ЗМКГБ
MACOLD

СТАТИЧКА АНАЛИЗА НА БРАНА МАТКА

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Резиме

Евалуацијата на стабилноста на браните е исклучително комплексна задача, која мора да се разгледува од повеќе аспекти (геотехнички аспект, конструктивно однесување на браната при градба, прво полнење и експлоатација, како и долготрајното однесување на браната и вградените материјали) и со анализа на различни товарни состојби. Критериумите за конструктивната стабилност мора да се исполнат со цел да се обезбеди стабилноста на браната.

Примената на методот на конечни елементи доведе до значајни промени при анализата на стабилноста на лачните брани овозможувајќи креирање нумерички модели за спроведување нелинеарна просторна анализа за различни товарни состојби (влијание од сопствена тежина, товар од вода, температурен ефект). Исто така, овозможена е примена на специјални конструктивни елементи за внесување на комплексната геометрија на лачните брани (контакт меѓу лаци) и за пореално симулирање на нивното однесување.

Во предметниов труд е даден приказ на евалуацијата на статичката стабилност на лачната брана „Матка“ (прва брана изградена во Република Македонија), со висина од 29,5 m, изградена на реката Треска во близина на Скопје, пуштена во употреба во 1938 година.

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

STATIC ANALYSIS OF MATKA DAM

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Summary

The evaluation of the dam stability is complex task, that must consider number of aspects (geotechnical conditions, structural behaviour during construction, first filling and service period apropos long term behaviour of the dam and embedded materials) and to take various loading scenarios into account. The criteria for structural stability must be met in order to provide dam safety.

The application of the finite element method introduced significant advancement for analysis of the arch dam stability thus enabling creation of numerical models for conveying of non-linear spatial analysis for various loading cases (self-weight effect, water load, temperature influence). Also, an application of special structural elements is enabled for input of the complex geometry of the arch dams (arch contacts) and thus more realistic simulation of their behaviour.

In the paper is displayed evaluation of the static stability of arch dam Matka (first constructed dam in R Macedonia), with height of 29.5 m, built on river Treska in near by of Skopje, commenced in 1938..

Key words: arch dam, stability, stresses, displacements

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1. INTRODUCTION

The analysis of the dam stability within design stage includes knowing and assessment of the stress-deformation state in the dam body as well and behaviour (interaction) of the contact zone of the dam and the foundation. However, there are cases in practices where is required updated analysis of the stability of existing dams (by application of advanced methods), constructed in the past, designed by applying of the classical methods for analysis.

In the paper is conveyed analysis of the stress-deformation state of the arch dam Matka (first built dam in R Macedonia in 1938) at action of static loading apropos an re-assesment of the stress-deformation state has been executed by application of advanced calculation aproach, based on the finite element method. The analysis are executed by application of the code SOFiSTiK. The aim of the analysis is evaluation of the behaviour of arch dam Matka, subjected to static loads, that will provide insight in the dam state and it will be monitored through out comparation of the measured values within the technical monitoring of the dam and calculated values with the numerical analysis.

2. MATKA DAM

Matka dam is constructed on river Treska at aproximatelly 14 km south-west of Skopje. It is regular service from 1938, and it is owned (along with the HPP) by EVN Macedonia AD Skopje, managed by EVN Macedonia power plants. The arch dam Matka, with total embedded concrete of 3,000 m^3 and 150 t reinforcement, creates reservoir of 3,55 milions m^3 [Tanchev, 2014; YUCOLD, 1970]. It is the oldest dam in R Macedonia, and in same time the only dam constructed before the World War II (1935-1938). The sound, hard and firm rock in the foundation and also the rather solid and reliable construction of the dam, have enabled it to be in service up to the present day, (Fig. 1), surviving without damage the strong Skopje earthquake of 26th July 1963. It dams the gorge of the River Treska, composed of sound and impermeable rock of shales and marbles, with high watertightness properties. The dam site is a narrow one, with a V-form, thus favorable for concrete arch dam. The dam structure is chosen accordingly to the dam site. by constant central angle (148º to 155º) and diferent radii of curvature (19 to 25 m). The dam consists of 10 arches (Fig. 2), each being 3 m high, except the uppermost, with height of 2.5 m. The thickness of the lowermost arch is 1.6 m, while that of the uppermost is 1.0 m. Below the lowermost arch, in the foundation, there has been constructed a massive concrete foundation (Fig. 2).

