НУМЕРИЧКА АНАЛИЗА НА БРАНАТА „СВ. ПЕТКА“

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Резиме

Евалуацијата на стабилноста на браните е исклучително комплексна задача, која мора да се разгледува од повеќе аспекти со анализа на различни товарни состојби. Критериумите за конструктивната стабилност мора да се исполнат за да се постигне стабилноста на браната.

Примената на методот на конечни елементи доведе до значајни промени при анализата на стабилноста на лачните брани, овозможувајќи спроведување нелинеарна просторна анализа, анализа за различни товарни состојби (влијание од сопствена тежина, товар од вода, температурен ефект), како и вклучување на карпестата средина во анализите. Исто така, овозможена е примена на контактни елементи за симулирање на однесувањето на контактните зони бетон - карпа во основата и бетон - инјекциона смеса во фугите на браната.

Во предметниов труд е даден приказ на евалуацијата на стабилноста во однос на состојбата напрегања - деформации, илустрирано со примерот на лачната брана „Св. Петка“, со висина од 64 m, изградена на реката Треска во близина на Скопје, а пуштена во употреба во 2012 година.

Клучни зборови: лачна брана, стабилност, напрегања, поместувања.

NUMERICAL ANALYSIS ON DAM ST. PETKA

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Summary

The assessment of dam's safety is complex task that has to be considered from numerous aspects by including of various loading states. The structural safety requirements must be met in order to ensure the dam stability.

The application of the finite element method has led to significant changes in the treatment of the arch dam stability, enabling non-linear spatial analysis, analysis of the arch dam for different loading states (gravity and water load, temperature effect), as well and inclusion of the dam foundation in the analysis. Also, application of contact elements for simulating the behaviour on the interface concrete-rock in the dam abutments and foundation as well and the behaviour on the interface concrete-grouting material in the joints in the dam body is enabled.

In this paper some aspects of the safety evaluation in accordance with the advanced approach for dam analysis regarding the stress-strain state is illustrated for the case of St. Petka dam, a 64 m high arch dam on River Treska in near by of city Skopje, commissioned in 2012.

Key words: arch dam, safety, stresses, displacements.

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1. INTRODUCTION

The dams, having in consideration their importance, dimensions, complexity of the problems that should be solved during the process of designing and construction along with the environmental impact are lined up in the most complex engineering structures [Tančev, 2005; Novak et all., 2007]. In Macedonia, up to now are constructed 27 large dams. Different types of dams are represented, having in consideration the various geological, topographical and hydrological conditions, among which 19 are embankment dams, 6 concrete arch dams and 1 concrete multiarch dam. The stored water is used for meeting the demands for water supply of population and industry, irrigation, production of electricity, flood and erosion control, provision of minimum accepted flows, recreation and tourism. The total stored water volume is about 2.4×10^9 m^3. The assessment of the stability and the behaviour of the dam during construction, at full reservoir and during the service period is of vital meaning for this type of structures. In 2012 was completed Saint Petka arch dam, in near by of city of Skopje, as final part of the cascade system on river Treska, along with dams Matka and Kozyak. The paper deals with some aspects of the numerical modelling of St. Petka dam, performed by means of the SOFiSTiK code.

2. ST. PETKA DAM

St. Petka dam is double curved thin arch dam, with height of 64.0 m (Fig. 1). The crest elevation is at 364.0 masl, with crest thickness of 2 m and length of 115 m, while the lowest elevation is at 300.0 masl, with thickness of 10.0 m. On the right bank the low quality zone of the rock foundation is replaced by concrete block, thus avoiding the weak foundation zones. The dam site is characterized by symmetric shape with steep slopes, apropos the left abutment is with inclination of 60°, while the right abutment has an inclination of 50°. The normal water level is at 357.30 masl, with reservoir volume of $12.4\bullet10^6$ m^3. The main purpose of the dam is electric power production. The St. Petka dam was commissioned in August, 2012.

