Theme B ANALYSIS OF A CONCRETE FACE ROCKFILL DAM INCLUDING CONCRETE FACE LOADING AND DEFORMATION USING PROGRAM PACKAGE SOFiSTiK

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Theme B

ANALYSIS OF A CONCRETE FACE ROCKFILL DAM INCLUDING CONCRETE FACE LOADING AND DEFORMATION USING PROGRAM PACKAGE SOFiSTiK

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INTRODUCTION

Mohale dam is concrete face rockfill dam, with height H=145 m, length of the dam crest 600 m and total fill volume of 7.5 millions m^3. In the process of reservoir impounding, when the reservoir level reached maximum elevation, significant dam movements occurred, this in turn increased the compressive stresses within the center portion of the concrete face, resulting in shear failure of the slab. The main objective of this analysis is to gain data for the deformation-stress state of the dam, including and the observed cracking pattern of the concrete face, using three-dimensional model with simplified topography, incorporation of the concrete face with use of the dam zoning and construction sequence. In other words, the history of the behaviour of the structure (related to construction sequence, water level histories and appearance of the cracking failure), should be followed by means of numerical experiments. These data would serve for comparison with the measured results that will be followed by interpretation of the obtained data and in the end conclusions concerning the obtained results and the behaviour of the dam.

DESCRIPTION OF THE THREE DIMENSIONAL NUMERICAL MODEL OF ANALYSIS

GENERAL PART

The used computer software SOFiSTiK for stress-strain analyses, produced in Munich (Germany), is based on the finite element method. It has a wide range of possibilities for simulation of dam behaviour and inclusion in the analyses of all necessary fenomena, important for real simulation of the dam behaviour, such as: an automatic discretization of the dam body taking into account the iregularities in the geometry of the dam base, application of different constitutive models for materials, simulation of the dam body construction and reservoir filling in increments, and so on. The program has rich possibilities for presentation of the output results. In our work, mainly plane graphical presentation was used, showing the output results in the required cross and longitudinal sections, and also in the required slab failure points. On Fig. 1 is shown the general view of the layout of the dam, Fig. 2 shows the typical cross section of the dam. In Table 1 are given the materials included in the dam body.

Table 1. Materials included in dam zoning, according to Figure 2.

Zone	Description	Lift height [m]
1A	Impervious fill	0.3
1B	Random fill	0.6
1C	Impervious fill	0.3
2A	Fine filter	0.4
2B	Durable crushed doleritic basalt	0.4
3A	Selected small quarry run rock	0.4
3B	Quarry run rockfill	1.0
3C	Quarry run rockfill	2.0
3D	Selected durable rock	NA
3E	Quarry run doleritic basalt	1.0/2.0

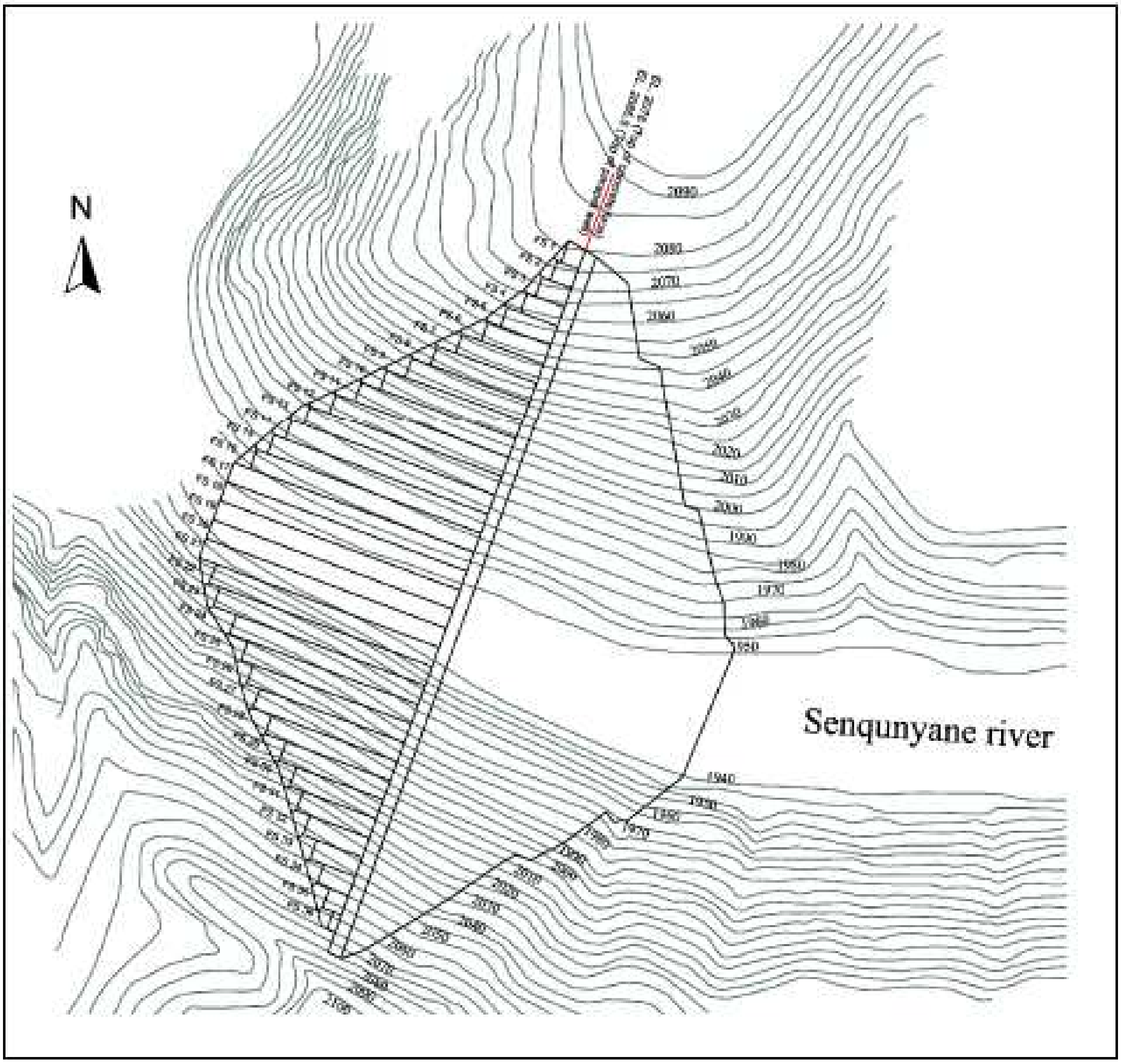


Figure 1. Layout of dam Mohale.

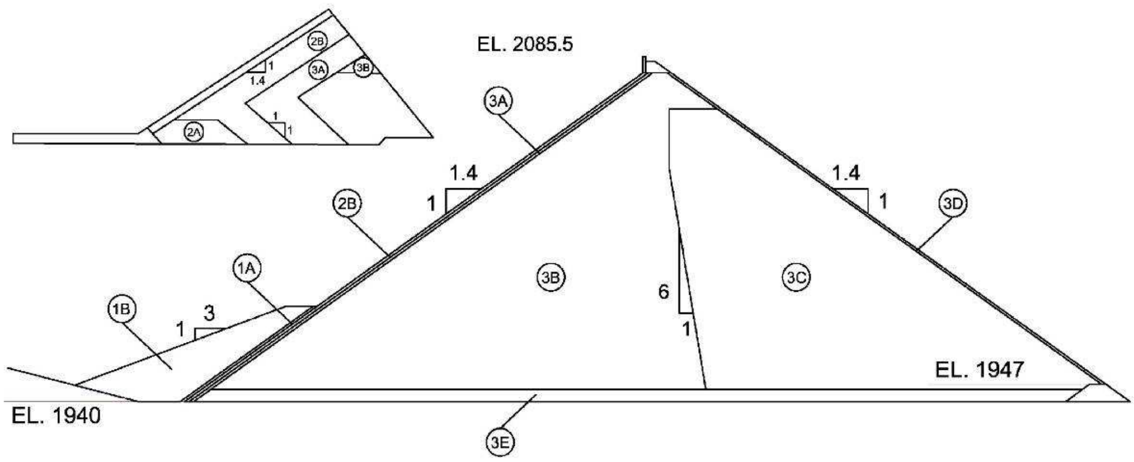
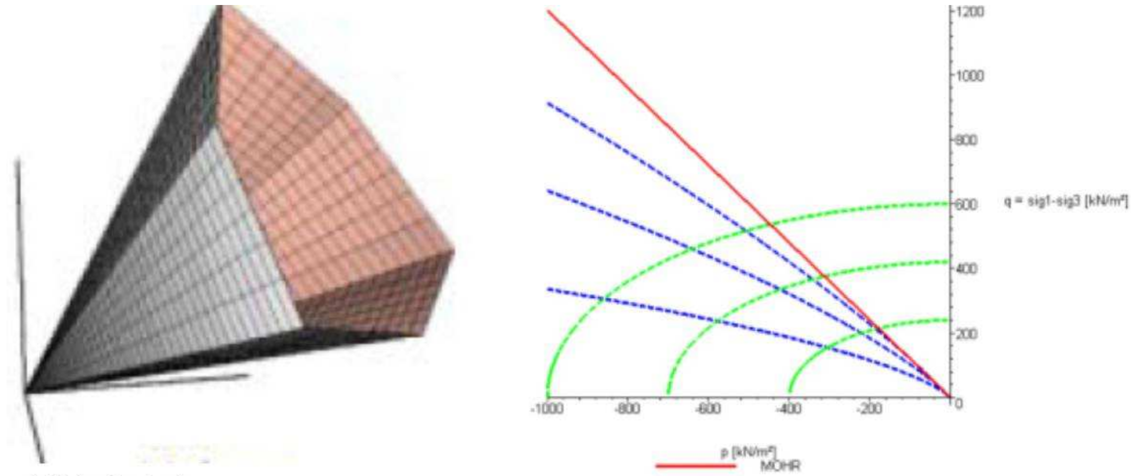


Figure 2. Typical cross section of dam Mohale with detail of the foundation slab of the concrete face.