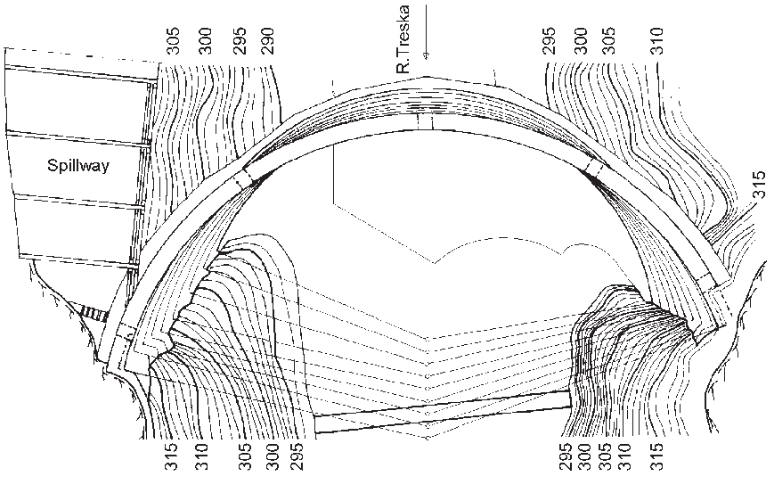

Figure 1. Matka dam layout, H=29.5 m.

The concrete, of which arches have been carried out, contained 325 kg of high-strength low-setting cement. It is interesting to note the careful treatment of the joints between arches. Between individual arches there have been carried out horizontal joints with several coatings of bitumen and a thin zinc sheet. For achieving water-impermeability, there has been built in a copper sheet of 0.5 mm, which has penetrated some 25 cm into the neighbouring arches. As an additional measure for improving the water-impermeability of the dam, the upstream face has been coated with an insulation mixture on the basis of bituminous emulsion, upon which there has then been carried out a protective layer of cement mortar over a wire mesh, fixed into the dam's body. On the right-hand side there have been constructed a spillway structure and an outlet works, while on the left-hand side there has been constructed the intake structure to the power house of the near-dam electric power plant. Miladin Pečinar, professor at the University of Belgrade, later on an academician, designed it. Due to some damages, in 1990 an repairment of the insulation has been undertaken, from elevation 311.5 to 317.5 masl.

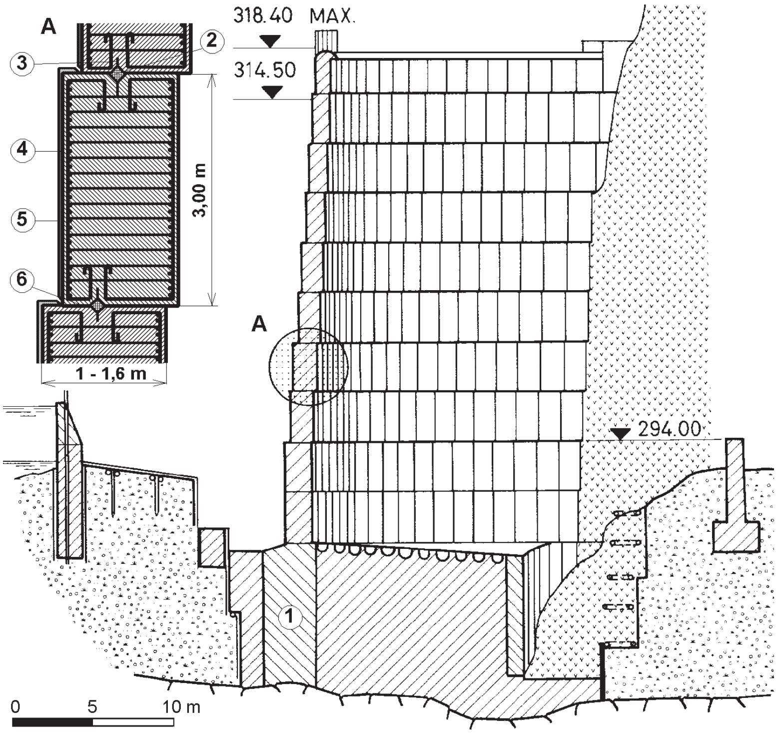

Figure 2. Cross-section of the Matka dam, along with a detail of the joint between arches (A). (1) Concrete foundation; (2) clay filling; (3) waterstop of copper sheet; (4) insulation of bituminous coating; (5) protective layer of reinforced mortar; (6) insulation coating and a thin zinc sheet.