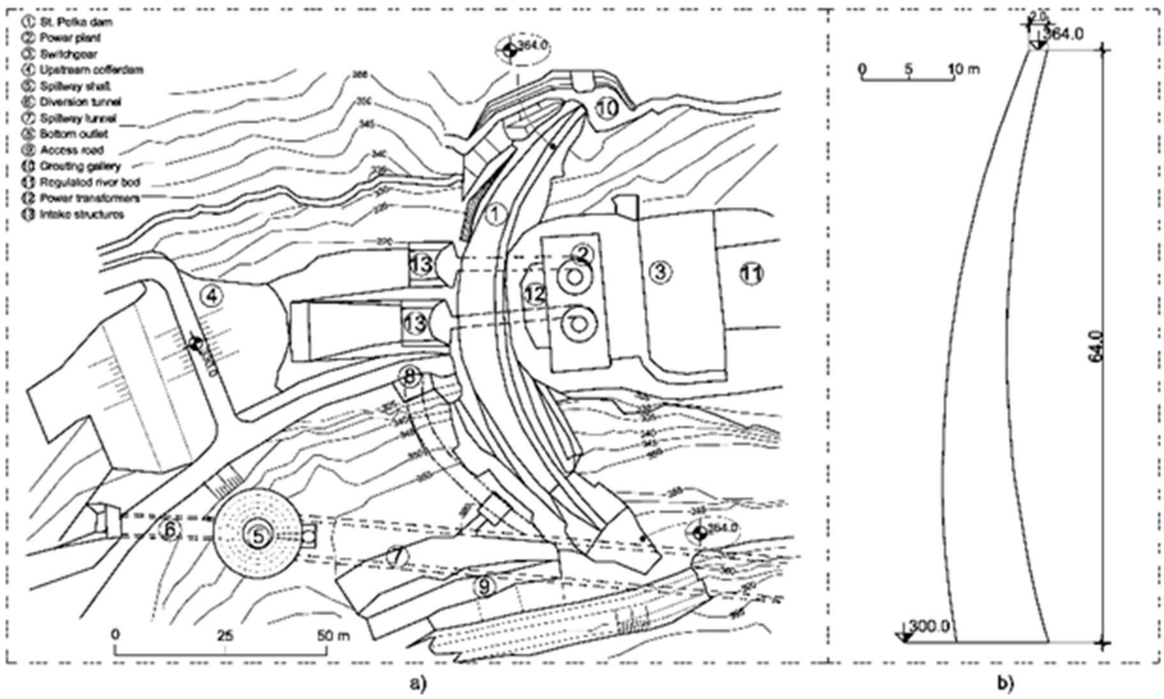


Figure 1. St. Petka dam. a) layout and b) typical cross-section.

3. DAM MODELLING

The static analysis of St. Petka dams dam is conveyed with application of the code SOFiSTiK. The program offers possibilities for complex presentation of the structures and simulation of their behaviour as well and including in the analysis of certain specific phenomena (automatic mesh generation based on given geometry, application of various constitutive laws, simulation of dam construction and reservoir filling, simulation of contact behavior etc.).

The following steps are required for the numerical analysis to be performed: (1) choice of material parameters – constitutive laws (one of the most complex tasks during the analysis), (2) adoption of dam geometry and (3) simulation of the dam construction and reservoir filling.

3.1 Input parameters for the materials

The program SOFiSTiK offers rich library of constitutive models for the materials, such as standard (concrete, steel, timber, soil and rock), but also and non-standard with option of user – defined laws of specific parameters.

The foundation is composed mainly of marble. The carbon schist appears as plates with mica sub-layers between them. At the upper zones of the dam site are detected diluvia and alluvial sediments, later excavated during the construction stage. The geotechnical input data for the modeled zone of the rock foundation are adopted on base on overall data from the geotechnical investigations and control testing before and during construction process (Synthesis Elaborate, 2004). Linear constitutive law is applied for the materials in dam foundation The input data for the materials are specified in Tab. 1. Three main fault zones are also included in the rock foundation model.

Table 1. Zoning per parameter of velocity of elastic waves of the massive deformability.

Zone	Velocity of elastic waves Vp [m/s]	Deformation modulus D [MPa]	Poison's coefficient μ
Fault	2500 – 3000	2500	0.30
Left bank	4000 - 4500	7000-8000	0.24
Right bank	3800 – 4000	5000-6000	0.26

The constitutive law for concrete is adopted according to EC 2, concrete type 30 [ICOLD, 2009; Eurocode 2, 1992]. The basic concrete parameters are given in Table 2.

