ДГКМ
ДРУШТВО НА ГРАДЕЖНИТЕ КОНСТРУКТОРИ НА МАКЕДОНИЈА

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ANALYSIS OF THE SEISMIC OF AN ARCH-GRAVITY DAM

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ABSTRACT

The Janneh dam is designed as an arch-gravity Roller Compacted Concrete dam (H=157 m), currently under construction in Lebanon. It was initially designed as a straight-gravity dam. However, due to seismic reasons, its layout has been curved.

In the paper is performed numerical analysis of dam Janneh, by spatial (3D) numerical models, thus applying two approaches for seismic analysis regarding the simulation of the water effects: by Westergaard method and as reservoir modelled as compressible water fluid. Namely, the model consists of dam body, rock foundation and water reservoir, modelled as sub-structural elements. The analysis were executed by application of code SOFiSTiK. Moreover, the time history analysis was carried out for Operational Basis Earthquake (OBE), with Peak Ground Acceleration PGA=0.37g.

The behaviour of the contact zone dam-foundation was simulated by interface elements of type spring, by specifying input parameters for non-linear behaviour of the zone.

Keywords: *dams, water effect, time history analysis, seismic response.*

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1. INTRODUCTION

The construction of arch gravity dams under favorable conditions (shape of the valley, strength of the bedrock, availability of construction materials) is a more and more competitive alternative. More specifically, when Roller Compacted Concrete (RCC) is used, such alternative may be comparable to a Conventional Vibrated Concrete (CVC) arch dam in terms of costs of the project. Moreover, the higher overall thickness of the dam allows its construction on bedrocks of lower quality compared to that required for CVC arch dams.

2. SPECIFIC BEHAVIOUR OF ARCH-GRAVITY DAMS

The behavior of straight gravity dams on wide valleys is well known and several international guidelines may be used in order to assist their design:

- The dam withstands the water pressure by means of shear strength at the dam/foundation interface and at several weak planes in the dam body and/or within the bedrock;
- Crack opening at the upstream toe is generally only allowed for unusual and extreme load cases (occurrence of MCE type of earthquake).

Except for high-seismicity sites, 2D rigid block analysis is usually considered sufficient to assess the stability of arch gravity or gravity dams. In cases when the layout of a gravity dam is curved with a small enough radius of curvature, arch effect is triggered, even under normal operating conditions. The arch effect laterally transfers a part of the water pressure to the abutments of the dam. This leads to an offloading of the central blocks and an overloading of the bank blocks.

In the present case study is analyzed the seismic response of Janneh dam (H=157m), currently under construction in Lebanon, thus applying numerical models including two approaches for simulation of the water effect: by Westergaard method (added masses) and case of reservoir modelled as compressible water fluid.

3. GEOMETRY OF THE DAM

The Janneh dam is designed as arch-gravity RCC dam. The dam was initially designed as a straight-gravity dam. However, due to seismic reasons, its layout has been curved.

The definition of the upstream and the downstream faces of the dam is cylindrical (simple curvature). The dam layout and typical cross sections A-A (at block B5) and B-B (at block B0) are displayed on Fig. 1. The maximal height of the dam above the excavation is designed at 157.0 m, with crest width of 10.0 m and length of 300.0 m. Maximal width at the dam base is designed to 66.0 m. The crest elevation is at 847.0 masl. Downstream slope from elevation 831.2 masl to elevation 752.4 masl is by slope 0.8H/1V.

4. DESCRIPTION OF PROGRAM PACKAGE SOFISTIK

The numerical analysis of dam Janneh is carried out by application of code SOFiSTiK, produced in Munich, Germany. The program is based on finite element method and has possibilities for complex modeling of the structures and simulation of their behavior. It also has possibility in the analysis to include certain specific phenomena, important for realistic simulation of dam's behavior, such as: discretization of the dam and foundation taking into account the irregular and complex geometry of the structure, simulation of stage construction, simulation of contact behavior by applying interface elements and etc. in order to assess the dam behavior and evaluate its stability. The code SOFiSTiK in its library contains and various standards and constitutive laws (linear and non-linear). Moreover, the code SOFiSTiK has possibility for spatial (3D) modelling of the structures, most common applied in case of arch and arch-gravity dams.