3. NUMERICAL MODEL

The static analysis of dam Matka is executed by application of the code SOFiSTiK, produced in Germany. The SOFiSTiK code is based on the finite element method and offers possibilities for complex modelling of the structures from geometrical aspect as well and simulation of their behaviour in case of various loading stages in chronology timeline. The code can also include and simulation of some specific phenomenas in case of dam analysis (stage construction and reservoir filling, behaviour of contact or interface zones etc.).

In order to perform the numerical analysis following steps are undertaken: (1) choice of material parameters – constitutional laws, (2) discretization of the dam with finite elements and creation of the numerical model and (3) simulation of the typical dam loading stages.

3.1 Constitutional law for concrete

The constitutional law for the concrete in the dam body and the dam foundation is adopted according to EC 2, concrete grade CG 30 [Eurocode, 1992; ICOLD, 2009], directly applied form the material library of the code SOFiSTiK. In Tab. 1 are specified main input data for concrete.

Table 1. Input date for concrete, according to EC 2.

Parameter	Mark	Value	Unit
Density	γ	24.00	kg/m3
Temperature coefficient	α	1.00*10-5	
Elastic modulus	E	31476	MPa
Poisson ratio	ν	0.20	

The stress-strain dependence is displayed on Fig. 3, by specifying the marksings according to the equations 3-1 and 3-2:

$$\sigma_c = f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \quad за \quad 0 \le \varepsilon_c \le \varepsilon_{c2} \qquad (3\text{-}1)$$

$$\sigma_c = f_{cd} \quad за \quad \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \qquad (3\text{-}2)$$

where as:

σ_c – compression stress of concrete

f_{cd} – design value of the compression strength of concrete

n – exponent

ε_{C2} – strain reached at maximal strength.

ε_{Cu2} – ultimative strain.

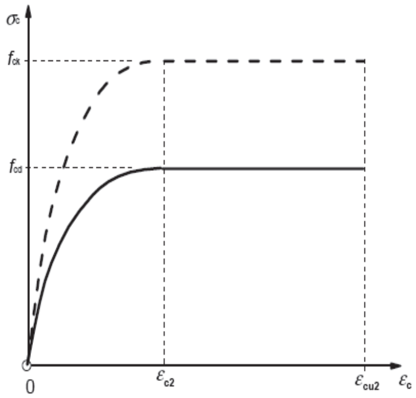

Figure 3. Constitutive law for concrete [Eurocode 2, 1992].

3.3. Discretization of the dam with finite elements

Within the discretization process it is important to apply the proper element type. In case of complex structure and behaviour (thin arch dam, founded on rock), where as calculated values for the stresses and displacements are of paramount importance for assessment of the dam stability state, is required to be applied powerful and reliable finite elements. The here applied elements are quadrilateral shell element by 4 or 8 nodes, and interface of type “kine”. The “kine” connections are applied in order to simulate the interaction of the independent arches, and in fact they are kinematic constraints. By such applied interfaces are generated common displacements of the arches at their contact surfaces. In the model are applied two types of kinematic constraints: (1) links that transfer forces and moments (Pk), and provide full continuity of the deformations at the arch contact and (2) links that transfer only vertical forces (Pz), and provide continuity of the vertical deformations. In the model within the intermediate zone of the dam, where as the arches are laid one over another almost per the full thickness are applied kinematic constraints of type Pk, while in the zones towards the left and the right bank of the dam are applied kinematic constraints of type Pz.

The numerical model consists of independent arches, modeled by shell elements, connected by kinematic constraints connections. The kinematic constraints provide simulation of the arch eccentricity. The non-deformable boundary condition is placed in the fixed zone of the individual arches and in the concrete foundation (Fig. 4). The dam is discretized by capturing of the zone within one arch and the fixation zone in separate groups of one material (concrete), Fig. 5 and Fig. 6. The model is adapted to simulate the dam behaviour as system of independent arches, thus including ten arches by various height (h=2.5 – 3.0 m), as well and the concrete foundation in the river bed, by height of 8.0 m.

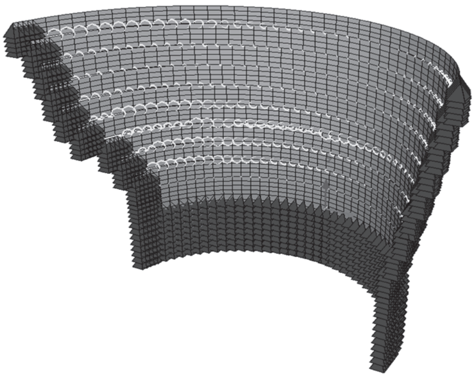

Figure 4. View from downstream side of the model, with boundary conditions and kinematic constraints.

