ДГКМ
ДРУШТВО НА ГРАДЕЖНИТЕ КОНСТРУКТОРИ НА МАКЕДОНИЈА

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МОДЕЛИРАЊЕ НА КОНСТРУКТИВНИТЕ ФУГИ КАЈ БРАНА СВ. ПЕТКА

РЕЗИМЕ

Контактните зони на материјали со различни деформабилни параметри при спроведување на нумерички анализи се симулира со контактни елементи. Во рефератот е даден преглед на досега применувани вакви елементи, со подетален осврт на контактниот елемент од типот “spring”, применет за симулација на контактните зони карпестата основа/брана и бетонски блокови/инјекциона смеса/бетонски блокови во телото на браната (фугите на браната), со случајот на лачната брана Св. Петка, со височина од 64.0 m, изградена на река Треска, пуштена во употреба во 2012 година.

Клучни зборови: лачна брана, контактен елемент, крутост, поместувања

Stevcho MITOVSKI[1]

STRUCTURAL JOINTS MODELLING AT DAM ST. PETKA

SUMMARY

The contact zones of materials with different deformable properties at carrying out of numerical analysis is simulated by interface (contact) elements. The paper gives short overview of the up to now applied interface elements and in more detail addresses the application of interface element of type “spring”, used for simulation of the contact zone rock foundation/dam and concrete blocks/grouting zone/concrete blocks (joints), in the case of St. Petka dam, a 64 m high arch dam on River Treska in Macedonia, commissioned in 2012.

Keywords: arch dam, interface elements, stiffness, 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 potential of the rivers in Republic of Macedonia is utilized to hardly 30%, and yet there is a permanent shortage of water for various purposes. Due to this fact, as well as due to strongly expressed uneven distribution of water, it is indispensably necessary to construct new dams with reservoirs. 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.

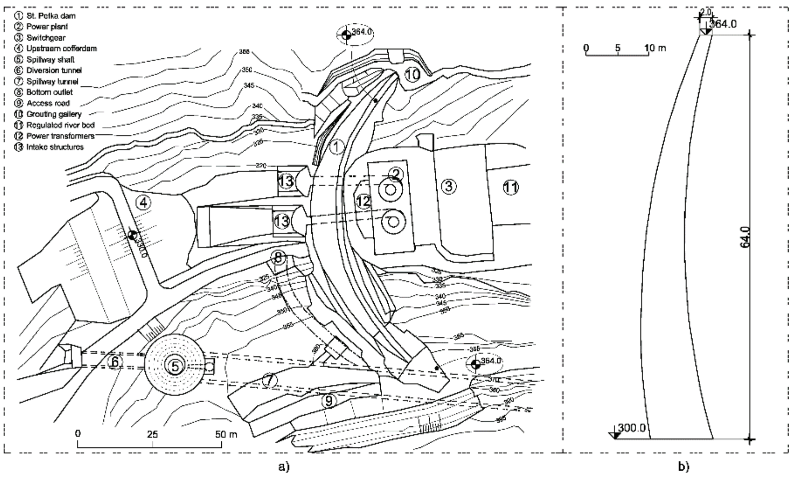


Fig. 1. St. Petka dam. a) layout and b) typical cross-section

3. DAM MODELLING

The static analysis of St. Petka dams dam is conveyed with application of the code SOFiSTiK, based on the finite element method. 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

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). An engineering-geological model of the terrain per parameter of deformability and shear strength is used, prepared on basis of the models per parameter of elastic waves velocity, obtained by geophysical methods before construction stage, as well and the model and section per parameter of weathering. 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.