Within the stage of numerical analysis, following steps must be undertaken, typical for this type of analysis: (1) choice of material parameters and constitutive laws (concrete. rock foundation and interface elements); (2) discretization of the dam and the rock foundation and (3) simulation of the dam behaviour for the typical loading states (as required in the topic formulation).

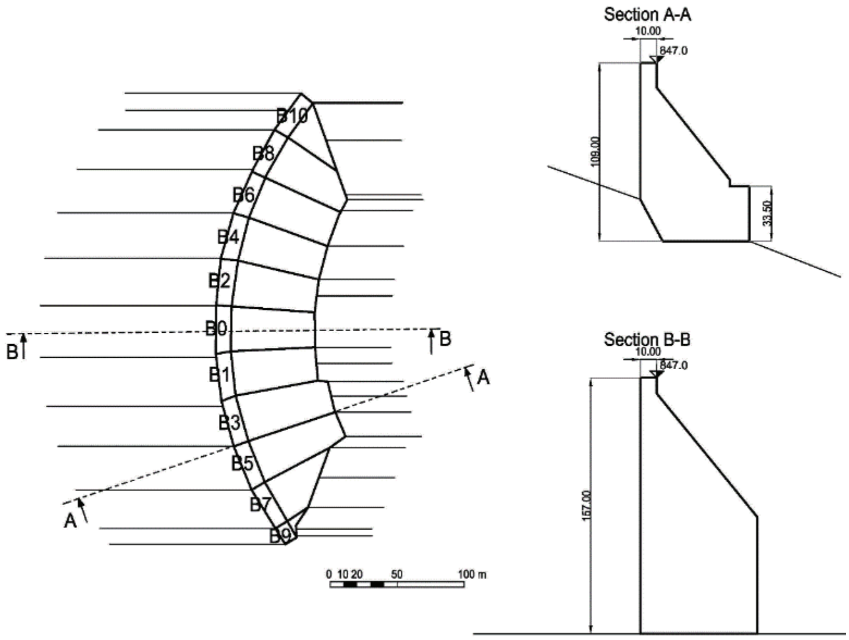

Fig. 1. Plane view and cross sections A-A and B-B of Janneh dam.

4.1 Input parameters and constitutive laws for the materials

The choice of material parameters, as input data for the stress-deformation analysis is complex process, taking into account various factors and influences. The stress state of the rock foundation in the numerical model is taken into account as initial state. A linear constitutive law is applied for the rock masses in the foundation, with input data specified in Tab. 1, according to the topic formulation.

The dam body is planned to be constructed by Roller Compacted Concrete (RCC). The dam construction will be simulated by subsequent horizontal layers of approximately equal height. Total of 10 subsequent horizontal layers (groups) are applied in the numerical model. The concrete is adopted as linear-elastic material, with input parameters also specified in Tab. 1.

The water load is important specific phenomena in case of dams analysis. Rather complex is the water effect simulation in numerical models in case of seismic loads, where there are generally two approaches – by Westergaard method (added mass) or reservoir modelled as compressible fluid.

Table 1. Material parameters.

Material	Density (kg/m^3)	Deformation modulus (GPa)	Poisson's ratio	c (kPa)	ϕ (°)	Tensile strength (MPa)
Concrete	2400	20	0.2	-	-	-
Bedrock	2800	25	0.25	-	-	-
Water	1000	0.00000125	0.499	-	-	-
Dam/ foundation interface	-	-	-	0	45	0

4.2 Discretization of dam body and foundation

Numerical analysis in the report are performed by spatial (3D) model, where the dam body and the foundation are modeled with volume elements. A powerful and reliable finite element should be applied in case where an analysis of structure with complex geometry and behavior is required, having in consideration that the correctly calculated deformations and stresses are of primary significance for assessment of the dam stability. In this case, for discretization of the dam body and the rock foundation are applied quadrilateral finite element (as auxiliary elements, type quad, by 4 nodes), volume finite element (type bric, by 8 nodes) and interface element of type spring.