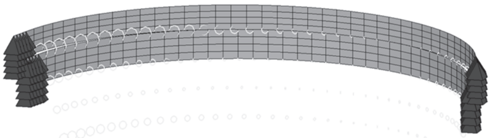

Figure 5. Zones with shell elements, connected with interface links and boundary conditions of the successive arches.

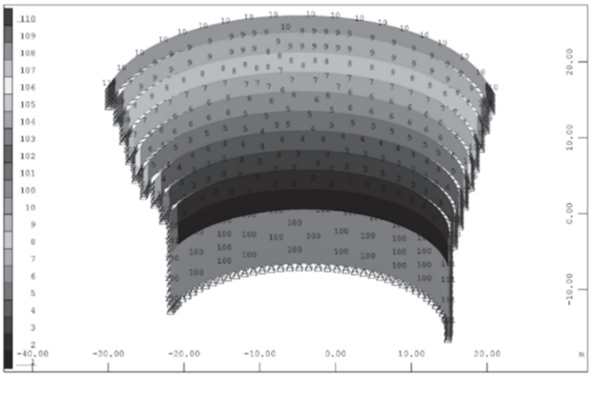

Figure 6. Dam division in groups.

3.5 Dam loading

The dam loading includes simulation of stage at full reservoir in summer period and at full reservoir in winter period. On Fig. 7 is displayed the temperature distribution in the, adopted according to temperature values and distribution from previous executed analysis for such dams [Mitovski, 2015; Kokalanov et all, 2007, ICOLD, 2009], due to the fact that we do not poses realistic measured values for the temperature in the dam body. The temperature distribution varies from the upstream to downstream face apropos an increment for temperature variation is applied in dependence of the time period (winter or summer). Hereby the critical scenarios are winter period, where as temperature varies in interval from -3 to -2ºC, by decreasing in the lower zones of the dam and variyng values in usptream-downstream and summer period, where as temperatures are in range 6-14ºC, by decreasing in the lower zones and variyng values in usptream-downstream. The hydrostatic pressure is applied along the upstream face of the dam as hydraulic loading.

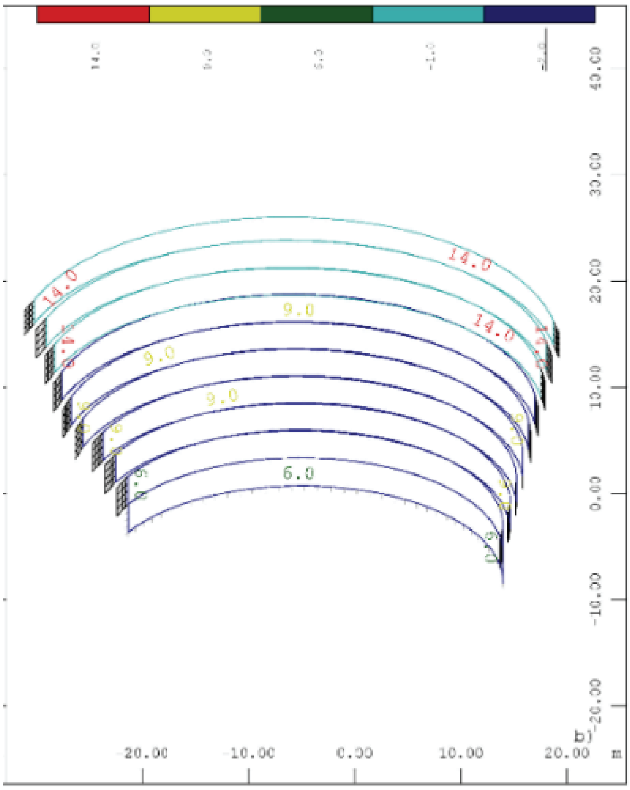

Figure 7. Temperature loading of the dam for summer period.

4. OUTPUT RESULTS

4.1 Displacements

On Fig. 8-a is displayed the displacements vector in XYZ-direction at full reservoir in winter period. Maximal displacements occurs in the intermediate left zone of the dam, approximately at 50% of the dam height, directed towards the dam banks and value of 3.9 mm. On Fig. 8-b are displayed horizontal displacements in X-direction (towards banks) at full reservoir in winter period. The generated displacements are approximately symmetrically distributed regarding the central cantilever, by maximal displacements in the intermediate left and right zone of the dam towards the banks, value of 1.5 mm in the right zone and 2.4 mm in the left zone. The “zero” line of displacements is approximately in the middle zone.