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

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

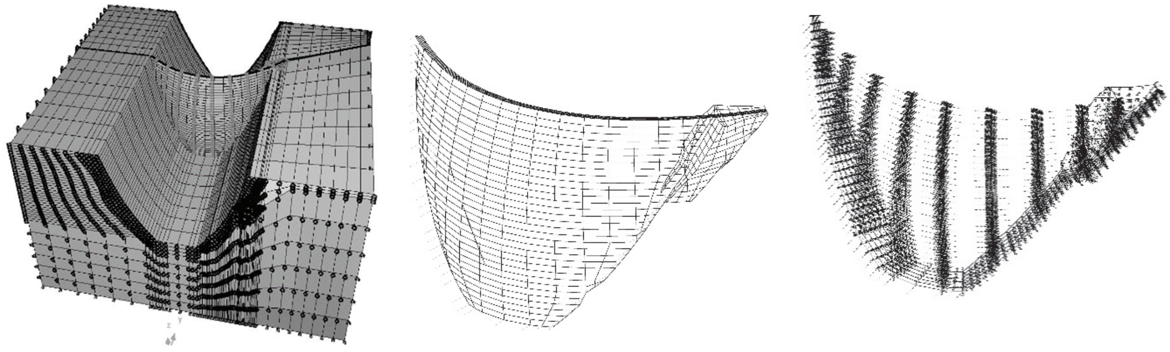
The constitutive law for concrete is adopted according to EC 2, concrete type 30 [ICOLD, 2009; Eurocode 2, 1992]. The basic concrete parameters are given in Table 2.

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	[$\times10^{-6}$ on °C]	1.0

Table 2. Concrete parameters

3.2 Model geometry

The spatial analysis of St. Petka dam is performed in stages. 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).

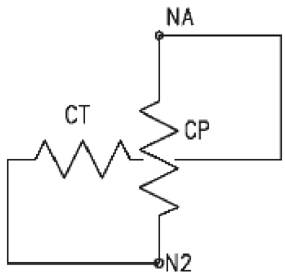


Fig. 3. Interface element of type "spring"

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.

By applying of conventional finite elements it is not possible to simulate specified contact zone behaviour due to the compatibility of the element edges. Therefore interface elements are applied for simulation of contact zones behaviour. In rock mechanics is assumed that the rock material behaves by Mohr-Coulomb law, where as the shear strength is expressed by cohesion and angle of internal friction. The contact zones behaviour is displayed on Fig. 4 [Goodmaн, 1976; Tancev, 1989; Hudson and Harrison, 1997; Hoek, 2000]. Under action of the tangential stress an relative displacement occurs of bodies I and II for value of Δs. Two cases are possible: a) No occurrence of sliding in the contact apropos the zone of elastic deformations, described by the equation:

$$\tau<\tau_f=c+\sigma_n \tan\delta \quad (1)$$

where as: τ_f – shear strength of the contact; c – contact cohesion; σ_n – normal stress; δ – sliding angle.

Curve part marked with (1) on Fig. 4-b are displayed occurred elastic shear deformations. The interface element parameters are determined by normal stiffness C_p and tangential stiffness C_t. In case of stress state by occurrence of elastic deformations in the contact the normal stiffness has the role of elasticity modulus, while the tangential stiffness is equaled with the distortion modulus G, according to the following equation:

$$\sigma_n = C_p \cdot \Delta n; \tau = C_t \cdot \Delta s \quad (2)$$

where as Δn and Δs are average normal and tangential relative displacement along the contact. According to the theory, the contact behaviour is displayed on Fig 4-b, c.

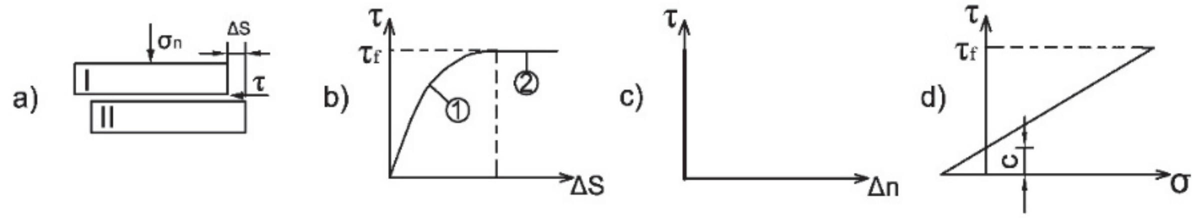


Fig. 4 Behaviour of contact zones

b) Sliding occurs in the contact. Such case appears when the tangential stress will reach the shear strength of the contact apropos when $\tau \geq c_a + \sigma_p \tan\delta$. The stiffness in such case has value of zero and the adjacent layers displace along the contact length independently one from another. In case of tension in the contact, it loses both stiffness.

The interface element can be applied by input of axial, lateral and rotational constant, prestressing, sliding and non-linear effects (failure load for compression or tension, creep, friction coefficient, cohesion, strain etc).