The model is composed of dam body, rock foundation and interface elements at contact dam-foundation. The rock foundation is modeled according to the specified geometry [1]. More precisely, the numerical model captures the dam body, limited by dam site shape apropos the rock foundation, with approximate length upstream of the central section 248.0 m and downstream of 265.0 m. The rock foundation below the dam is taken in consideration at approximate depth of 223.0 m. The length of rock foundation in the left bank is approximately 227.0 m, while in the right bank the length is 242.0 m. The spatial model has geometrical boundaries, limited to horizontal and vertical plane. In these planes are defined the boundary condition of the model (Fig. 2). The horizontal plane in the lowest zone of the model is adopted as non-deformable boundary condition, vertical planes perpendicular on X-axis are boundary condition by applying fixed (zero) displacements in X-direction and vertical planes perpendicular on Y-axis are boundary condition by applying fixed (zero) displacements in Y-direction. The discretization is conveyed by capturing of the zones of various materials in the model – concrete and rock foundation.

The behavior of the contact zone dam – foundation is simulated by interface elements (type "spring", Fig. 3). Namely the interface elements are used to model the contact dam-foundation, which enable differential displacements in the zone. Interfaces in principle, act as compressed ones, i.e. the relative displacement along the contact are in fact displacements in tangential direction of the spring. The behavior of the springs is described by two parameters: normal stiffness C and tangential stiffness Ct. On the current level of development in geotechnics, several approaches of shear strength testing are known, but there are still cases when it is very usual to adopt or assume them, and very often this problem is not even treated. Along with this, it is very difficult to conclude how close is the prognosis of the parameters to the actual conditions which are expected in the phase of exploitation of the structures. The theory and methodologies for determination of shear strength along discontinuities are also developed [2, 3]. Furthermore, there are some developed methodologies to define shear strength along interfaces concrete-rock mass in a phases of design for concrete dams [4, 5]. Some detail analyses of shear strength parameters in designing of fill dams can be found in Barton and Kjaernsli [6], while summarized overview is given by Tančev [7, 8]. The research of input data for the stiffness properties by specially arranged laboratory direct shear test can be done by applying the Hoek's apparatus [9]. In the present case there are no specified input data and for simulation the non-linear behavior are adopted values of $C=1\times10^8$ KN/m^3 and $Ct=1\times10^6$ KN/m^3, and also is applied input for non-occurrence of tension stresses along with input of friction and cohesion parameters, thus applying Mohr-Coulomb constitutive law for the contact zone behavior. Namely, the nonlinear behavior of the contact zone is enabled by input of non-possibility for crack occurrence i.e. if tension stresses occurs there is possibility for crack generation in that zone, depending whether or not are exceeded the allowable tension stresses of the concrete.

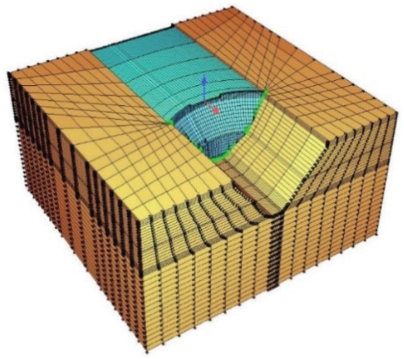

Fig. 2. View of the numerical model dam-reservoir-foundation, with boundary conditions.

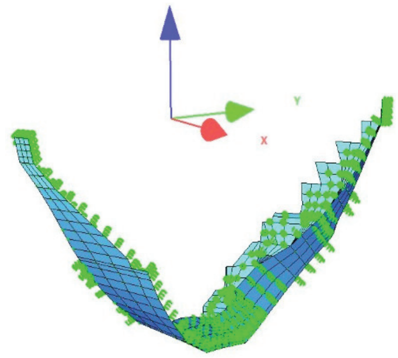

Fig. 3. Contact zone dam-foundation, modelled with interface elements.

5. SEISMIC ANALYSIS

The aim of this section of the report is to study the seismic behaviour pf the dam. The seismic reponse is analyzed in case of Operating Basis Earthquake (OBE). In the further analysis for estimation of the dynamic stability of the dam, it is required to determine the response of the structure under action of design earthquake Z_1 (synonym for OBE). For the seismic analysis, in case of OBE, Z_1 a duration t=15s is adopted for the acceleration record. For the seismic analysis in time domain, for Z_1 the peak ground acceleration is $PGA_{1x} = 0.37g$. The earthquake action is simulated by time history analysis.

5.1 Initial state for seismic analysis

The carried out analysis in case of static loading, for full reservoir (water elevation at 839.0 masl) serves as initial state for the seismic analysis.