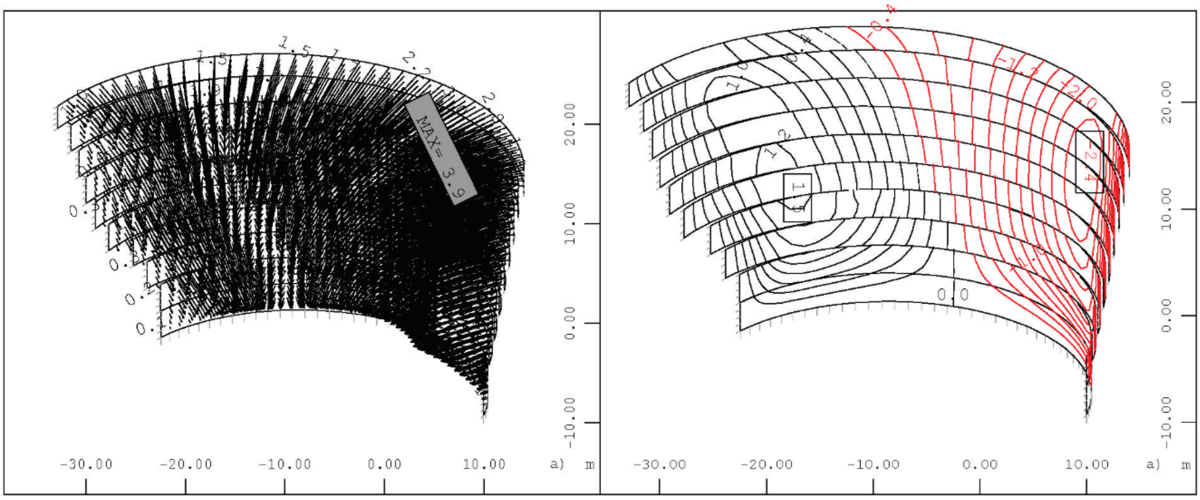

Figure 8. Displacements in the dam's body at full reservoir and winter period, [mm]. a) Displacements vector in XYZ-direction; b) Isolines of horizontal (banks direction) displacements.

On Fig. 9-a are displayed the horizontal displacements (Y-direction) at full reservoir in winter period. It can be seen that the displacements are dominant in downstream direction, and maximal displacement occurs in the left zone, approximately at 50% of the dam height, value of 3.2 mm. On Fig. 9-b are displayed vertical displacements at full reservoir in winter period. The generated displacements are dominant in gravity direction (settlements), maximal value of 1.4 mm in the middle part of the dam crest.

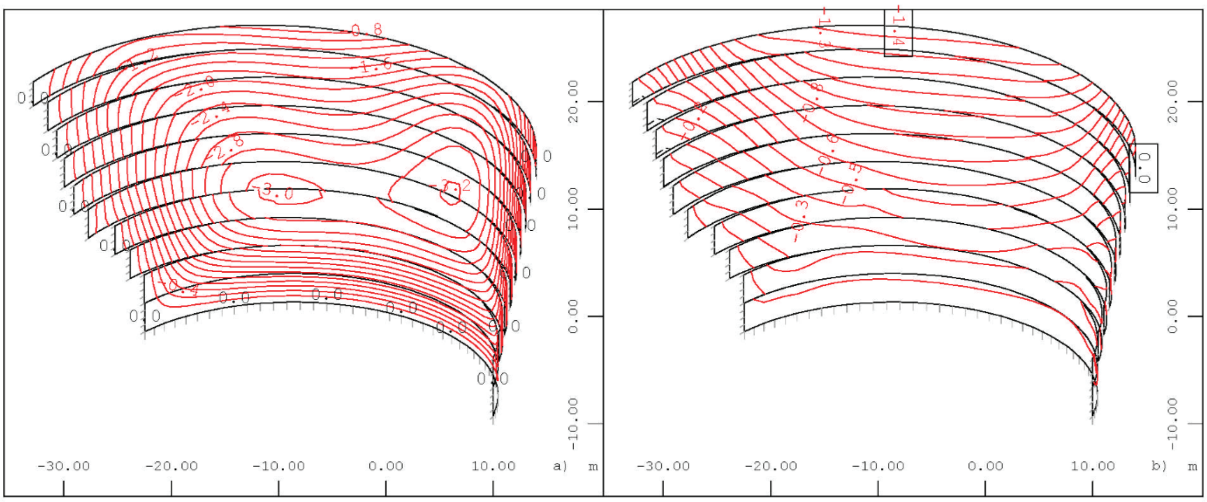

Figure 9. Displacements in the dam's body at full reservoir and winter period, [mm]. a) Isolines of horizontal (Y-direction, "-" denotes displacements towards downstream) displacement; b) Isolines of vertical displacements ("-" denotes setlements).

