1996.). Estimation of freeboard reduction during the strong earthquake can be roughly taken as the additional displacement occurring under static loads. This displacement is caused by stiffness degradation, produced by dynamic loading. Namely, during the earthquake shaking, the dynamic shear modulus of the soil drops from a maximum value to a lower value, that is compatible with the dynamic shear strain (Wieland M., Malla S., 2002.).

Slope sliding mechanisms in rock-filled dams are a result of: (1) the shear stress increase during the seismic action (Petkovski L., Paskalov T., 2003.) and (2) the shear resistance decrease after the seismic excitation caused by: (a) excess pore pressure and (b) decrease of shear strength parameters from peak to residual values. When shear stresses exceed the shear strength of the material in certain zones (usually in shallow sliding surfaces), they cause permanent deformations which accumulate over the duration of the earthquake (Petkovski L., Tančev L., 2003.). The damage is usually manifested as longitudinal cracks along the crest or in the slopes, depending of the shape and the size of the critical sliding surface, but the structure does not collapse.

Permanent displacement along the dam slopes

The QUAKE analysis is the first step in the study of the dynamic response in time domain. The results of this analysis – the stresses during the earthquake – are used as an input in SLOPE in which the dynamic stability of the structure is evaluated (Geo-Slope SLOPE/W v5, 2001). The permanent displacements induced by earthquake shaking were calculated using the sliding block analysis procedure, proposed by Newmark (Paskalov T., 1985.). For this purpose, the yield acceleration of a potential sliding mass (figures 16) and the time history of the average absolute earthquake of this mass (in correlation with acceleration from the response, figure 12), had to be determined first. These sliding movements along the dam slopes, during the earthquake excitation, are a result of shear stress increase. When these stresses exceed the shear stress of the material (usually in shallow sliding surfaces near the dam crest), they caused permanent deformations, figure 15 and 17.

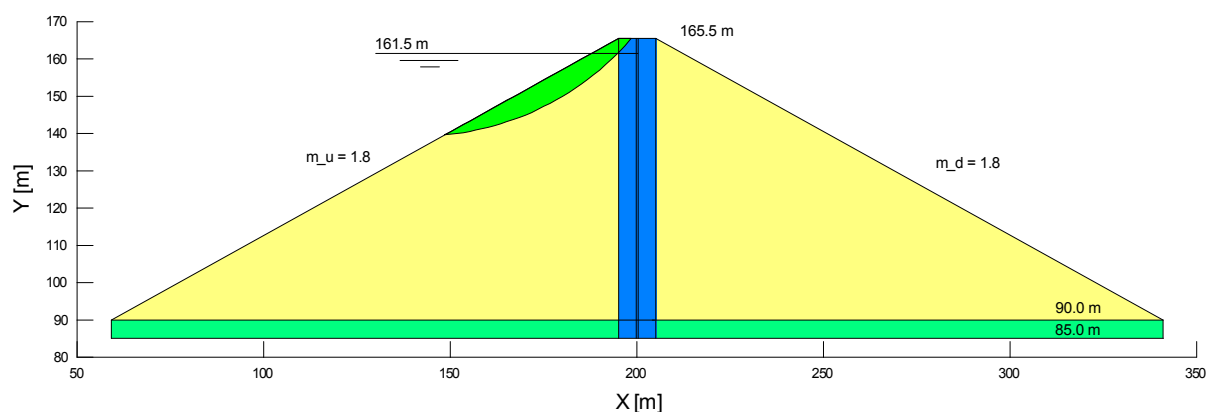


Figure 15. Shape of critical slip # 19, with maximal deformation of upstream slope $d = 1.27$ m upstream