CONSTITUTIVE RELATIONSHIP AND PARAMETERS OF THE MATERIALS

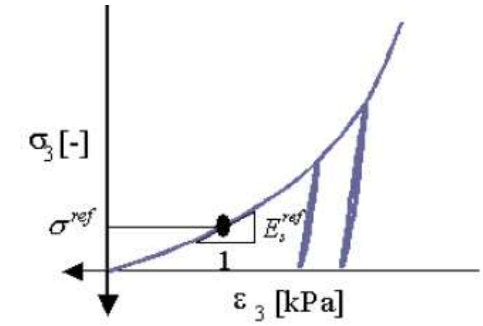
The choice of the parameters for the materials is one of the most important, but in same time and most complex tasks in the numerical analyses of dams. The rockfill body of the dam was built of three different materials with similar properties. In the analytical model they are presented with constitutive material model GRAN from the library of SOFiSTiK. That is extended elastoplastic material with an optimized hardening rule (*single* and *double hardening*) for soil materials. Hardening is limited by the material's strength, represented by the classic Mohr/Coulomb failure criterion (Fig. 3a). The hardening rule is based on the hyperbolic stress-strain relationship proposed by Kondner/Zelasko, which was derived from triaxial testing (Fig. 3b). Additionally, the model accounts for the stress dependent stiffness according to equations (1−3). A further essential feature is the model's ability to capture the loading state and can therefore automatically account for the different stiffness in primary loading and un−/reloading paths.



a) Mohr-Coulomb b) Kondner-Zelasko hyperbolic relation

Figure 3. Mohr-Coulomb failure criterion and hyperbolic stress-strain relationship.

Rockfill material shows stiffness behaviour dependent of the stress state. The oedometric modulus magnitude depends on the effective axial stress state according to the following relationships, equation (1):



$$E_s = E_{s,ref} \cdot \left(\frac{|\sigma_3| \cdot \sin\varphi + c \cdot \cos\varphi}{p_{ref} \cdot \sin\varphi + c \cdot \cos\varphi} \right)^m \tag{1}$$

E_{50} is defined as secant stiffness that corresponds to a 50−percent mobilisation of the maximum shear capacity, given by equation (2).

$$E_{50} = E_{50,ref} \cdot \left(\frac{|\sigma_1| \cdot \sin\varphi + c \cdot \cos\varphi}{p_{ref} \cdot \sin\varphi + c \cdot \cos\varphi} \right)^m \tag{2}$$

An analogous approach for the elastic un−/reloading stiffness yields, given by equation (3):

$$E_{ur} = E_{ur,ref} \cdot \left(\frac{|\sigma_1| \cdot \sin\varphi + c \cdot \cos\varphi}{p_{ref} \cdot \sin\varphi + c \cdot \cos\varphi} \right)^m \tag{3}$$

The face of the dam was built of the concrete C 25. There was no date of the laboratory test of the concrete and the reinforcement, so the standard concrete and steel properties where adopted (Fig. 4 and Fig. 5).

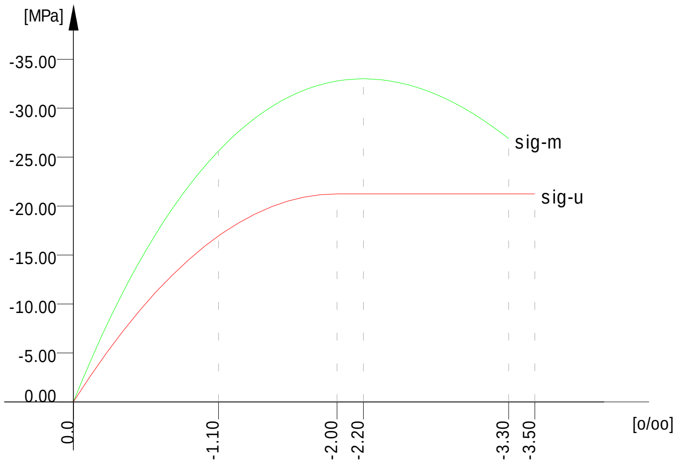


Figure 4. Stress-strain relationship of concrete C 25.

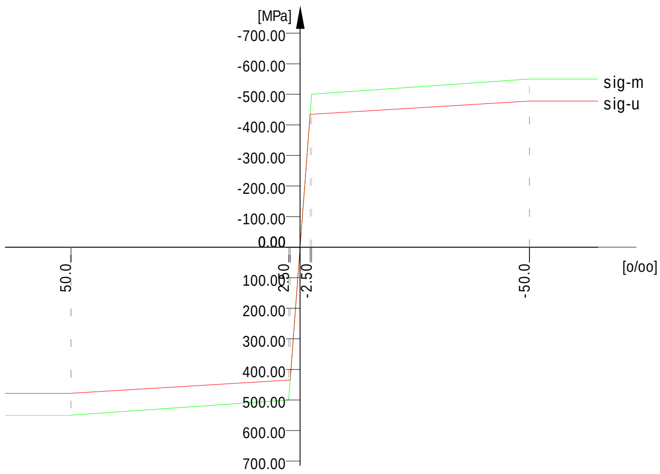


Figure 5. Stress-strain relationship of steel S 500.

NUMERICAL MODEL

Using the given data three-dimensional (3D) mathematical model of the dam was build (Fig. 6). The finite element mesh was generated using SOFiSTiK automatic mesh generator. Three types of element were used to model the dam's body: BRIC and QUAD elements. The body of the rockfill dam was modelled with 3D solid elements (8^{th} node issoparametric).The solid element of SOFiSTiK is the BRIC (volume) element (Fig. 7), a general six-sided element with eight nodes.

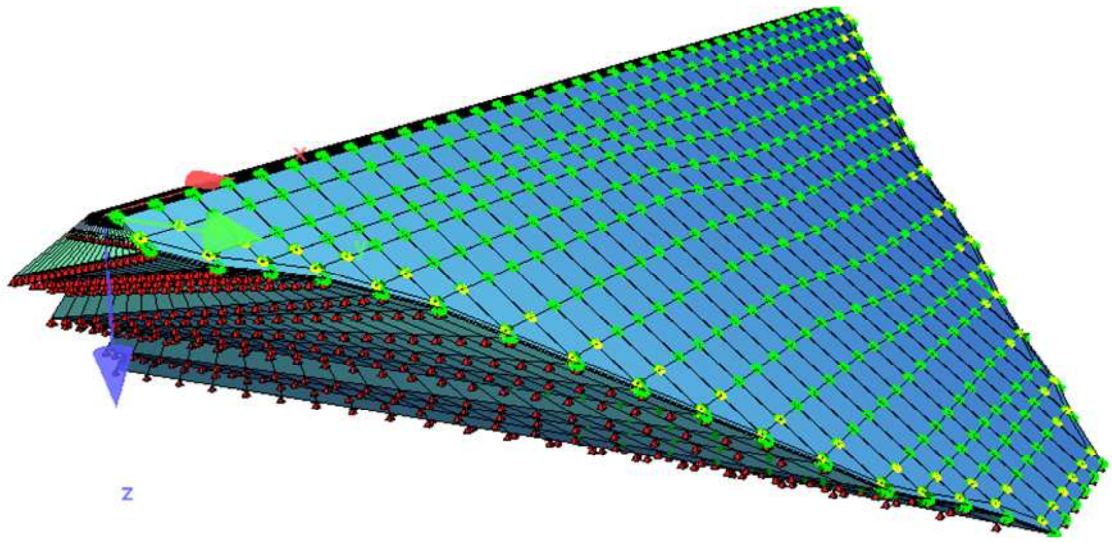


Figure 6. General view of the 3-D model of the dam-foundation system.

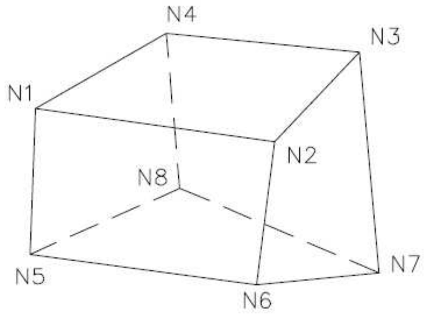


Figure 7. BRIC element.

The plane element QUAD (shell) (Fig. 8) is a general quadrilateral element with four nodes. The surfaces of the BRIC element can be described by special QUAD-elements, which can also be employed for the display of stresses in BRIC-elements. The QUAD-elements are introduced in order to simplify the process of generation of the water pressure loads.

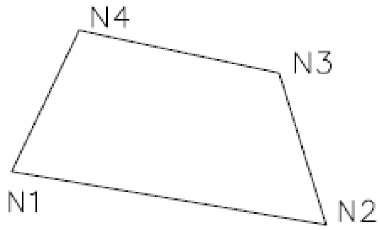


Figure 8. QUAD element.