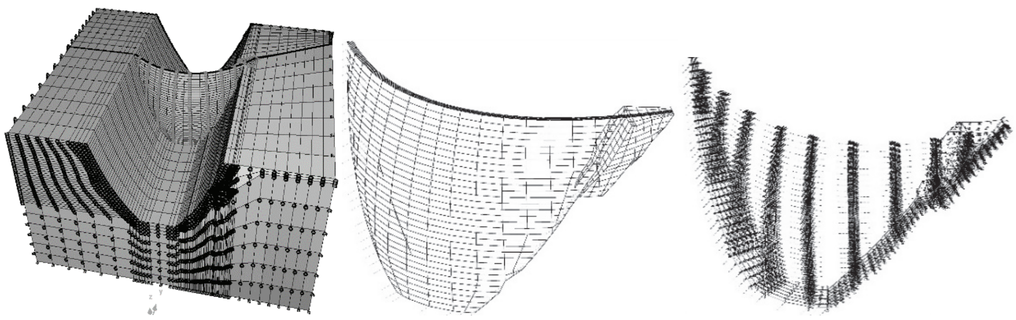
Table 2. Concrete parameters.

Parameter	Dimension	Value
Elasticity modulus	[N/mm^2]	31939
Poisson coefficient	/	0.20
Self-weight	[kN/m^3]	24.0
Nominal strength	[N/mm^2]	30.0
Coefficient of thermal expansion	[×10^{-6} on °C]	1.0

3.2 Model geometry

The numerical model is composed of the dam body and the rock foundation. The rock foundation boundaries are adopted according to literature [ICOLD, 1987]. More detail, the numerical model is composed of dam body, limited by the dam site banks, and rock foundation, with length upstream and downstream of the dam central cantilever in interval (1÷2)H, where

H denotes dam height apropos a length of 100 m is adopted, while the rock foundation under the dam is adopted at depth of 65.0 m (Fig. 2a). By such parameters is defined the non-deformable boundary condition (displacements in X, Y and Z direction are fixed at the lowest section). The discretization is conducted by capturing of zones with different materials – concrete and rock foundation. Per dam crest length the dam is divided in vertical segments with length of 6-12 m. The segments are divided horizontally in 16 groups. The height of one segment is 4 m. The dam's thickness is divided in 6 layers (groups). In this way, the different distribution of the temperature along the thickness, measured from installed thermometers, are taken in the analyses adequately. The layers are created by specified number of volume bodies, constructed of one type of material. The concrete block, constructed to replace some weak rock zones in the right abutment of the dam site, is also included in the model (Fig. 2b). At contact surface dam - foundation as well and at contact surface of concrete blocks of the dam body (dam joints) are applied interface elements (Fig. 2c) in order to simulate the interaction behavior at contact of materials with different deformable parameters. The interface elements are of type "spring", thus connecting two areas of quadrilateral elements (applied only as geometry elements).



a) b) c)

Fig. 2. St. Petka dam model. A) Spatial view from stream side, b) Discretization of the dam body and the concrete block, c) Interface elements at contact dam-foundation and in at contact concrete-grout mix in dam joints.

3.3 Interface elements

The behavior of the contact zone dam/foundation and contact zone between the dam blocks (joints) is simulated by interface elements of type "spring" (Fig. 3).

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 generally is described by two parameters: normal stiffness C and tangential stiffness Ct. The interface element can be applied by input of normal (axial), tangential (lateral) and rotational constant, prestressing, sliding and non-linear effects (failure load for compression or tension, creep, friction coefficient, cohesion, strain etc).

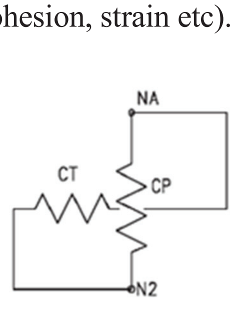


Fig 3. Interface element of type "spring".

On the current level of development in geotechnics, several approaches of shear strength along contact zones 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 [Goodman 1974]. Furthermore, there are some developed methodologies to define shear strength along interfaces concrete-rock mass in a phases of design for concrete dams [Anđelković, 2001; Jovanovski M. et all, 2002; Barton and Kjaernsli, 1981; Tanchev, 1989; Tanchev 2014]. The investigation and adoption of input data for the specified stiffness properties can be done by specially arranged laboratory direct shear test by applying the Hoek's apparatus [Mitovski, 2015]. Namely, the values for the lateral constant were adopted on base of laboratory and „in situ" testing on the shear strength parameters by Hoek apparatus at the contact concrete/rock foundation and concrete/grout mixture in the dam joints (Fig. 5). The output results from one sample from the testing are displayed on Fig. 5. The adopted values are $C_p=16\times10^6$ kN/m^3 and $C_t=1\times10^6$ kN/m^3 for contact dam-foundation and $C_t=5\times10^6$ kN/m^3 for contact concrete/grout mixture.