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] Some detail analyses of shear strength parameters in designing of fill dams can be found in Barton and Kjaernsli [Barton and Kjaernsli, 1981], while summarized overview is given by Tanchev [Tanchev, 1989; Tanchev 2014].

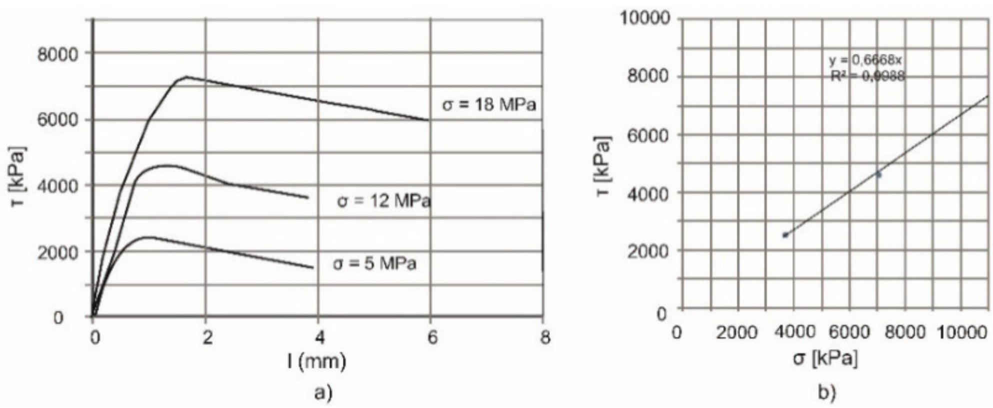


Fig. 5. Output results for testing sample 378 I/A, contact concrete-grout mixture at constant normal load of 6, 12 and 18 MPa. a) Dependence shear load-displacement, b) dependence shear/normal load $\tau = \sigma(s)$

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 [Mitovski, 2015]. 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. The output results from one sample from the testing are displayed on Fig. 5. The adopted values are Cp=16×106 kN/m3 and Ct=1×106 kN/m3 for contact dam-foundation and Ct=5×106 kN/m3 for contact concrete/grout mixture.

3.3 Dam loading

The analysed 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 grouting load, applied on the joint surfaces between two concrete blocks, was also simulated in the analysis (Fig. 8). 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 (Fig. 9).