The input data in the analysis were the given data regarding the geometry of the dam and the material properties. In order to simulate the construction sequences the body of the dam was modelled with 18 construction stages (horizontal layers). The concrete face of the dam was modelled by shell elements. It was applied at the body in two construction stages. The face elements are with variable thickness. The thickness of the concrete was determined as a function of the hydrostatic pressure, following the equation e=0.30+0.00H. Adopted reinforcement is 0.4% in vertical direction and 0.35% in horizontal direction. The concrete face was built in several strips (15 m width) and joint gaps between the strips were adapted 2 cm (Fig. 9). For the needs of the nonlinear analysis the shell element are treated as layered in 10 concrete layers, (Fig. 10). The reinforcement is also simulated by smeared layers. That model is able to reproduce the 3D crack pattern developing. Construction stages where simulated in the following sequences

CS 1 ÷ CS 11	- rockfill body up to elevation 2040
CS 12	- rockfill body + concrete face up to elevation 2040
CS 13 ÷ CS 17	- rockfill body up to elevation 2078 + concrete face up to elevation 2040
CS 17	- rockfill body + concrete face up to elevation 2078

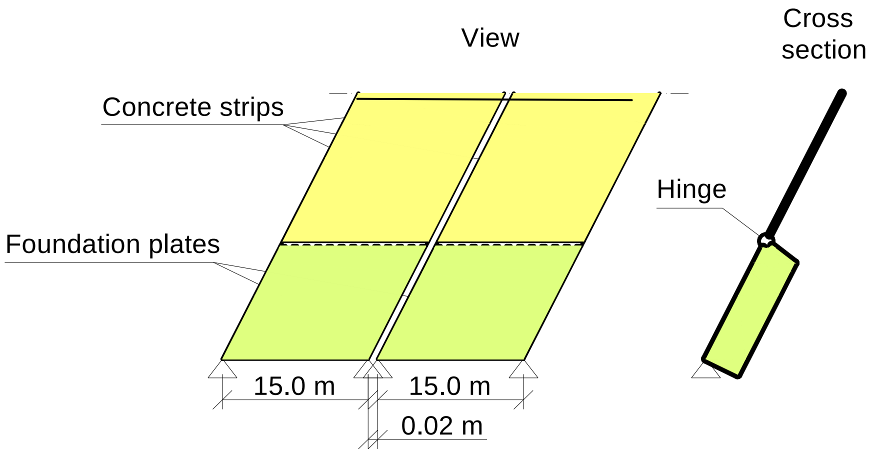


Figure 9. Display of the concrete face discretization.

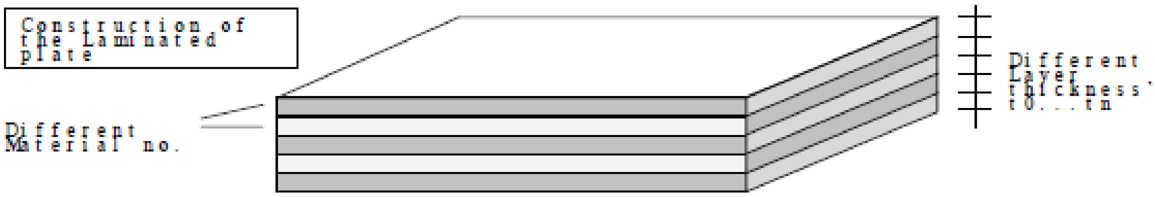


Figure 10. Simulation of the construction of the shell elements in layers.

The numerical model gives two possibilities for simulation of the supports:

- Simply support connection where there are no movements in the supported points.
- Supports by linear/nonlinear bedding. With this type of supports some effects characteristic for soli-structure interaction can be simulated (sliding of the dam body, lifting in the case of the tension, linear/nonlinear friction and similar).

The strips of the concrete face are connected with stiffer foundations. The interface between these elements is with edges hinge that allowed independent rotation of strips and foundation along the common edges, but keep the connection between the both.

INTERFACE

The dam was build of elements with very different stiffness. In order to avoid numerical problem during nonlinear analyses, rigid connection of that type of element should be avoided. The strips faced element initially has gap of 2 cm (Fig. 11). During the construction and impounding, free movement between neighbour points happened till the points contact each other. After that the point moved together without, and some forces can be transferred between the points. This type of the interface was simulated with nonlinear spring that cannot transfer tension forces. The spring will be activated in the moment when the distance between the points became equal to the gap (2 cm).

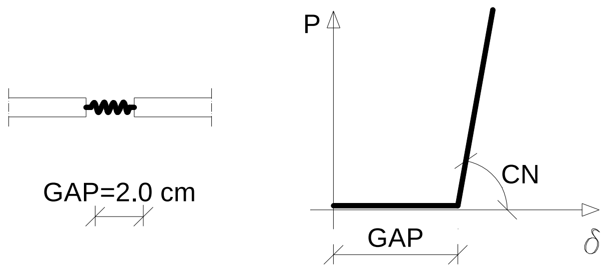


Figure 11. Spring-force displacement relation.

The connection between the concrete strips and the rockfill body was simulated with nonlinear distributed springs. The characteristics of these springs are as follow (Fig. 12):

- CN – axial spring constant. Usually very large number to preserve contact between the strips and the body in the normal direction.
- CT – lateral spring constant. That constant can allow movement between the interface surfaces. The value of that constant dependent of the friction between the two surfaces.
- CRAC – failure load. In the case CRAC = 0, no tension between the surface is allowed.
- GAP – the spring will be activated when the distance between the points became equal to the gap's value.

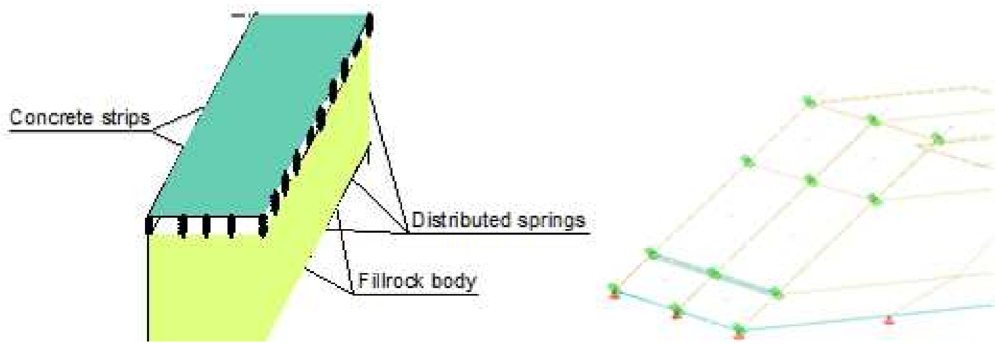


Figure 12. Connection between the concrete strips and the rockfill body.

LOADING OF THE DAM

According to the given data, appropriate loads from self weight and water pressure were created. SOFiSTiK has very efficient tools for load generation. The self weight loads are applied with the values of the unit weight of the materials in the dam body and division of the dam body in appropriate construction sequences, according to the given data, with command to act in the gravity direction. The simulation of the dam construction in layers is displayed on Fig. 13. The temperature, and the creep and shrinking effects were not taken into consideration.

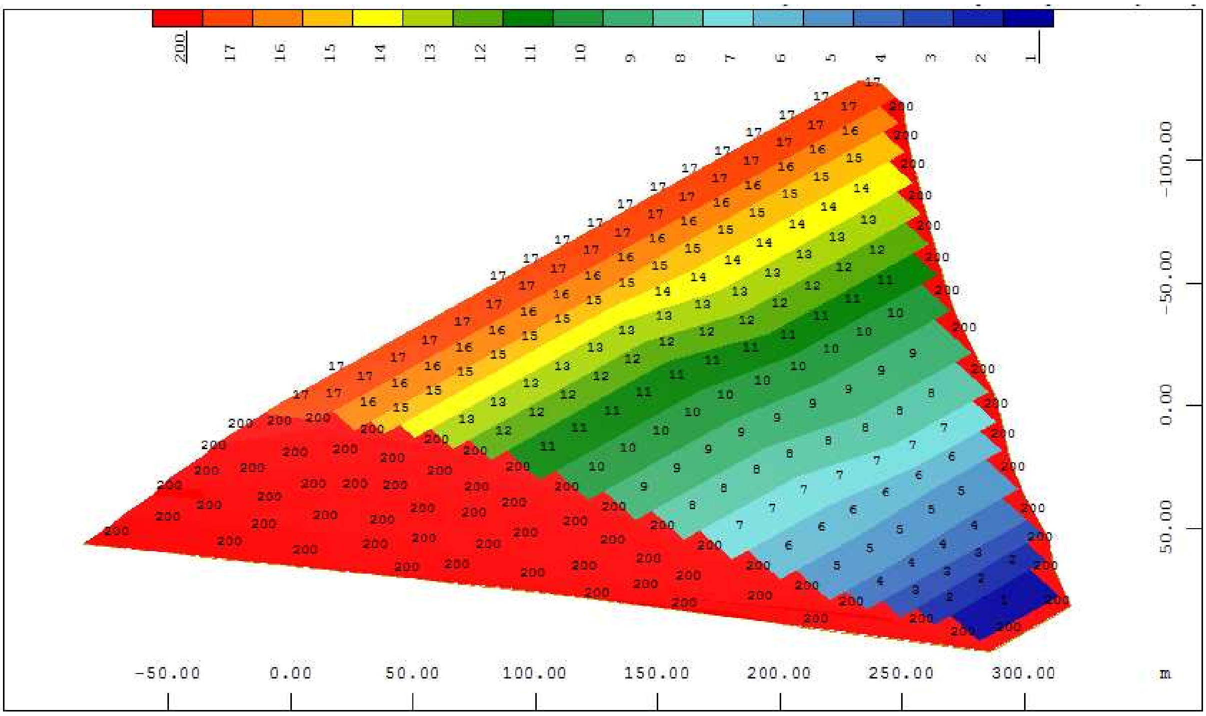


Figure 13. Simulation of dam construction in layers.

With the VOLU-statement, a loading on volumes, group of BRIC-elements or all QUAD-elements within the volume is enabled. The changes of the pressure load along tree edges are specified. All referenced elements of the group or the volume will be loaded. The selection of elements is only by a sheared cube (Fig. 14), defined by three selectable directions p_1-p, p_3-p_2, p_5-p_4, which must not be collinear. For transfer of the water pressure loads, some QUAD elements faces on the upstream of the dam's body was created, that are enabling to introduce the concrete face in the analysis and to apply the water load adequately (Fig. 15 and 16). This elements has only geometrical values, they have no contribution on the total structure stiffness. Referencing these QUAD elements in the VALU statement cause simple transfer of the water pressure in appropriate node loads. The water load is applied as hydrostatic pressure in increments.

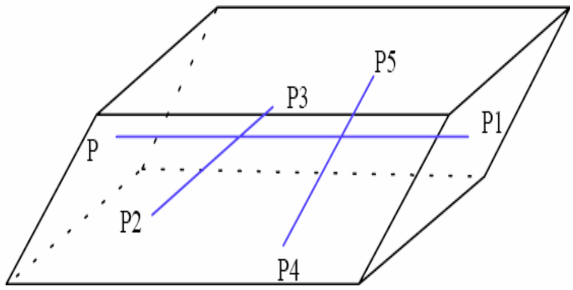


Figure 14. Sheared cube.