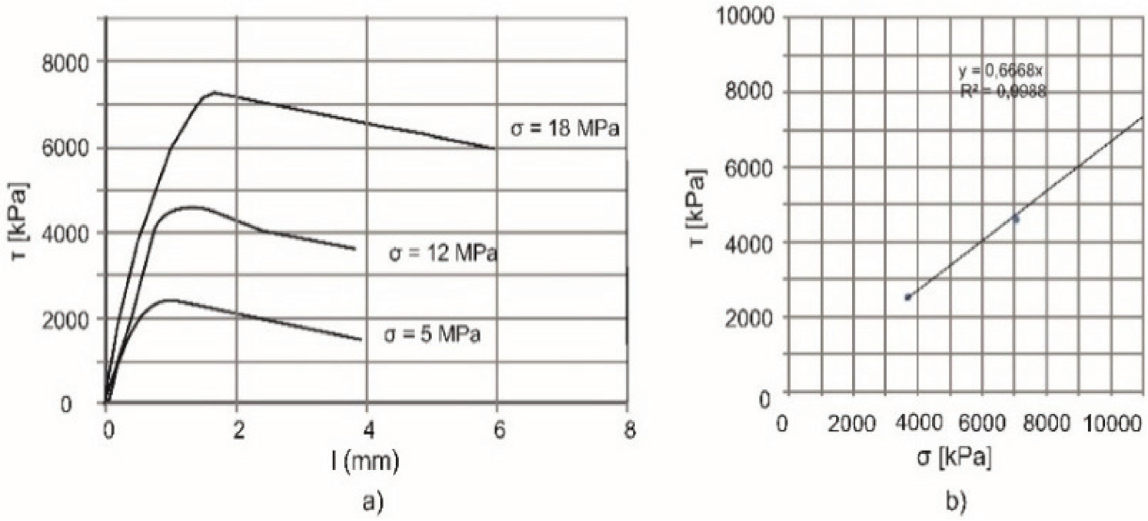
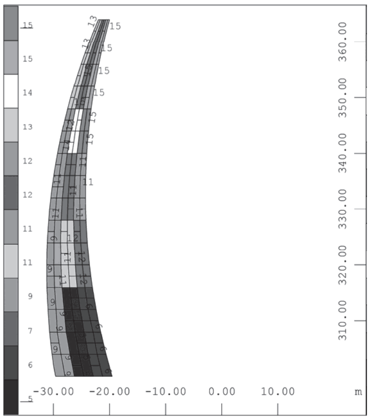


Fig 5. Output results for testing sample 378 I/A, contact concrete-grout mixture at constant normal load of 6, 12 and 18 MPa. a) Dependence shear load-displacement, b) dependence shear/normal load τ=σ(s).

3.4 Dam loading

The analyzed loading states of the dam include state after dam construction, joint grouting and first reservoir filling. The state after dam construction considers dead weight and temperature load, while the state at first reservoir filling considers dead weight, temperature load (Fig. 6), grouting load and hydrostatic pressure (Fig. 7). The dam filling commenced in June 2012 and reached normal water elevation of 357.30 masl at end of July 2012. The temperature load is adopted in accordance with the monitoring data of the temperature in the dam body for the specified loading states apropos previously specified time periods. The dam joint grouting was performed in the period March-April, 2012. The grouting was done in five stages in height of 12 m per stage, with grouting pressure of 7 bar at bottom and 5 bar at top of one grouting section of 12 m, as applied and displayed on Fig. 8.