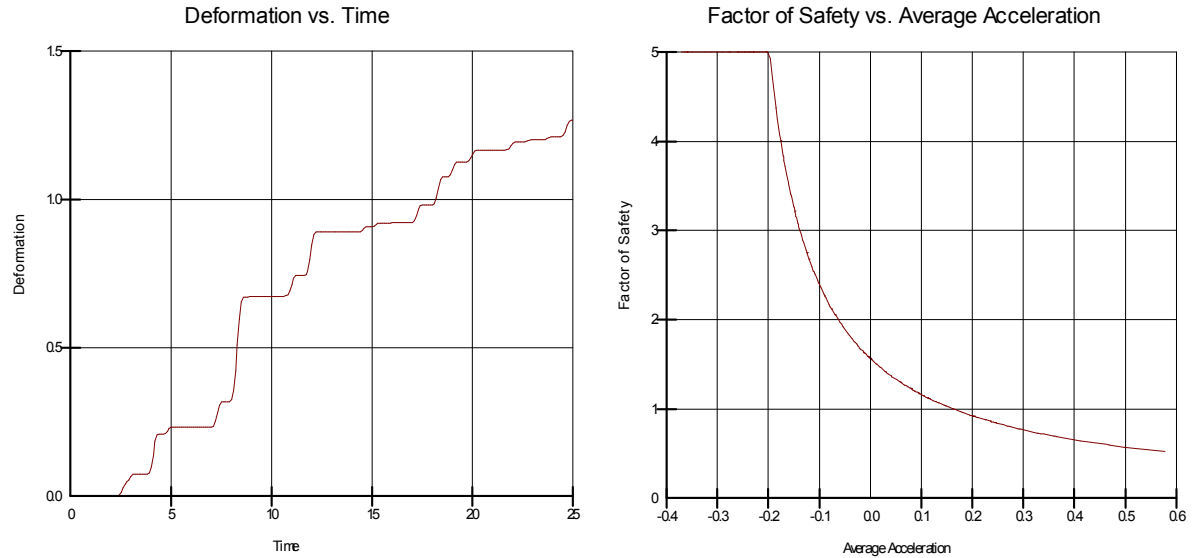


Figure 16. Deformation and yield acceleration of critical slip of upstream slope # 19

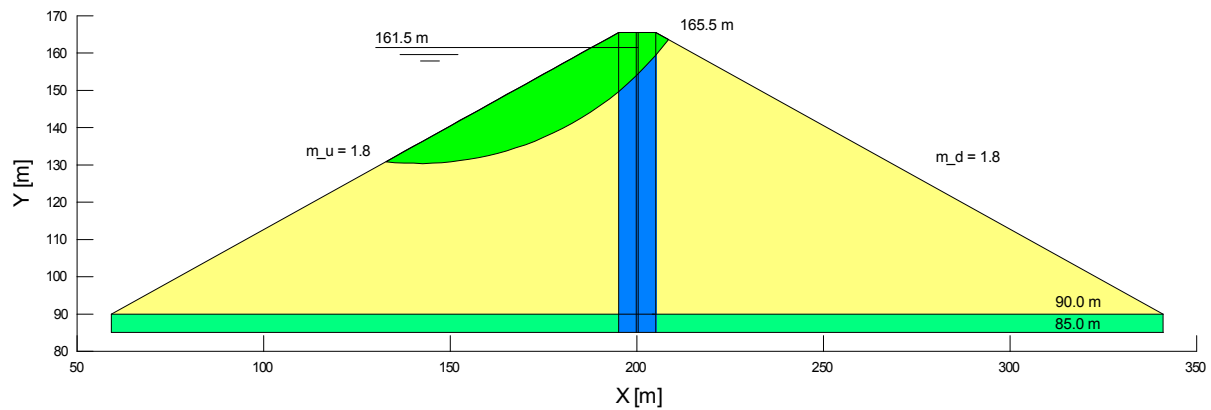


Figure 17. Shape of slip # 20 with maximum cutting of the asphalt diaphragm below normal water level, $d = 0.28$ m

Seismic settlement of the dam crest

The earthquake excitation produces also a general settlement due to the densification of the granular materials. Here, the seismic settlements are estimated by an alternative approximate procedure. During the earthquake shaking, the dynamic shear modulus of the local material drops from a maximum value G_{\max} to a lower value G , figure 18. Therefore, the stiffness degradation, produced by the dynamic loading, is assumed $G/G_{\max} \approx 0.3$. The seismic settlements can be roughly taken as the additional vertical displacements occurring under the 70 % of static loads, subjected from the stiffness degradation produced by the dynamic loading (figure 19).

The largest reduction of the freeboard, due to the permanent sliding displacement and seismic settlement is $Y_s = 1.29 + 1.04 = 2.33$ m, so the total reduction of the freeboard would be of the order of 2.33 m. The crest level of the dam, after the MCE, will be $1,065.5 - 2.33 = 1,064.06$ m ASL. The available freeboard over the full reservoir level (normal water level), will be $1,063.17 - 1,061.50 = 1.67$ m. Therefore, the reservoir water will not overflow the dam in this condition.