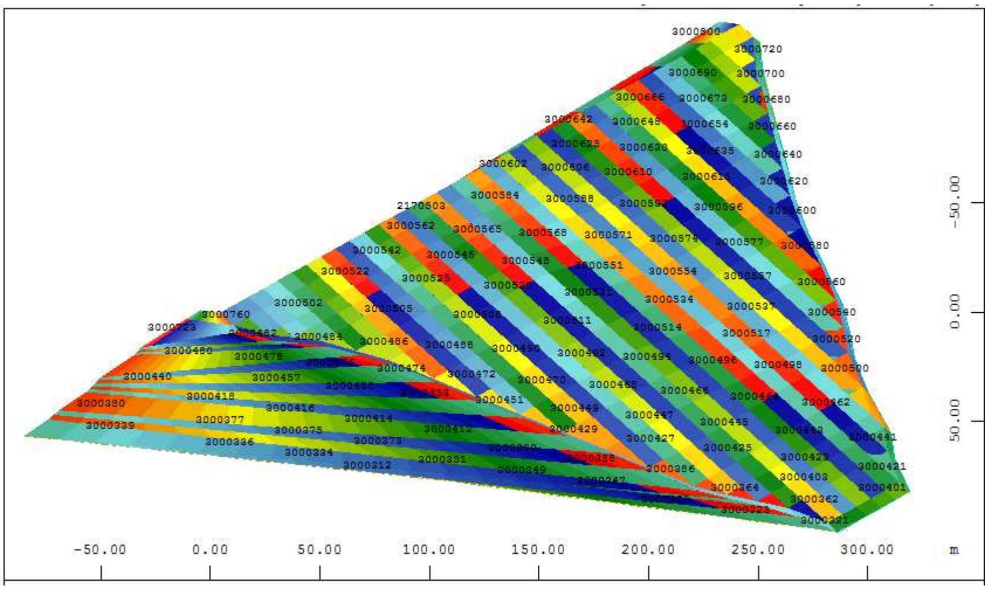


Figure 15. QUAD elements faces on the 3-D model of the dam's body.

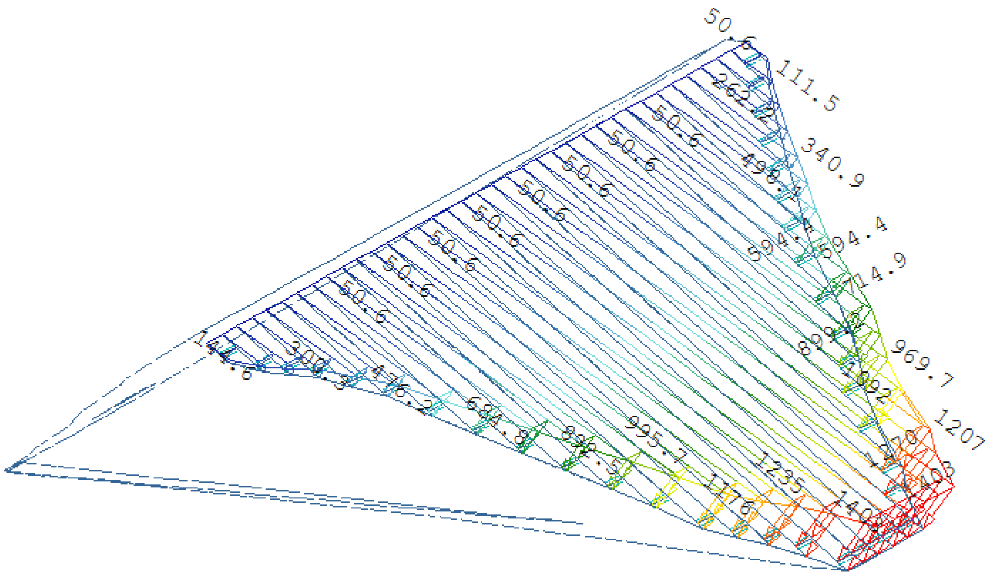


Figure 16.. Full water load applied on the 3D model of the dam, step 20.

ANALYSES OF THE DAM

Fill settlements

The dam was analyzed on statically action of self weight and water pressure loads. In order to obtain complete information about the stress-deformation state of the dam, in the following are displayed the obtained results for the displacements and the stresses in the dam body in the required cross sections for two loading stages (Fig. 17): stage after the end of dam construction and stage after the impounding of the reservoir.

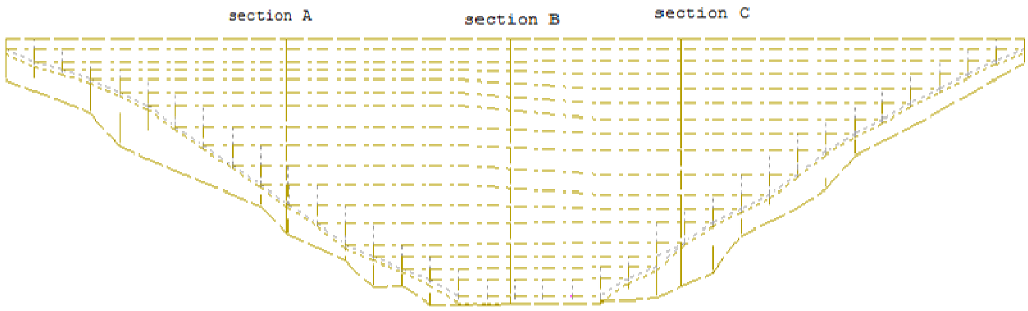


Figure 17. Location of the sections A, B and C in the dam.

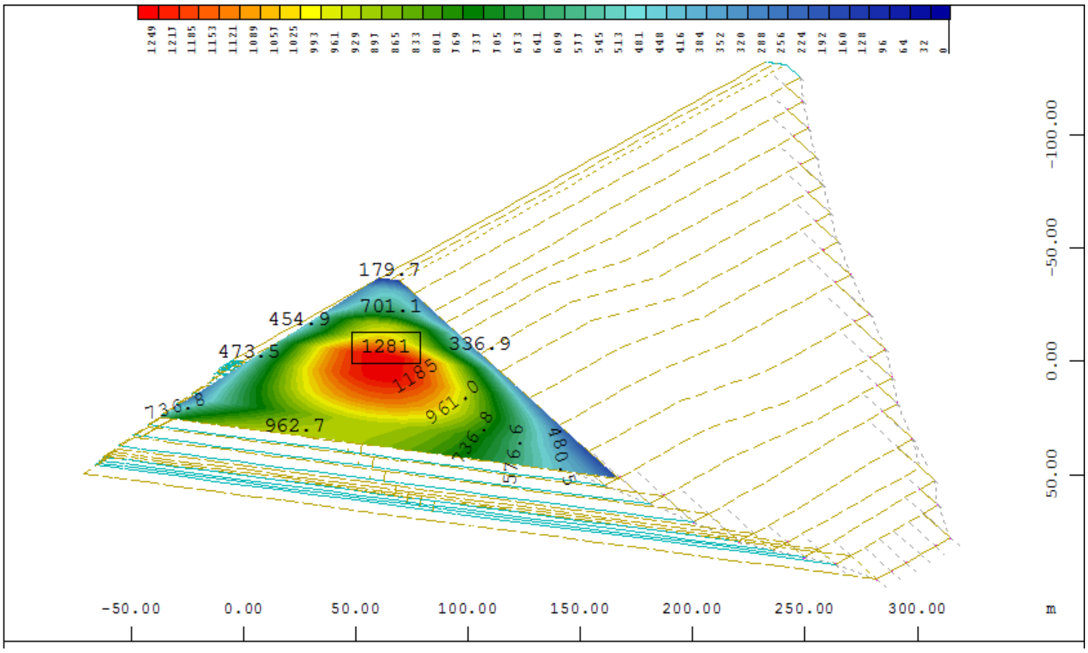


Figure 18. Vertical displacements after the dam construction, section A, Y = (0.0 ÷ 1281) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

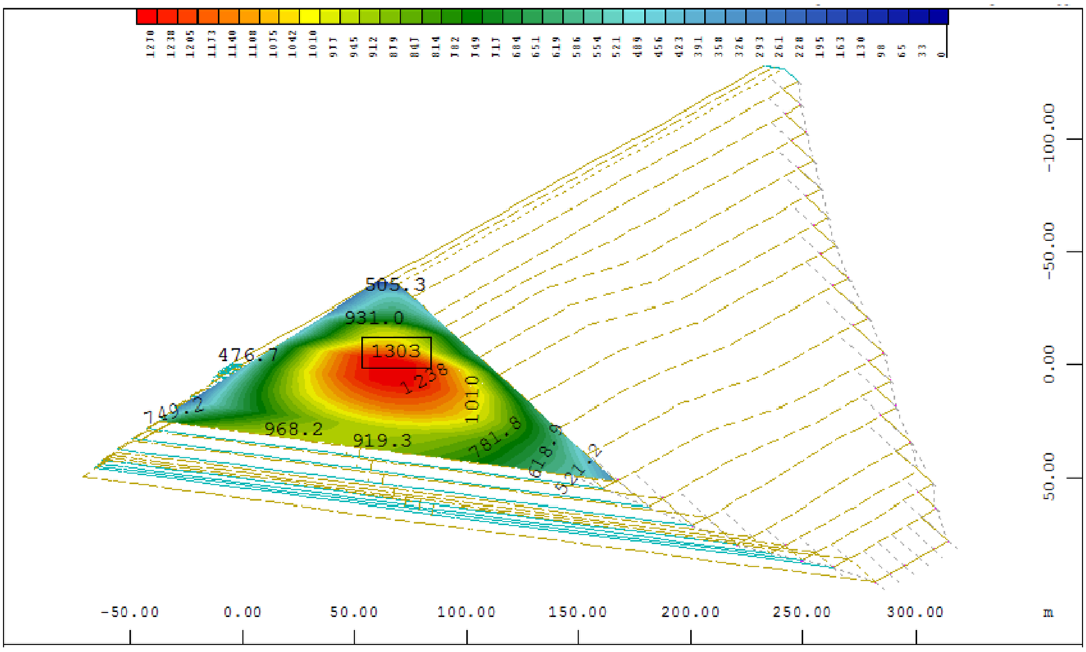


Figure 19. Vertical displacements after reservoir impounding, section A, Y = (0.0 ÷ 1303) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