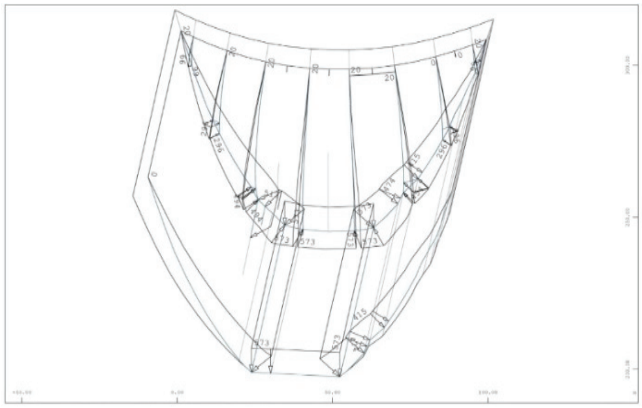


Fig. 6. Temperature loading, applied on the dam. Fig. 7. Water loading, applied on the dam.

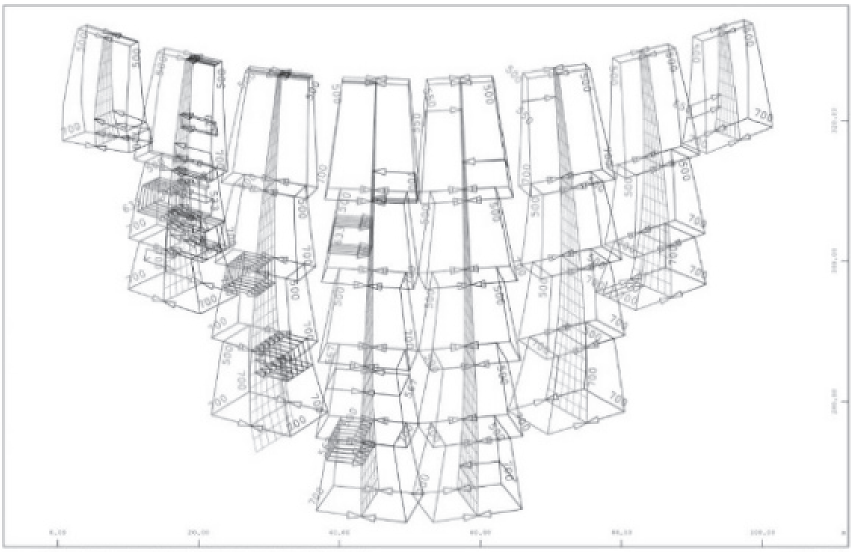


Fig. 8. Grouting load, applied along surface of both sides of the dam joint.

4. OUTPUT RESULTS

The dam behaviour is assessed upon values and distribution of the displacements and stresses in the dam body. The dam analysis includes four stages within period March-July, 2012, apropos period of grouting process and reservoir filling. The dam construction in 16 horizontal layers is simulated with taking in consideration of the dam foundation stress state as initial state and afterwards state at full reservoir up to normal water level of 357.30 masl was simulated. This enables realistic simulation of the chronology of dam loading during construction and at full reservoir.

On Fig. 9-a are displayed horizontal displacements in the upstream-downstream direction in the central dam cantilever after construction, while on Fig. 9-b are displayed horizontal displacements upon grouting stage. It can be seen that in the zone till 70% of the dam height the displacements are in upstream direction, and on the remaining zone they are directed in downstream direction, with maximal displacements in the dam crest of 5 mm. The horizontal displacements upon grouting stage are directed upstream, with increased maximal value of 9 mm on the upstream face of the dam, at around 60% of the dam height, mainly due to the increase of temperature effect.