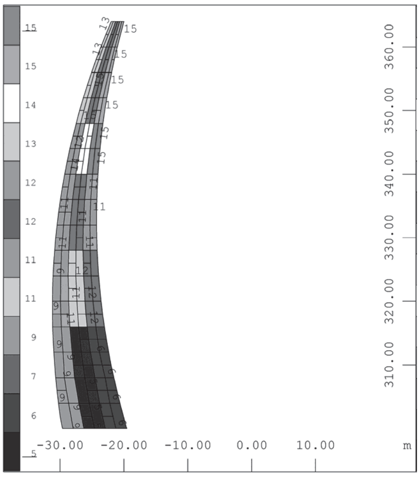


Fig. 6. Temperature loading, applied on the dam

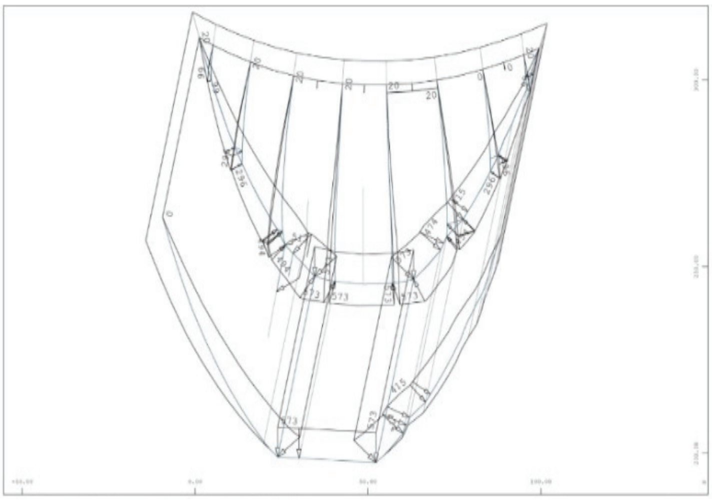


Fig. 7. Water loading, applied on the dam

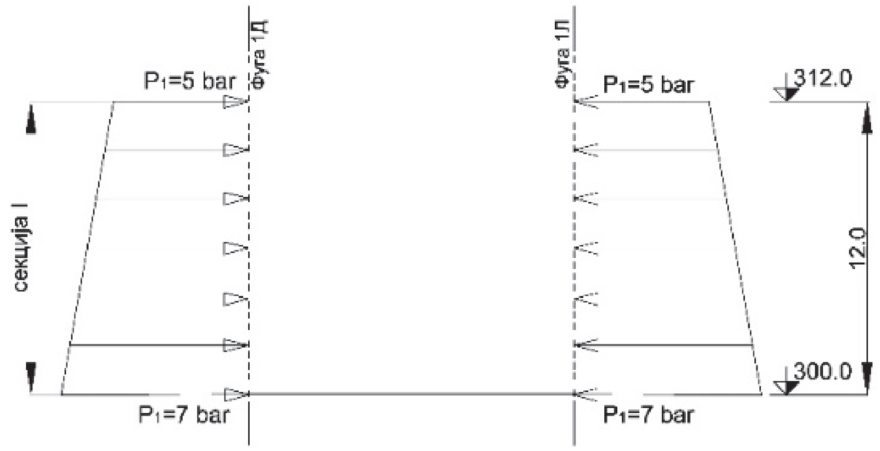


Fig. 8. Grouting load (pressure) for section I in joints 1L (left) and 1R (right)

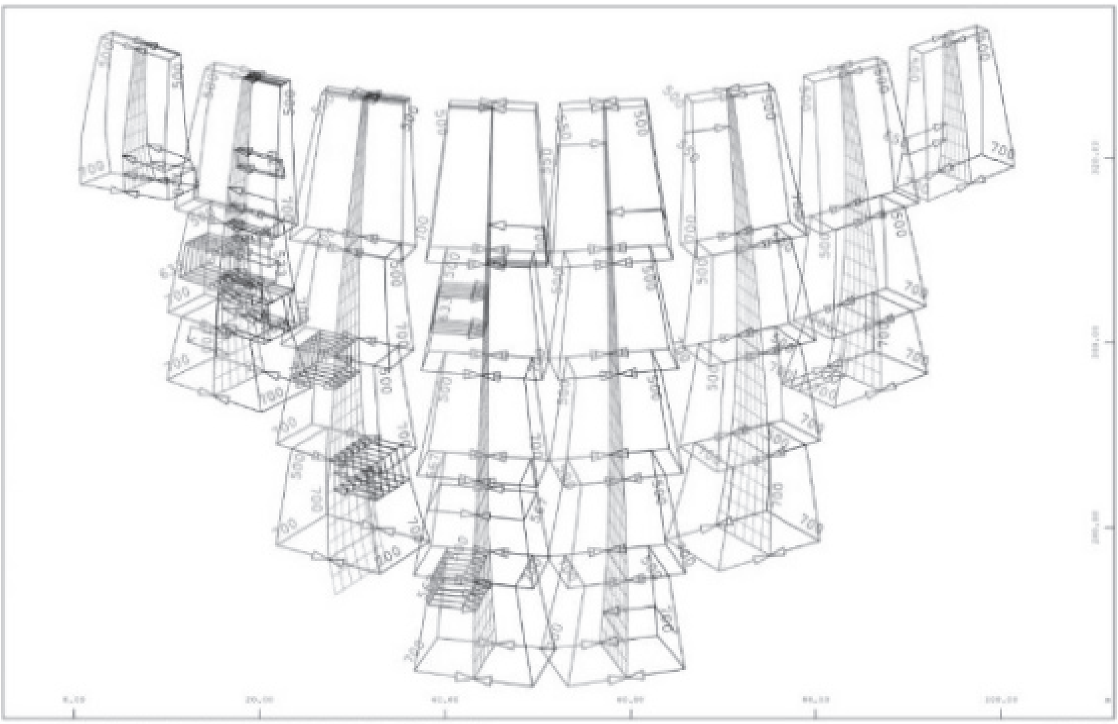


Fig. 9. Grouting load, applied along surface of both sides of the dam joint

4. OUTPUT RESULTS

The displacements of the interface elements are analysed. On Fig. 10 are displayed spring displacements in vector form upon grouting process, while on Fig. 11 are displayed spring displacements for full reservoir. Displacements are increased in case of full reservoir state due to the temperature increase. Maximal displacements in the joint upon grouting process is 0.89 mm. At full reservoir state the maximal displacement in the joints is 1 mm. In the dam are installed measuring devices – joint meters, for observation of the joint behaviour (Fig. 12). On Fig. 12 are displayed calculated and measured values for joint displacements upon grouting stage. It can be concluded that there is good matching of the measured and calculated values regarding the direction and size of the displacements for the state upon grouting stage.