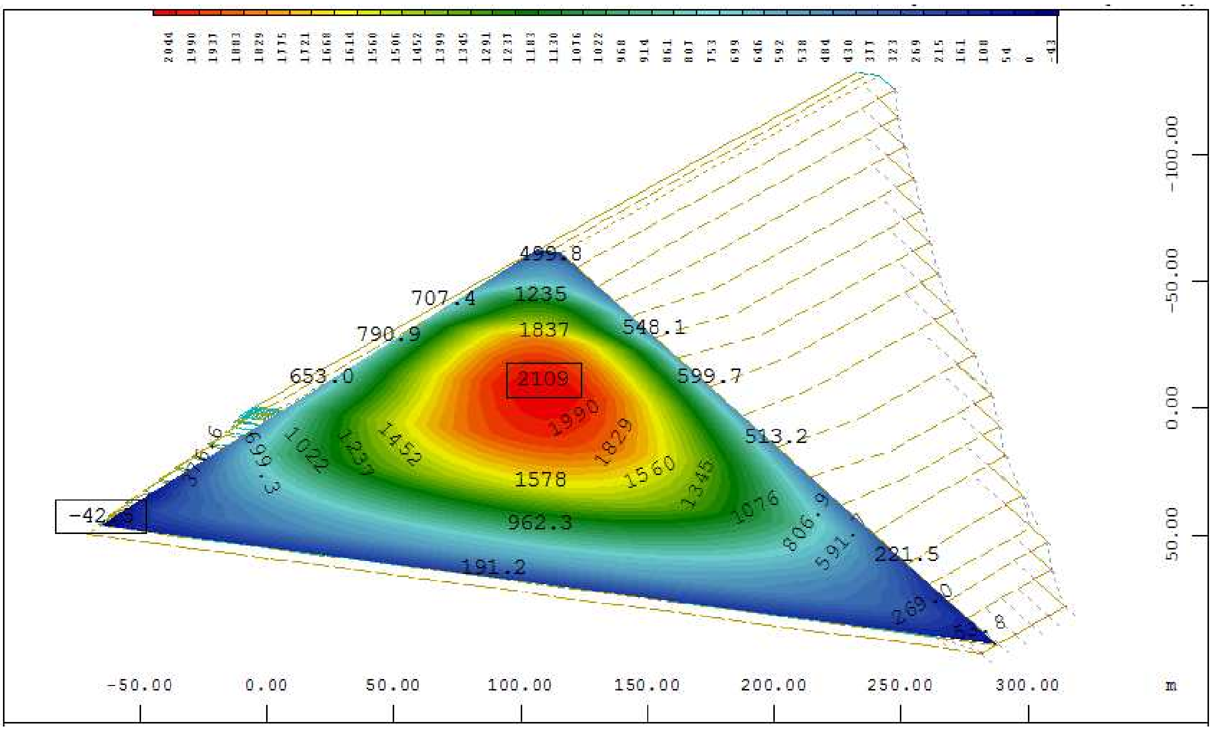


Figure 20. Vertical displacements after the dam construction, section B, Y = (0.0 ÷ 2109) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

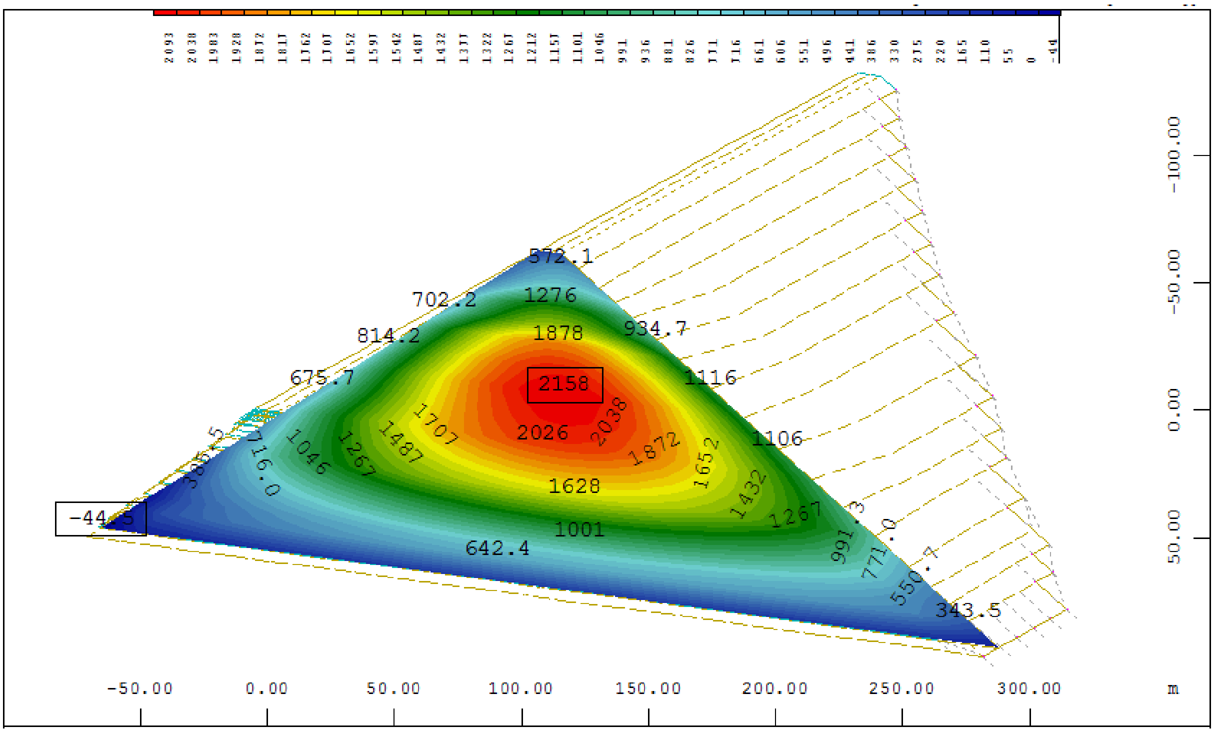


Figure 21. Vertical displacements after reservoir impounding, section B, Y = (0.0 ÷ 2158) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

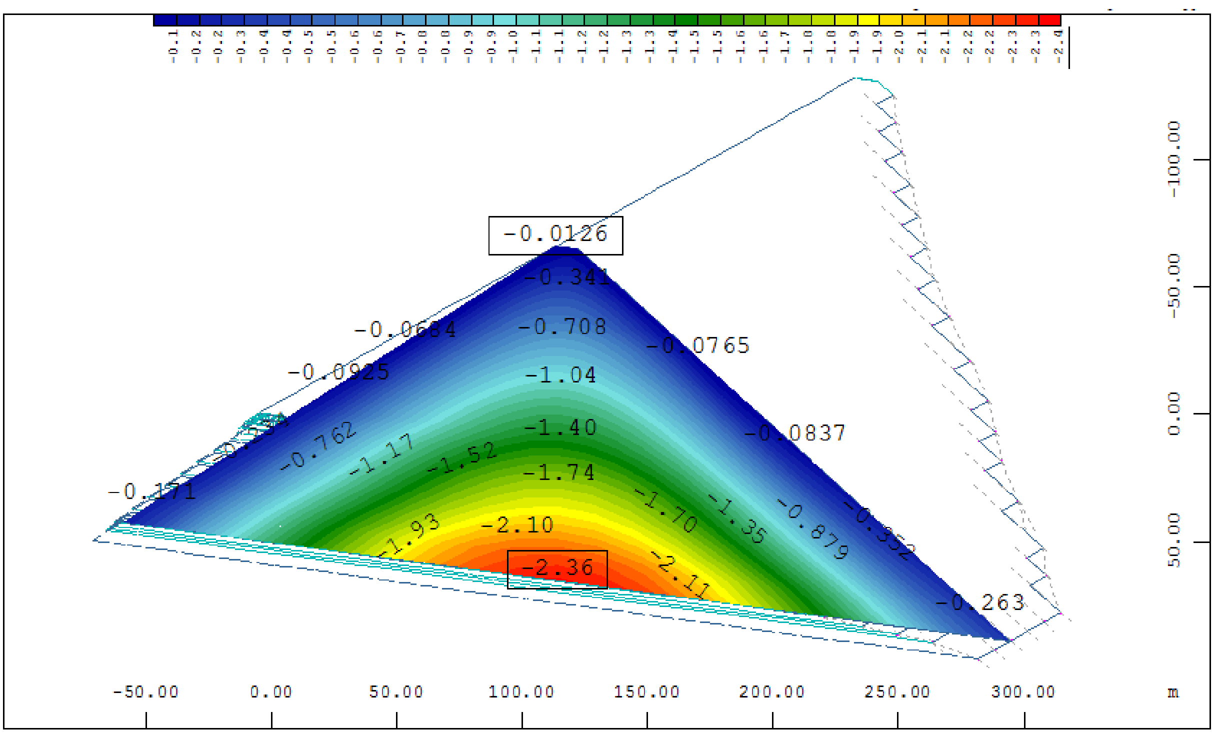


Figure 22. Vertical stresses after dam construction, section B, σ_Y= (-0.0126 ÷ -2.36) MPa.