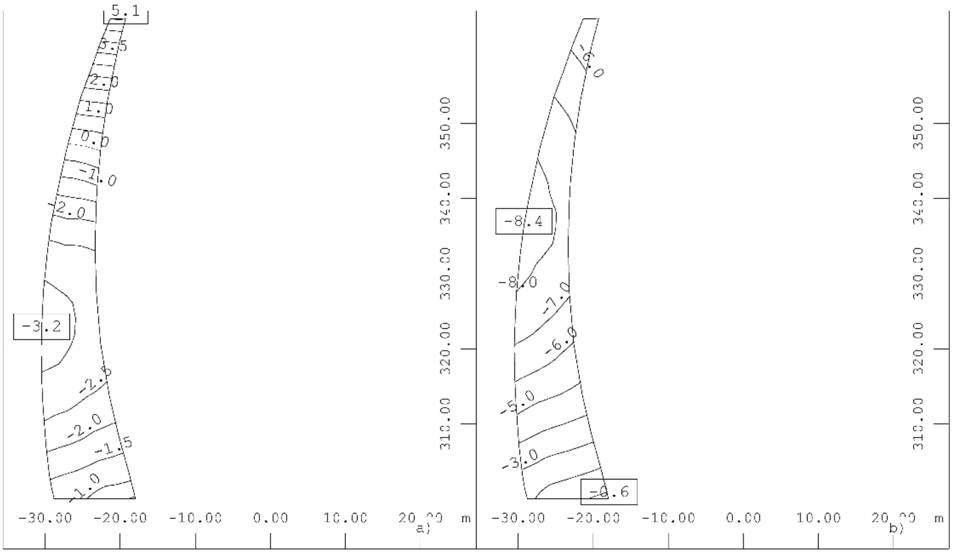


Fig. 9. Horizontal displacements in the dam central cantilever [mm]. a) Isolines of displacements after dam construction; b) Isolines of displacements upon grouting stage.

On Fig. 10-a are displayed horizontal displacements in the upstream-downstream direction in the central dam cantilever before reservoir filling, while on Fig. 10-b are displayed horizontal displacements at full reservoir. Due to the temperature rise, the dam as expected deforms in upstream direction. Maximal displacements occurs on the upstream face of the dam, value of 14 mm, approximately at 75% of the dam height. For the case of full reservoir, due to the water load, the displacements are with lowered intensity to the upstream side. Maximal displacements occurs in the dam crest, value of 12 mm.

By analysing the horizontal displacements after dam construction n longitudinal section per dam axis (Fig. 11-a) in the lower zone occurs displacements in upstream, while in the upper zone in the downstream direction. The isolines of horizontal displacement at full reservoir are in upstream direction, with maximal value at dam crest of 13 mm.

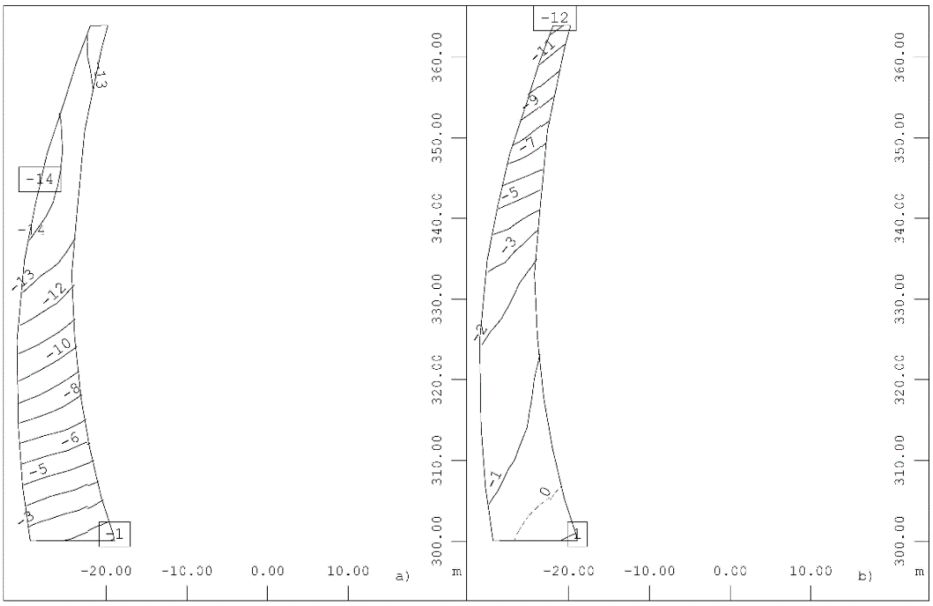


Fig. 10. Horizontal displacements in the dam central cantilever [mm]. a) Isolines of displacements before reservoir filling; b) Isolines of displacements at full reservoir.