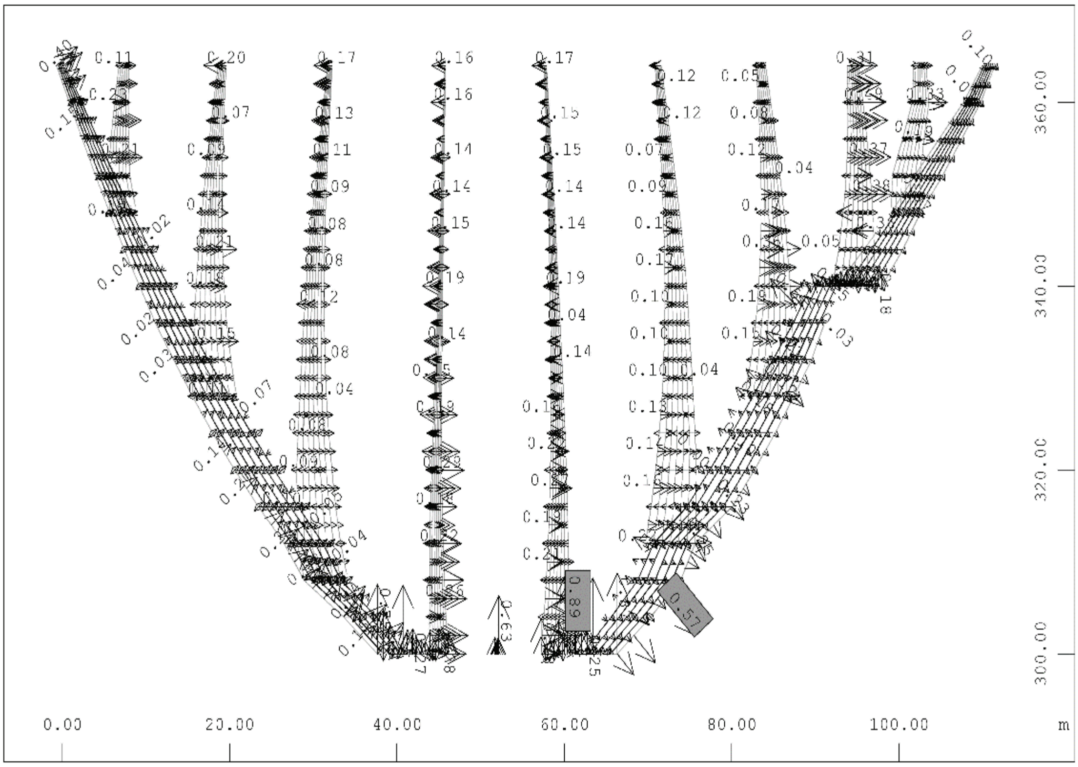


Fig. 11. Joint displacements and contact dam-foundation displacements upon grouting state, [mm]