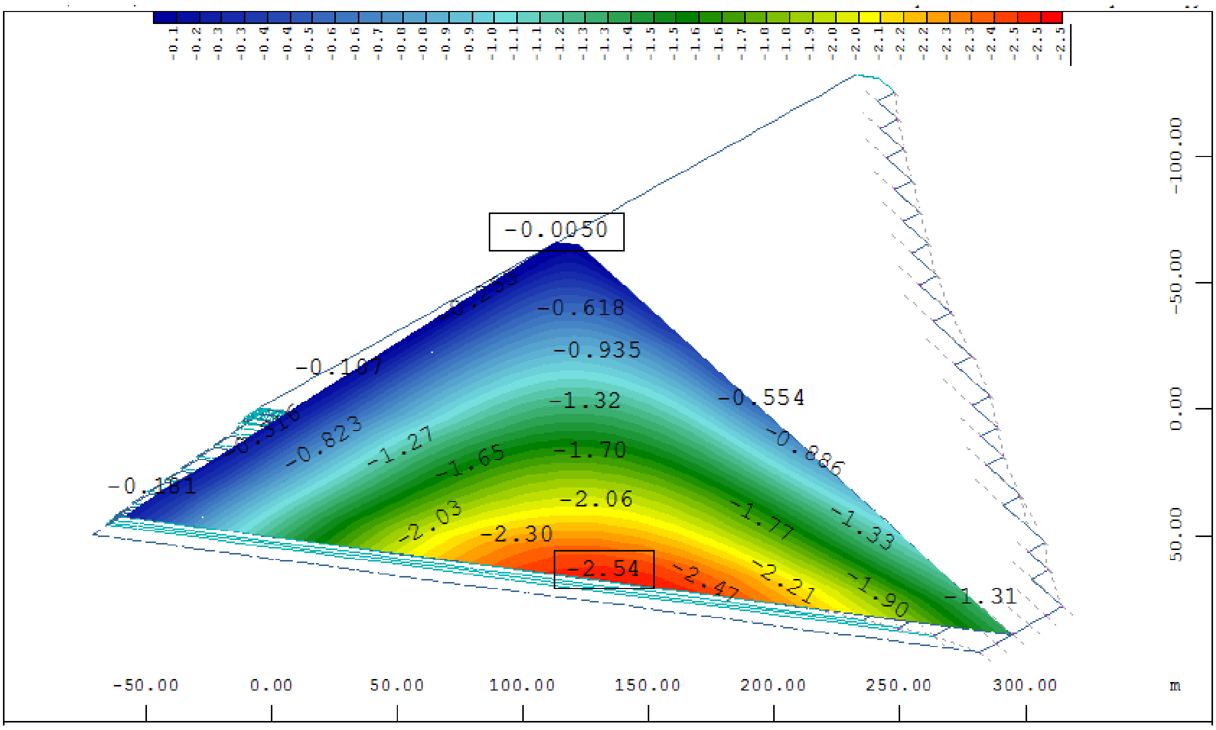


Figure 23. Vertical stresses after reservoir impounding, section B, σ_Y= (-0.005 ÷ -2.54) MPa.

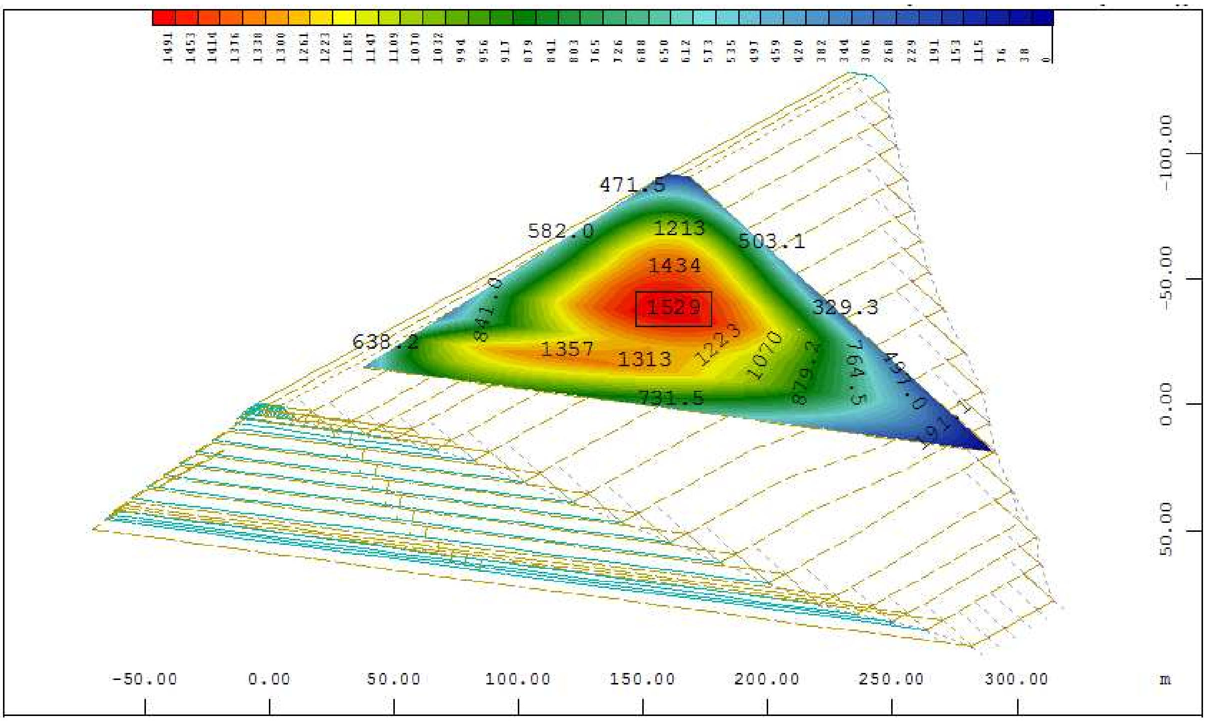


Figure 24. Vertical displacements after the dam construction, section C, Y = (0.0 ÷ 1529) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

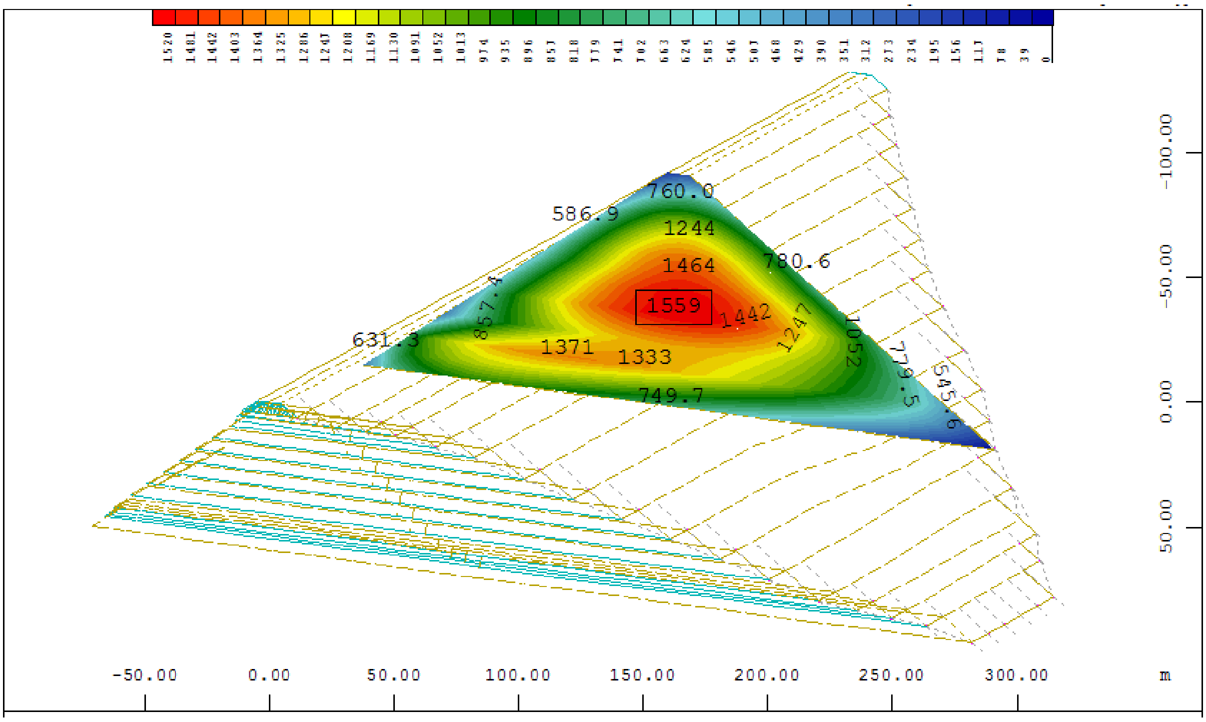


Figure 25. Vertical displacements after reservoir impounding, section C, Y = (0.0 ÷ 1559) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