On Fig. 10-a is displayed the displacements vector in XYZ-direction at full reservoir in summer period. Maximal cumulative displacements occurs in the dam crest in resulting XY direction, by maximal displacements in near by of the right bank of 8.3 mm. On Fig. 10-b are displayed isolines of horizontal displacements at full reservoir in summer period. From the displacements distribution it can be noticed that maximal displacements occurs in the dam crest, towards right (6.9 mm) and left bank (6.1 mm). In the intermediate and lowermost zone the displacements are practically equal to zero (dashed line in these zones denotes "zero" line of the displacements).

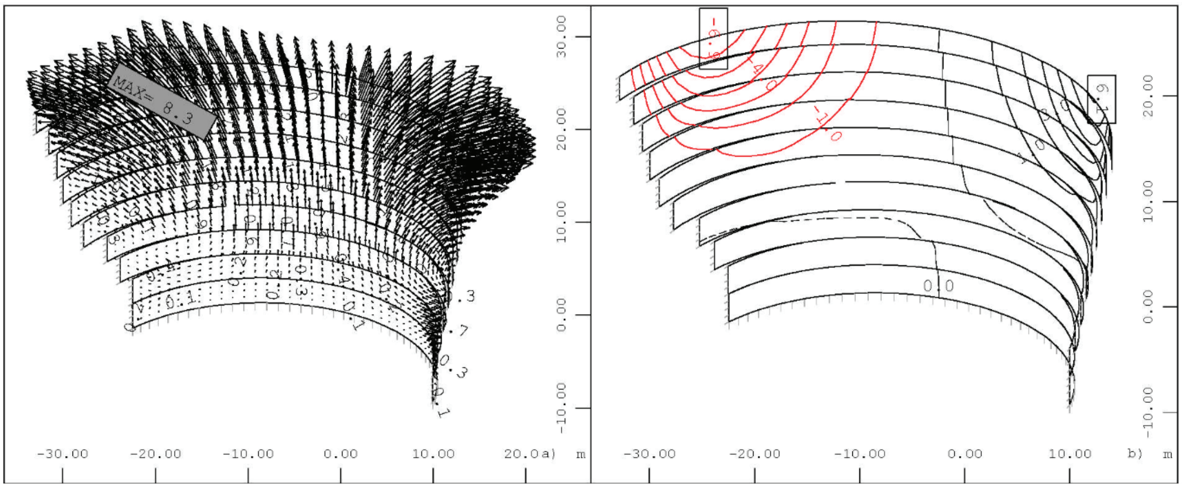

Figure 10. Displacements in the dam's body at full reservoir and sumer period, [mm]. a) Displacements vector in XYZ-direction; b) Isolines of horizontal (banks direction) displacements.

On Fig. 11-a are displayed the isolines f horizontal displacements (Y-direction) at full reservoir in summer period. It can be seen that the displacements are dominant in upstream direction, maximal occurred displacement of 4.8 mm in the dam crest towards the right bank. On Fig. 11-b are displayed vertical displacements at full reservoir in summer period. The generated displacements are dominant in opposite of gravity direction (raising), maximal value of 2.9 mm in the middle part of the dam crest, mainly affected by the temperature increasment.