As previously mentioned, the water effect within the seismic analysis can be modeled by Westergaard method and by treating of the reservoir as compressible fluid. The general formulation by the Westergaard method assumes parabolic distribution of the hydro-dynamic pressure along the dam upstream face, and the normal hydro-dynamic pressure in certain point of the curved surface is proportional on the full normal acceleration.

The various types of executed analysis offers possibility to research the time dependent parameters of the seismic response of the dam. Although, data regarding the maximal stress superposition, duration and zone of occurrence are possible to obtain. The input data is the time history of accelerations (Fig. 4), thus calculating the response during the excitation. By such method the seismic response of the structure in time domain is obtained by direct integration of the motion equation.

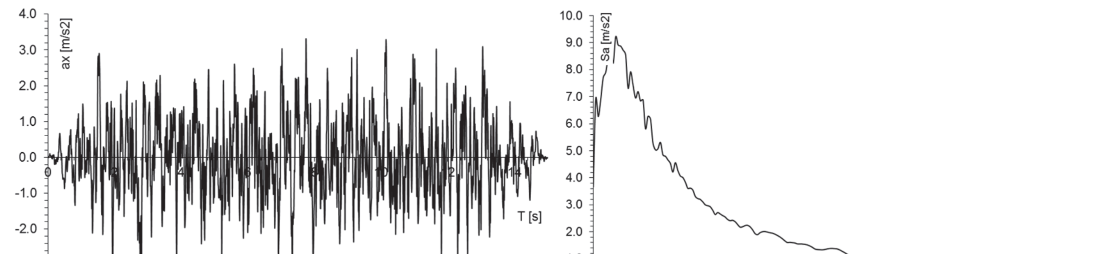

Fig. 4. Accelerograms of horizontal component of accelerations and response spectrum for damping DR = 0.05 for OBE, Z_1, $a_{x,max}$=0.37g.

5.2 Linear pseudo-static analysis based on site response spectrum acceleration

The dynamic properties of the dam in case of full reservoir are determined by analysis of 50 tone modes of the free oscillations. According to Eurocode 8 [10], recommendations, the percentage of activated mass should be over 90%, that is met here (97% in X-direction). The period of first modal shape is T_1=0.40 s and it is a case of symmetrical model shape, while the second is asymmetrical, by T_2=0.28 s (Fig. 5).

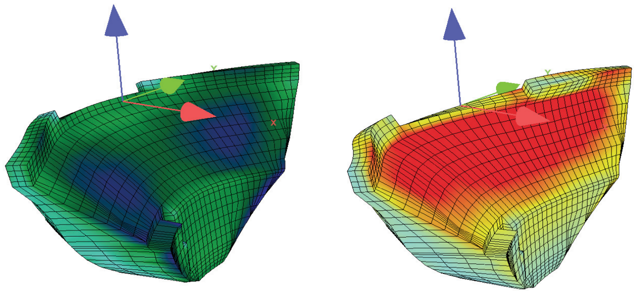

Fig. 5. First (left) and second (right) modal shape, f_1=2.49 Hz and f_2=3.49 Hz respectively.

Regarding the models with time history analysis a concise overview has been done for models no. 1 and no. 2 and in the following are displayed and commented some of the output results for accelerations and displacements during the excitation period.

5.3 The dam's response at OBE earthquake

The model no. 1 includes initial state of the dam, applied interface elements by Mohr-Coulomb law and water effect simulated by Westergaard method. The model no. 2 includes initial state of the dam, interface elements, following Mohr-Coulomb law and water effect simulated by applying the water as compressible fluid.

On Fig. 6 is displayed dynamic response of the dam and response spectra under action of the specified OBE earthquake in node no. 124, central node at dam crest for section B-B at elevation 839.0 masl, where maximal accelerations are expected to occur. From the output results it can be noticed that in case of model no. 1 the dam responds to OBE earthquake by amplification of the horizontal acceleration in the crest multiplied by 11 regarding the peak ground acceleration (dynamic amplification factor DAF=11). The dynamic response of the dam in case of model no. 2 under action of the OBE earthquake at node no. 124 is also displayed on Fig. 6, along with the response spectra. From the output results it can be noticed that dam responds by amplification of the horizontal acceleration in the crest multiplied by 8.15 regarding the peak ground acceleration (DAF=8.15).