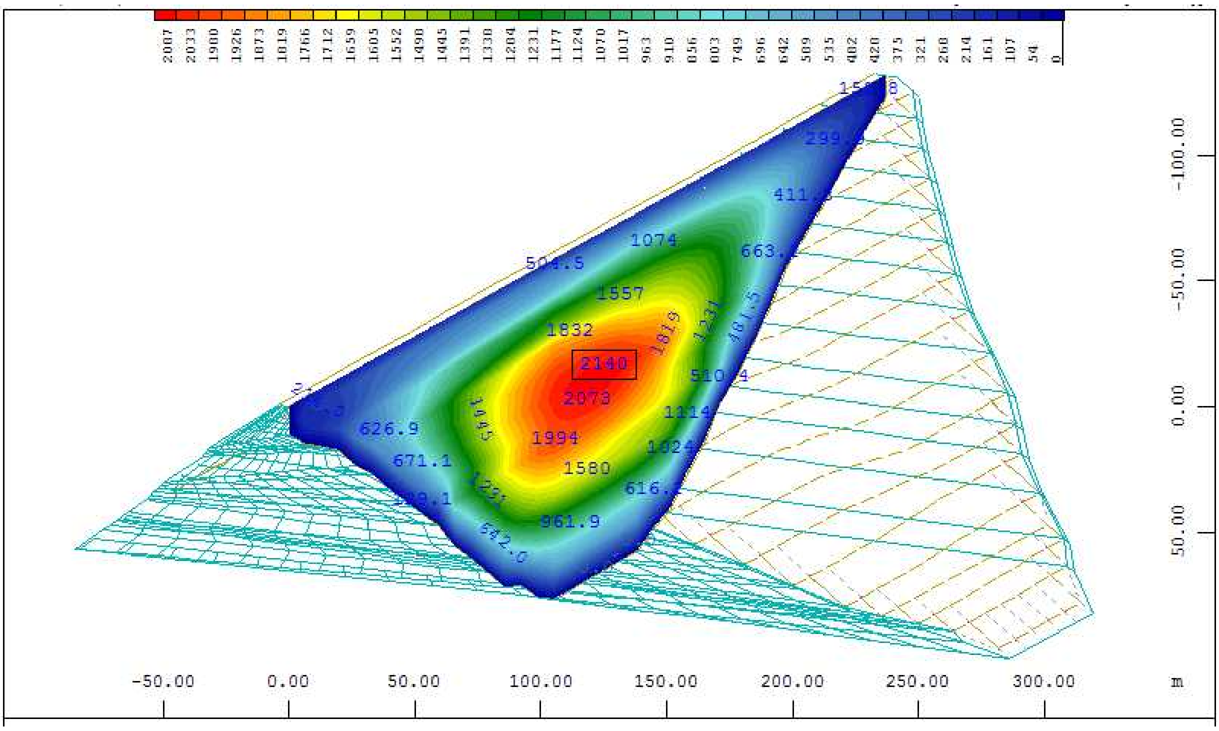


Figure 26. Vertical displacements after dam construction, longitudinal section of the dam, Y = (0.0 ÷ 2140) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

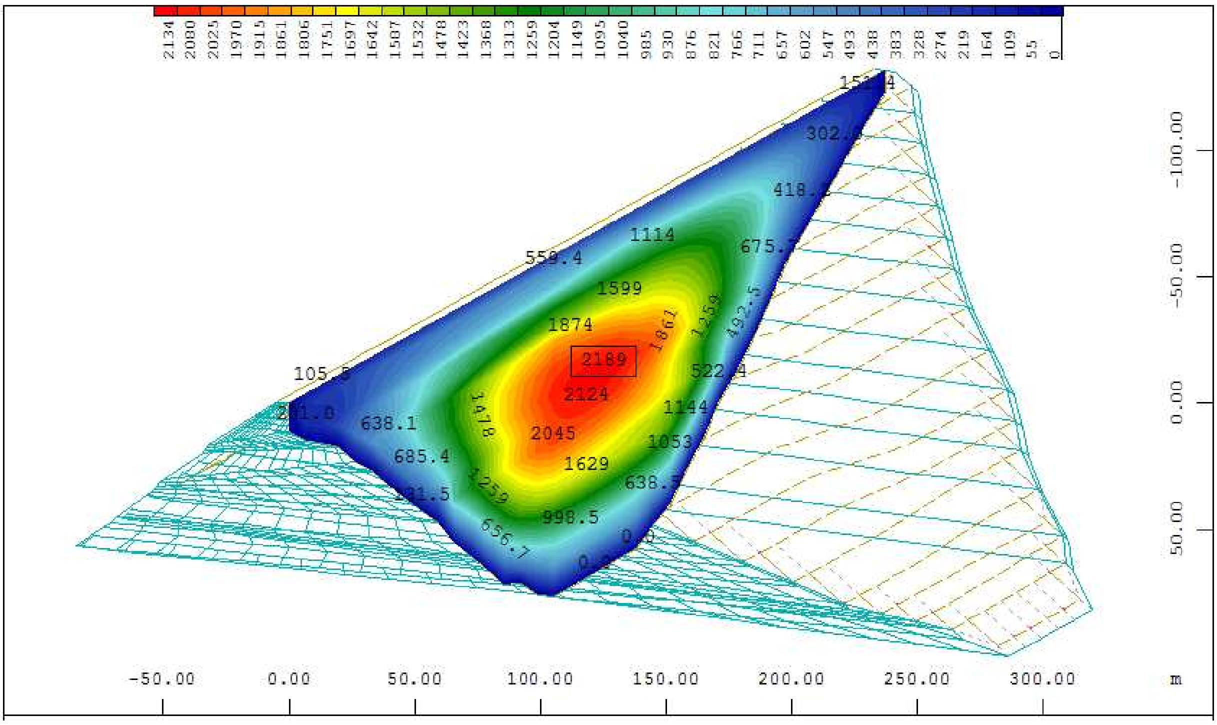


Figure 27. Vertical displacements after reservoir impounding, longitudinal section of the dam, Y = (0.0 ÷ 2189) mm, (-) is displacement opposite of the gravity direction, (+) is displacement in gravity direction.

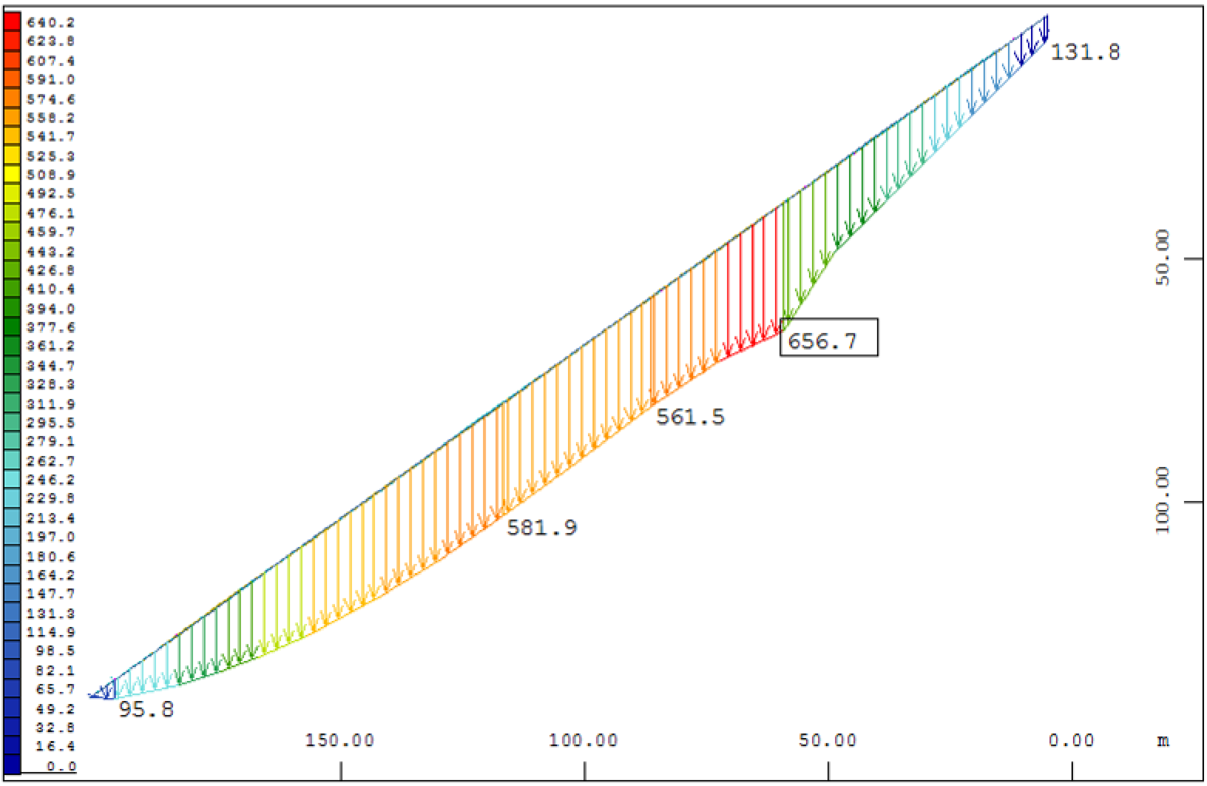


Figure 28. Surface settlements for lake level at EL 2075, upstream side.

Strains and stresses within the Concrete Face

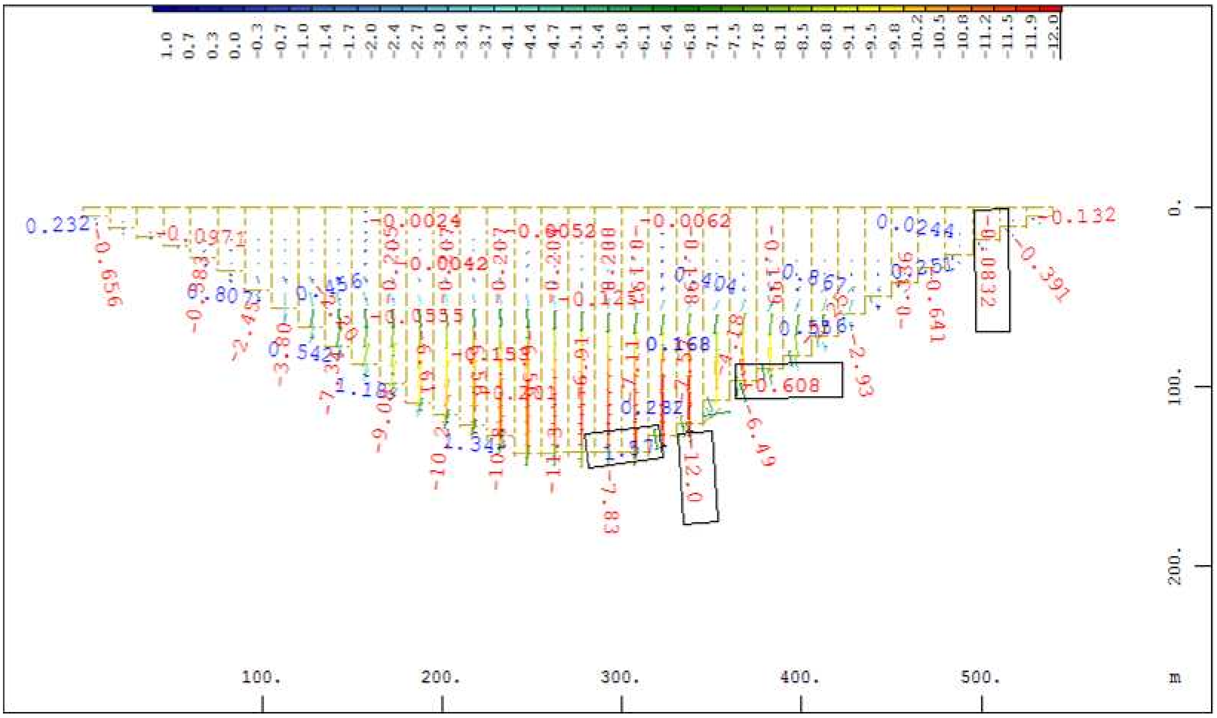


Figure 29. Top non-linear principal stress in the concrete face after dam construction, σ=(0.083 ÷ -12.0) MPa.