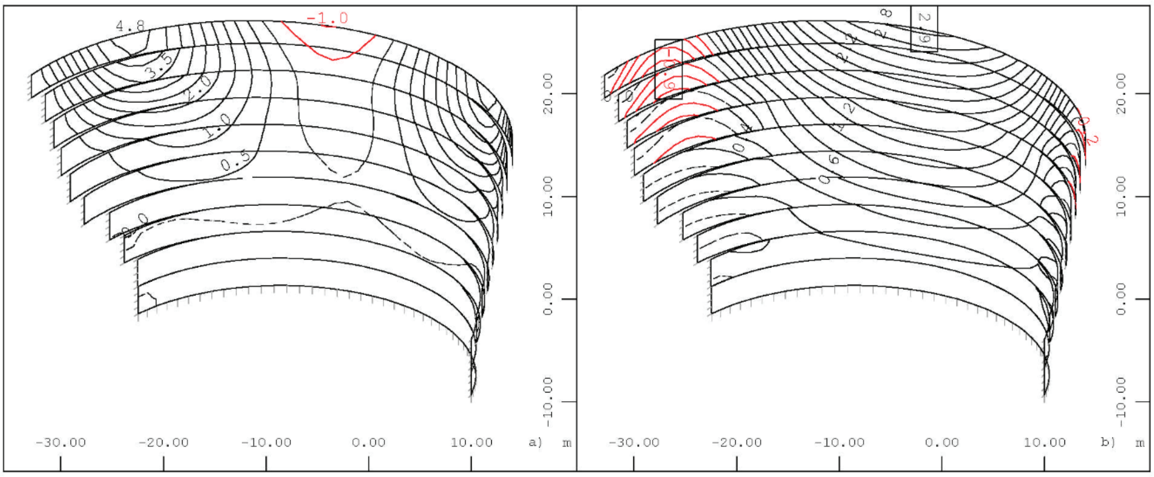

Figure 11. Displacements in the dam's body at full reservoir and summer period, [mm]. a) Isolines of horizontal (Y-direction, "-" denotes displacements towards downstream) displacement; b) Isolines of vertical displacements ("-" denotes setlements)..

4.2 Stresses

Principal (arch) stresses σ_1 in the dam body on upstream face, at full reservoir in summer period are displayed on Fig. 12-a. In the intermediate zone all to the ¾ of the dam height the stresses are dominantly on compression, reaching maximal value in the left lower zone of the dam of 3.0 MPa. At the dam crest, in the left and right bank there is occurrence of local tension zones, value of 2.30 MPa. Principal (cantilever) stresses σ_3 on the upstream face of the dam at full reservoir in summer period (Fig. 12-b) are dominantly compressive, by occurrence of maximal value of 9.60 MPa in the dam crest in near by of the left bank.

Principal (arch) stresses σ_1 in the dam body on downstream face, at full reservoir in summer period are displayed on Fig. 12-c. In general they have similar distribution as of the upstream face, however there is significant increase of the tension stresses in the dam crest on the right bank, value of 5.10 MPa, due to the temperature influence, that is prevailing in case of thin arch dams. In this case there is exceedance of the maximal allowable tension stresses σ_{doz}=2.5 MPa, occurred in local zone in the right zone below the dam crest, thus indicating of a potential critical zone.The compression stresses are lowered compared to the upstream face, by maximal value of 1.60 MPa, occurred in the lower right zone of the dam. The isolines of the principal stresses σ_3 on the downstream face at full reservoir in summer period (Fig. 12-d) are dominantly compressive thus reaching maximal value in the dam crest towards the left bank, value of 4.10 MPa, by similar distribution as the same stresses on the upstream face (Fig. 12-b).