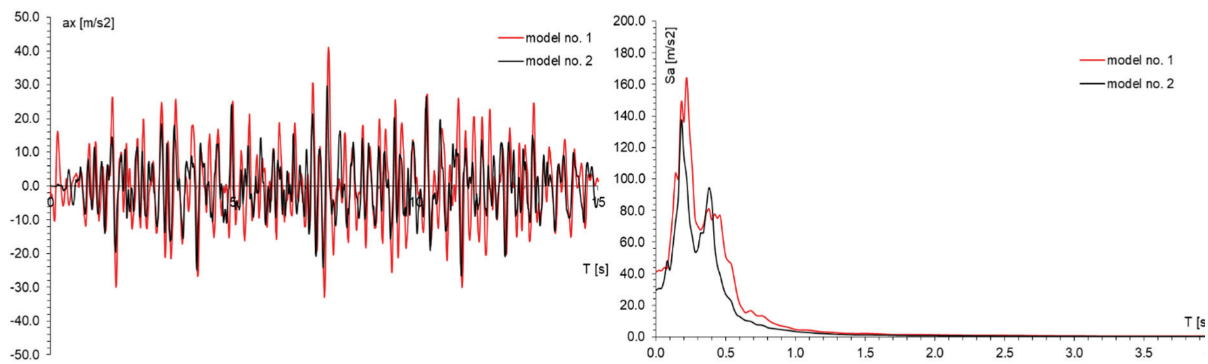

Fig. 6. Accelerations ax [m/s^2] ÷ T [s] and response spectra Sa [m/s^2] ÷ T[s], in horizontal direction for damping DR = 0.05, node 124 at action of (OBE, Z1) for model no. 1 and model no. 2.

In case of arch dams also the response at node at one quarter of the dam's crest is analyzed also. Namely, on Fig. 7 is displayed the dynamic response and the response spectra at node 14 (crest node at section A-A, elevation 839.0 masl approximately at one quarter of dam's crest length). It can be concluded that the dam's response is analogous as in case of central node 124, only with lower amplification factors. The obtained dam's response is more intensive by model no. 1, with value of dynamic amplification factor DAF=3.75, compared to model no. 2, with DAF=3.15.

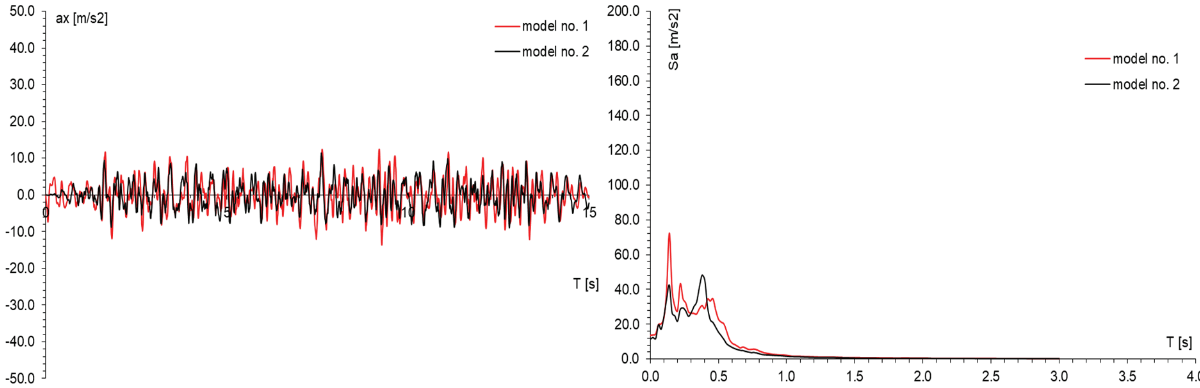

Fig. 7. Accelerations ax [m/s2] ÷ T [s] and response spectra Sa [m/s2] ÷ T[s], in horizontal direction for damping DR = 0.05, node 14 at action of (OBE, Z1) for model no. 1 and model no. 2.

On Fig. 8 are displayed relative displacements at node 124 at dam's crest. The total amplitude of the displacements is approximately 170 mm.