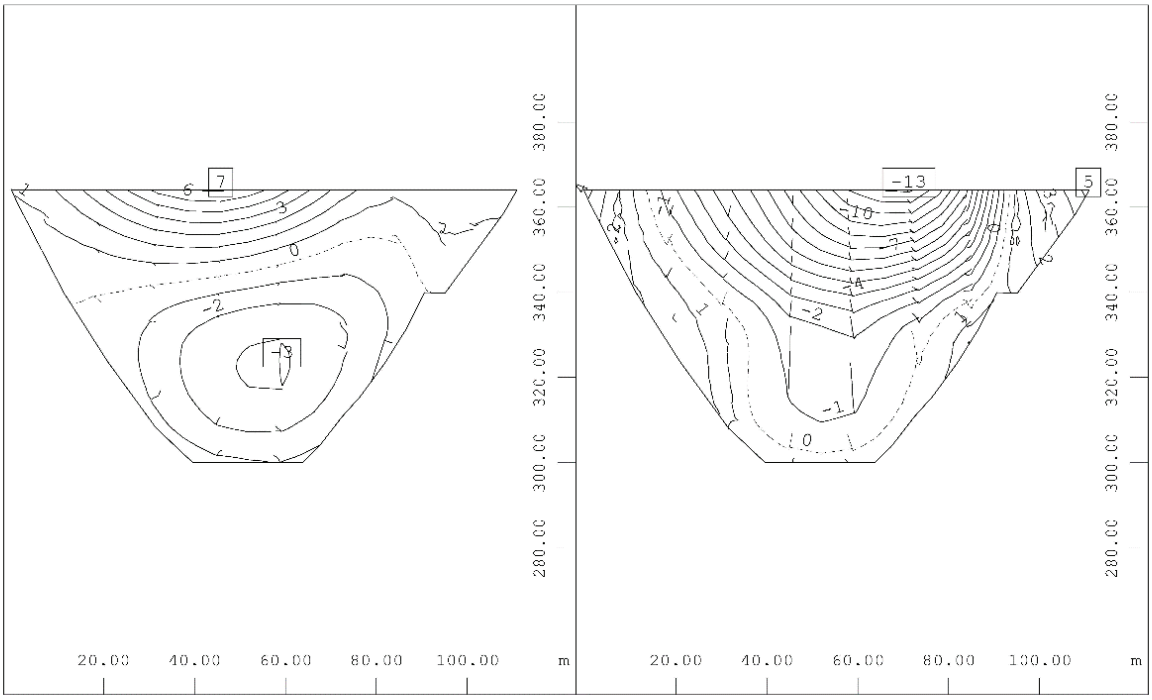


Fig. 11. Horizontal displacements in longitudinal section in dam axis [mm]. a) Isolines of displacements after construction; b) Isolines of displacements at full reservoir.

Isolines of principal stress σ_3 after dam construction (Fig. 12) reach maximal value of 6.40 MPa, at the upstream toe of the dam. Principal stresses σ_3 at full reservoir (Fig. 13) have heterogeneous distribution, with maximal value of 7.30 MPa, in the upstream toe of the dam also.

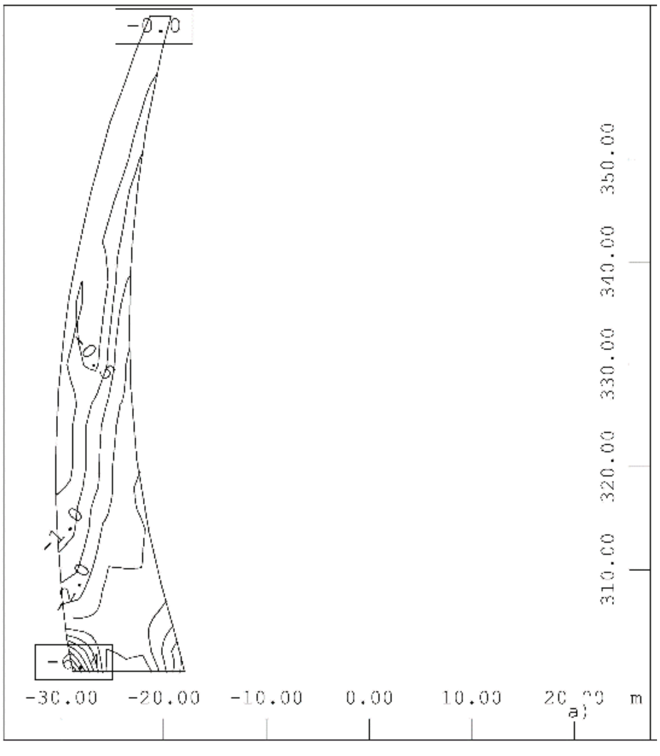


Fig. 12 Isolines of principal stresses σ_3 in the central dam cantilever after construction.