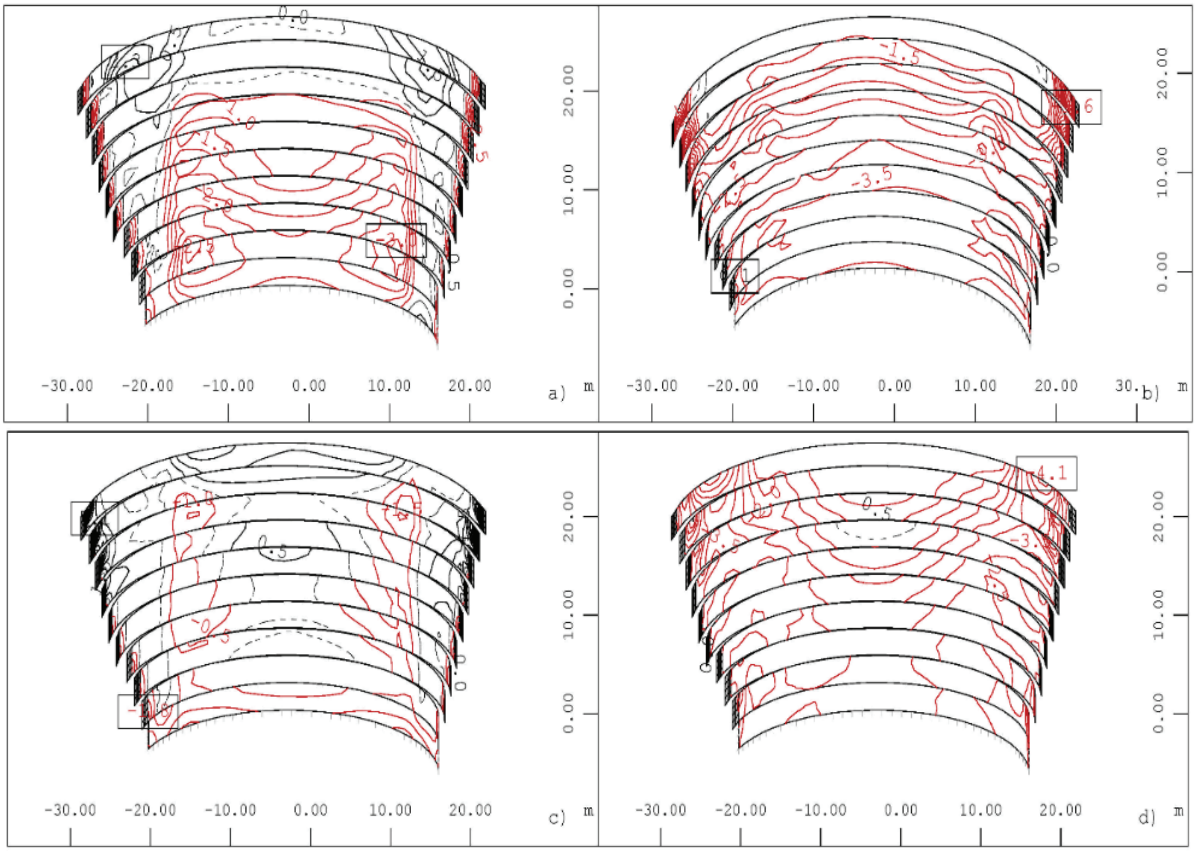

Figure 15. Principal stressed in the dam at full reservoir in summer period [MPa]. a) Isolines of principal stresses σ_1 on upstream face; b) Isolines of principal stresses σ_3 on upstream face; c) Isolines of principal stresses σ_1 on downstream face; d) Isolines of principal stresses σ_3 on downstream face.

5. CONCLUSION

By analyzing the output results from the hereby executed numerical analysis of Matka dam, by numerical model composed of shell elements and kinematic constraints, following conclusions can be drawn out:

(a) The maximal resultant displacements in the dam body (XYZ-direction) at full reservoir in winter period are approximately at ½ of the dam height, in the left and right zone, in direction from the bank towards the dam, with maximal reached value of 3.9 mm. In correspondence to the resultant displacements are the horizontal (side) displacements, characterized by symmetrical distribution, reaching maximal value of 2.4 mm and 1.5 mm in the left and the right zone of the dam. The horizontal displacements (Y-direction), are dominant in downstream direction, reaching maximal value of 3.2 mm in the left intermediate zone of the dam. Vertical displacements are in gravity direction, maximal value of 1.4 mm occurred in the middle part of the dam crest.

(b) The maximal resultant displacements in the dam body (XYZ-direction) at full reservoir in summer period occurs in the dam crest, towards the left and right zone, with maximal reached value of 8.3 mm. Same as previous loading case, in correspondence to the resultant displacements are the horizontal (side) displacements, reaching maximal value of 6.1 mm and 6.9 mm in the left and the right zone of the dam crest. The horizontal displacements (Y-direction), are dominant in upstream direction, reaching maximal value of 4.8 mm in the right zone of the dam crest. Vertical displacements are in opposite of gravity, maximal value of 2.9 mm, occurred in the middle part of the dam crest.

(c) The maximal resultant displacements in the dam body (XYZ-direction) at full reservoir in winter period are approximately at ½ of the dam height, in the left and right zone, in direction from the bank towards the dam, with maximal reached value of 3.9 mm. In correspondence to the resultant displacements are the horizontal (side) displacements, characterized by symmetrical distribution, reaching maximal value of 2.4 mm and 1.5 mm in the left and the right zone of the dam. The horizontal displacements (Y-direction), are dominant in downstream direction, reaching maximal value of 3.2 mm in the left intermediate zone of the dam. Vertical displacements are in gravity direction, maximal value of 1.4 mm occurred in the middle part of the dam crest.

(d) The stresses values and distribution for full reservoir in summer period is mainly influenced by the temperature variation. The maximal allowable compressive stresses are not exceeded (σ_{doz}=10.0 MPa), while there is a case of exceedance of the maximal allowable tension stresses (σ=5.10 MPa > σ_{doz}=2.5 MPa), occurred in local zone in the right zone below the dam crest, thus indicating of a potential critical zone.

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