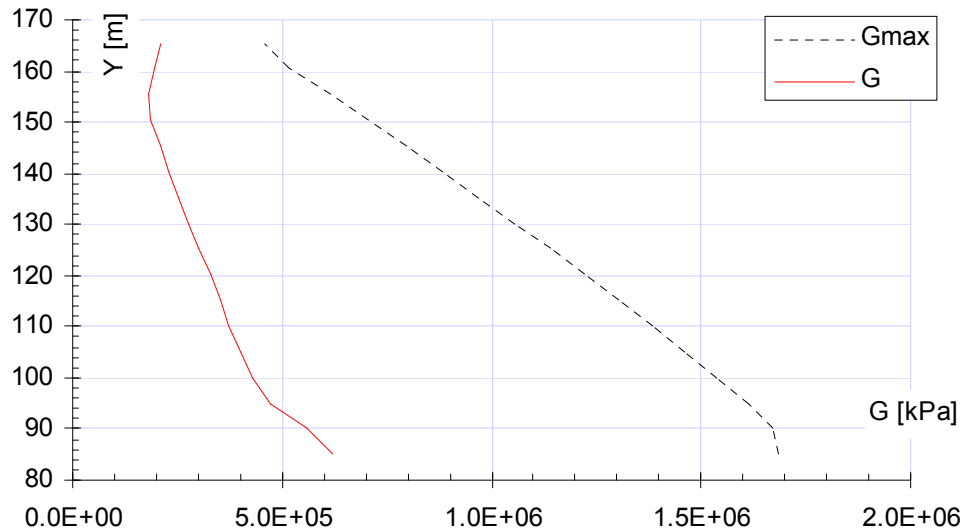


Figure 18. Dynamic shear modulus at the axes of the cross section of the dam, or vertical section at X =200.0 m

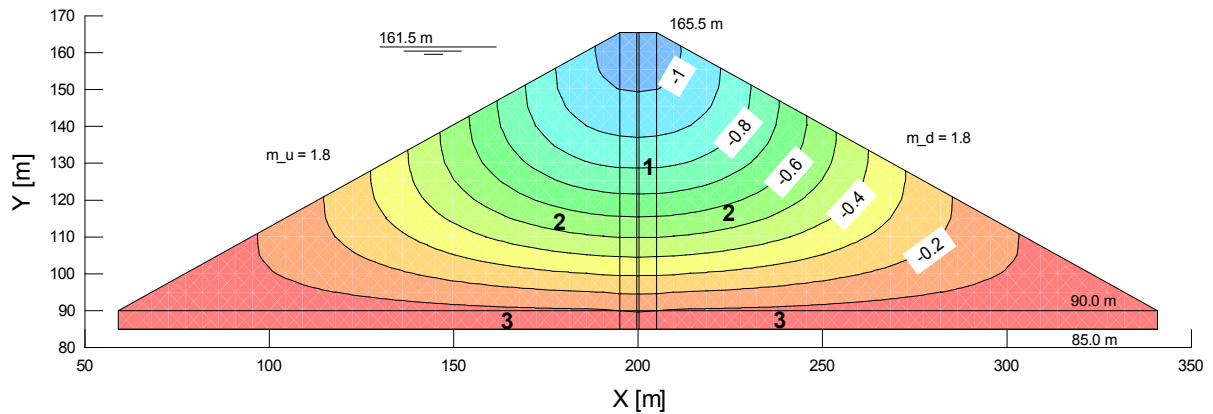


Figure 19. Vertical displacements due to the MCE stiffness degradation, obtained with LE model

ANALYSES OF THE RESULTS AND CONCLUSIONS ABOUT DYNAMIC STABILITY

The real seismic response of rock-filled dam with asphalt concrete diaphragm, which represents an exceptionally complex engineering problem, can be only be studied by using the dynamic analyses in time domain. The numerical experiment should be carried out in steps, with gradual increase of the complexity of the model. This approach enables following of the changes in the results as a consequence of the sophistication of the model, which can be either explained by the means of engineering logic or discarded as a result of numerical instability. This type of numerical experiment provides valuable insight into the sensitivity of the solution to the level of approximation of the problem. The main conclusions of this research are as follows:

(a) In the case of rock-fill dam with core, there is water pressure in the embankment, and we have to deal with effective stresses. The key difference of rock-fill dams with upstream impervious screen and dams with central synthetic diaphragm is treatment of pore pressure. Therefore, for the actual condition of earthquake loading, which is reservoir at normal level, the critical position for stability of the structure is: (1) downstream slope - for dam with synthetic screen and (2) upstream slope - for dam with asphalt concrete core.

- (b) During the maximum credible earthquake (MCE), sliding displacements of the order of 129 cm (upstream) could take place along the dam slopes, near the crest. These large displacements are for the shallow slip surfaces, which do not cut the impervious element of the dam - the asphalt diaphragm.
- (c) The displacements of the deep slip surface, which cut the whole dam crest and the asphalt diaphragm below normal water level of 1061.5 mASL, is 28 cm, which cannot destroy the functionality of impervious diaphragm, because its thickness is 60 cm.
- (d) The safety of the dam against the overflow can be accepted, because after the reduction of the freeboard due to MCE excitation, from 4.0 to 1.67 m, there is not any possibility of catastrophic and uncontrolled water release from the reservoir.

However, if the dam is subjected to MCE, than the permanent displacement will be large (upstream), which no doubt is serious structural damage of the dam. The shape and the size of the sliding surface shows that, regardless of the value of the permanent deformations, there is no danger of rupture large than 0.3 m in the asphalt core. Therefore, it could be confirmed that uncontrolled emptied of reservoir is not possible, which means that there is no risk of collapse of the structure as a whole. The final conclusion from numerical experiments of dynamic response of this dam, subjected by MCE, is that it possesses enough safety against dam brake due strong earthquakes, which is the main aim of the dynamic analysis.

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