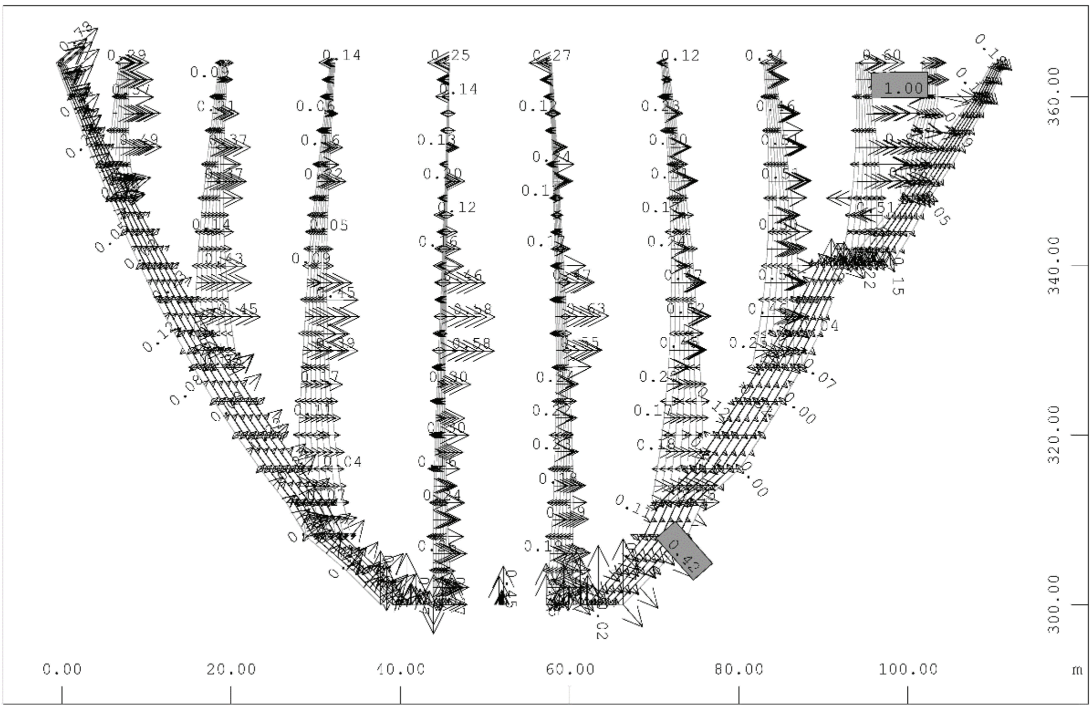


Fig. 10. Joint displacements and contact dam-foundation displacements at full reservoir state, [mm]

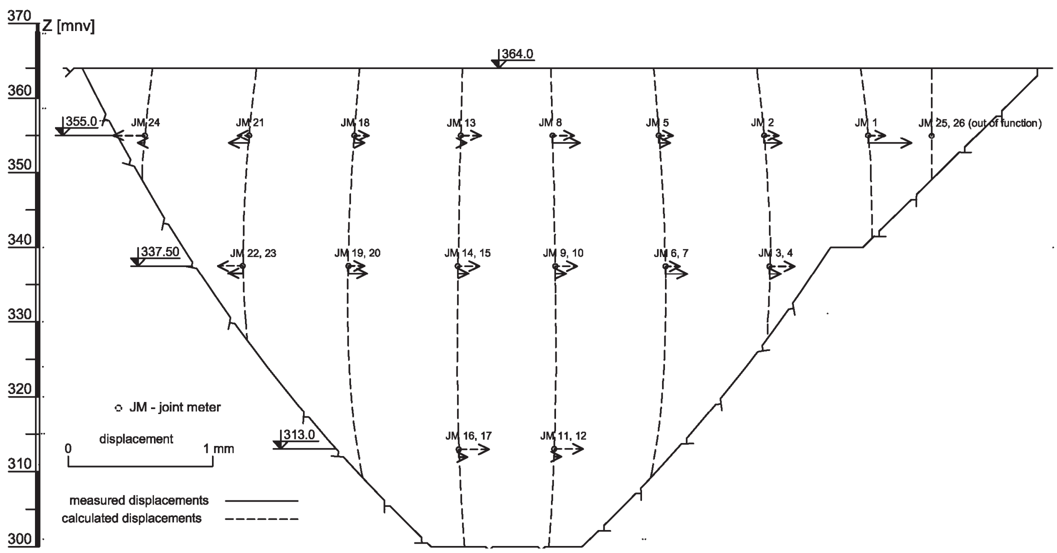


Fig. 12. Comparation of calculated and measured joint displacements upon grouting stage

6. CONCLUSIONS

The prediction of the behaviour of the concrete arch dams during construction and at full reservoir is essential for 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 lead to additional information on the dam behavior and will reflect on the accuracy of the numerical model and the monitoring devices. This is the essence of the calibration process, which is continuous. 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 joints behaviour within the dam body was simulated by interface elements of type spring. The input parameters were determined by laboratorial testing treating the shear strength along discontinuities in this case by application of Hooke's apparatus.

The joints behaviour is analyzed by displacements values and direction, obtained within the expected ones for such structure and loading cases. The comparation of calculated and measured displacements shows good matching thus leading to conclusion that the adopted input values for the normal and tangential stiffness are well chosen.

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