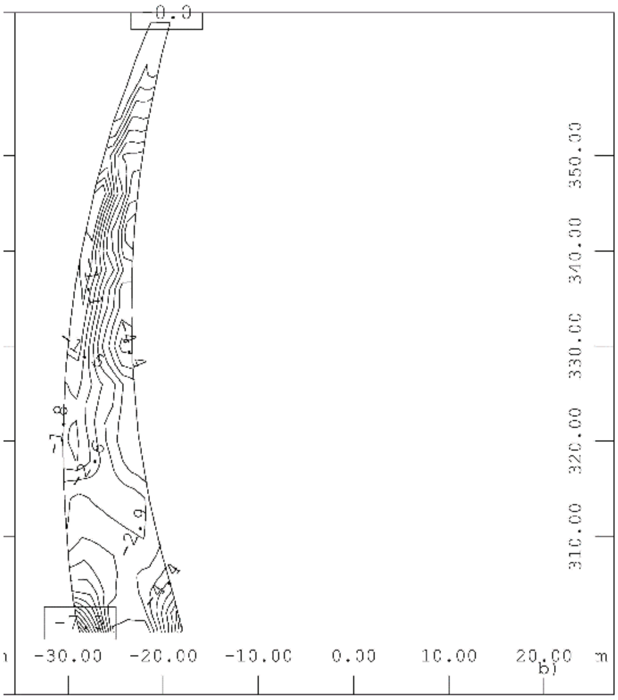


Fig. 13. Isolines of principal stresses σ_3 in the central dam cantilever at full reservoir.

5. CONCLUSIONS

The prediction of the behaviour of the concrete arch dams during construction and at full reservoir is essential for assessment of the dam stability and providing limit data for displacements and stresses in the dam for the engineers – designers of these structures.

The St. Petka dam was analyzed with application of the program package SOFiSTiK, based on the finite element method. The loading was applied in accordance with realistic loading states of the dam – after construction and at full reservoir. Beside the primary loads of the dam, grouting load was also included in the model.

The obtained data from the numerical analysis should be compared with monitoring data of the dam that would provide additional knowledge for the dam behavior and will reflect on the accuracy of the numerical model and the monitoring devices. Any disagreement between this data would indicate on improper dam behavior that would require investigation and analysis of the eventual problem and taking measures.

The obtained values and distribution of the displacements for the analyzed stages of the dam are according to the expected and can not endanger the dam safety and service.

Maximum allowable compressible stresses for usual loading state of the dam (self weight, temperature and hydrostatic pressure) should be less then 1/3 of specified compressible strength. The calculated stresses are less then the allowable values, thus this criteria is met.

REFERENCES

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

Barton N., Kjaernsli B.: Shear strength of rockfill. Journal of the geotechnical engineering division, Vol.107, N0 GT7, July 1981

Eurocode 2: Design of concrete structures, European Committee for Standardization, 1992

Finite element methods in design and analysis of dams, ICOLD Bulletin 30a, 1987.

Goodman R., "Methods of Geological Engineering in Discontinuous Rocks" West Publishing Company, New York, 1976
Hoek E., Practical Rock Engineering, 2000
Hudson J., Harrison J."Engineering Rock Mechanics - An Introduction to the Principles", Pergamon Press, Elsevier Science Ltd., 1997
ICOLD Bulletin no. 145, The physical properties of hardened conventional concrete in dams, January, 2009
ICOLD Bulletin 30a, Finite element methods in design and analysis of dams, 1987.
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
Novak P., Moffat A. I. B., Nalluri C., Narayanan R. "Hydraulic structures", Taylor & Francis Group, London, 2007.
Synthesis Elaborate on performed investigation and testing for concrete arch dam at dam site "St. Petka" on river Treska, Civil Engineering Institute "Macedonia", 2004.
Tanchev L., (2014) Dams and appurtenant hydraulic structures, Second edition, A.A. Balkema Publ., CRC press, Taylor & Francis Group plc, London, UK
Tanchev Lj. (1989) Static analysis of rockfill dams. Studentski zbor, Skopje (in Macedonian)
Jovanovski M., Gapkovski N., Anđelković Vl., Petrović Lj.: 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, 2002, pp.78-86 (in Macedonian)