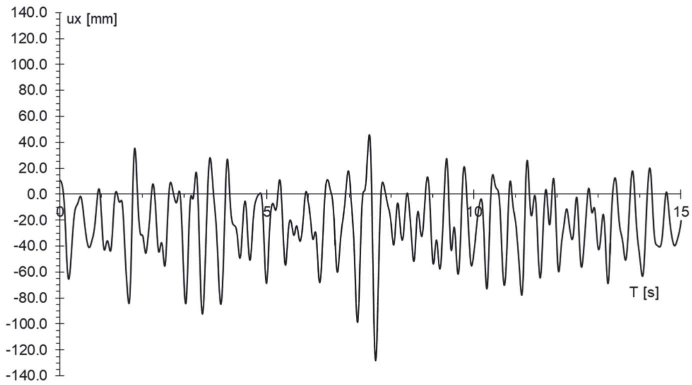

Fig. 8.1 Displacements ux [mm] ÷ T [s] in horizontal direction for damping DR = 0.05, node 124 at action of (OBE, Z1) for model no. 2.

6. CONCLUSIONS

From the numerical experiment of simulation of the structural behaviour of Janneh dam following concluding remarks can be drawn out:

1. The prediction of the arch gravity dam behaviour during construction, after construction, reservoir filling and service period by means of numerical models is of primary importance.
2. The interface elements are applied at the contact dam-foundation, thus simulating the contact behaviour. It is a very complex part of the modeling and it requires specific shear tests for determination of the input parameters (normal and tangential stiffness or more complex non-linear stress – strain curve).

3. The seismic analysis treats the dam stability at action of strong earthquakes. In the present case study are determined the dynamic properties of dam Janneh for full reservoir for 50 tone modes of the free oscillations. The period of first modal shape is T_1=0.40 s and it is a case of symmetrical model shape, If the natural periods of the dam are in the descending region, the dynamic response of the dam will decrease and cracks will occur. The values for the period and modal shapes are typical for such type of dam.
4. Regarding the obtained results from the time history analysis, in case of node. no. 124 (central node at dam crest) it can be stated that under action of OBE earthquake by PGA=0.37 g, the dam responds by DAF=11 (model no. 1) and DAF=8.15 (model no. 2). The diagrams of time history of accelerations have proper shape of dynamic response of arch gravity dams.
5. By analysis of the response spectra (for OBE excitation) it can be noticed that zone of resonant effects is in the interval (0.3-0.4) s, mainly corresponding to registered data for natural periods for such structures.
6. The simulation of water effect by Westergard method generates higher dynamic amplification factor compared to the approach where water is treated as compressible fluid. However, additional analysis are required by adopting various input parameters for the water fluid properties, as well and applying interface elements at contact zone dam-reservoir.

REFERENCE

[1] Andrian F., Agresti P., Mathieu G., (2017). 14th International Benchmark Workshop on Numerical Analysis of Dams, Theme B, Static and seismic analysis of an arch gravity dam, ARTELIA Eau & Environment

[2] Goodman R., (1968). Effect of joints on the strength of tunnels – Research on rock bolt reinforcement, Omaha District, Corps of Engineers Omaha, Nebraska

[3] Goodman R., Taylor R., Brekke T., (1968) A Model for the Mechanics of Jointed Rock.

[4] Anđelković Vl. (2001) Analysis of shearing modulus on bedrock-concrete interface. Monography: Managing water resources of Serbia (in Serbian)

[5] Jovanovski M., Gapkovski N., Anđelković Vl., Petrović Lj. (2002) Some possibilities for determination of bedrock-concrete interface shearing strength in Hoek's box. Proceedings from the First symposium of Macedonian Association for Geotechnics, Ohrid, pp.78-86

[6] Barton N., Kjaernsli B. (1981) Shear strength of rockfill. Journal of the geotechnical engineering division, Vol.107, N0 GT7

[7] Tanchev L., (2014) Dams and appurtenant hydraulic structures, Second edition, A.A. Balkema Publ., CRC press, Taylor & Francis Group plc, London, UK

[8] Tančev L. (1989) Static analysis of rockfill dams. Studentski zbor, Skopje (in Macedonian)

[9] Mitovski S., (2015) PhD thesis, Static analysis of concrete dams by modelling of the structural joints, Ss Cyril and Methodius University, Civil Engineering Faculty – Skopje

[10] Eurocode 8: Design of structures for earthquake resistance, 1998.