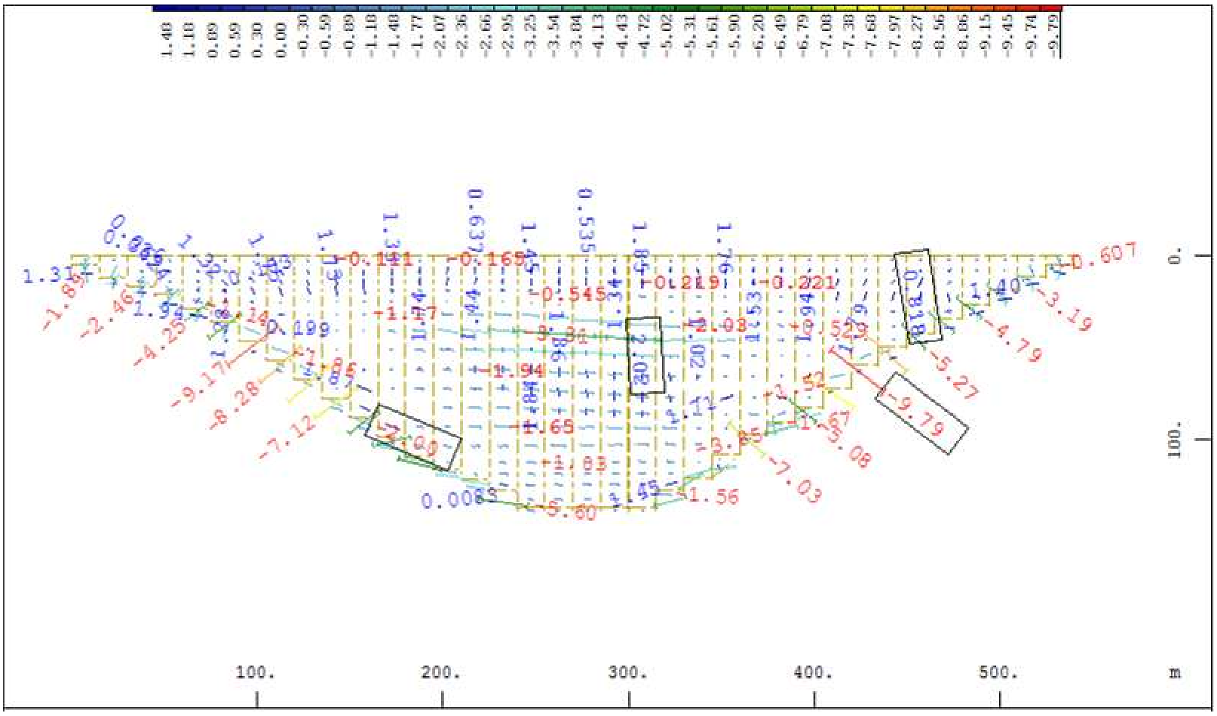


Figure 30. Top non-linear principal stress in the concrete face after reservoir impounding, σ = (1.48 ÷ -9.79) MPa.

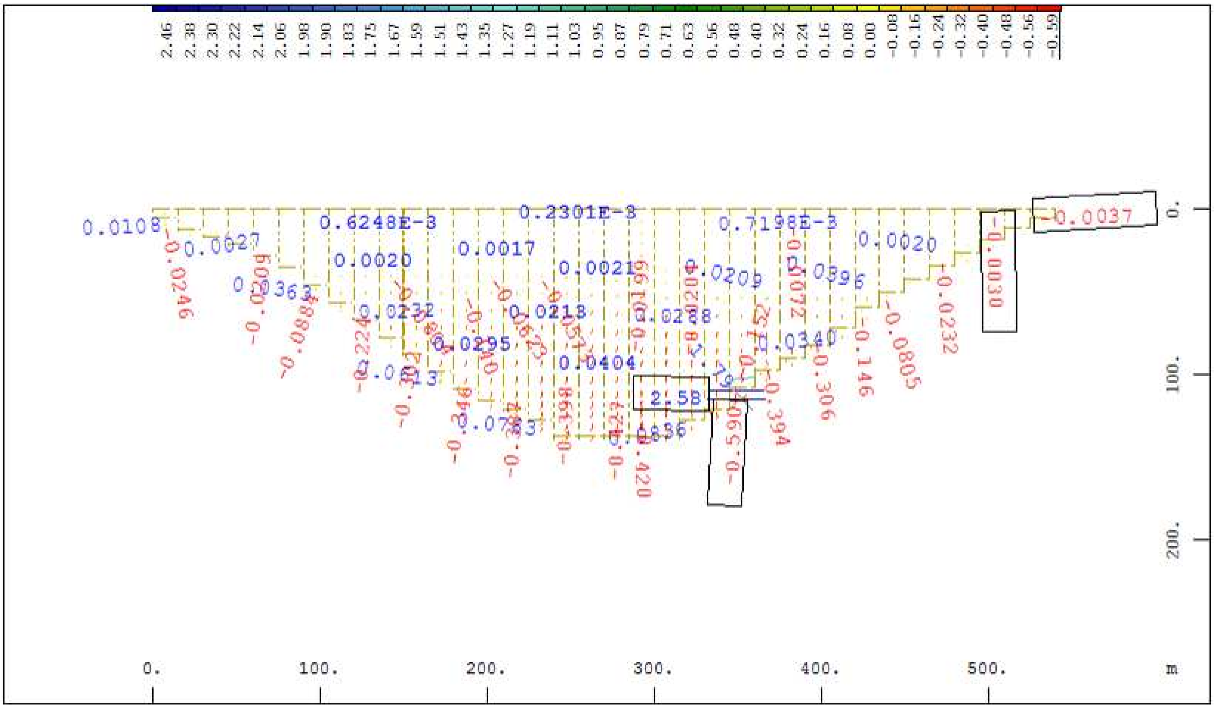


Figure 31. Top non-linear principal strains in the concrete face after dam construction, ε = (2.58 ÷ -0.59) ‰.

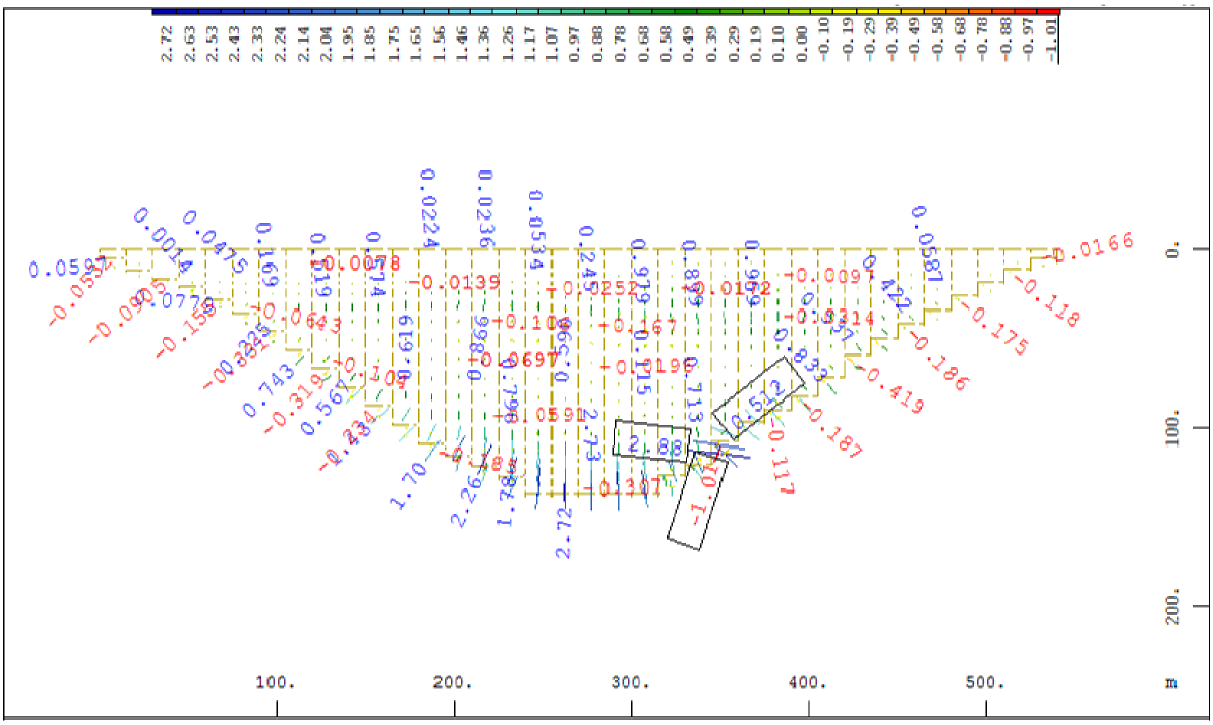


Figure 32. Top non-linear principal strains in the concrete face after reservoir impounding, ε = (2.72 ÷ -1.01) ‰.

Face deflections

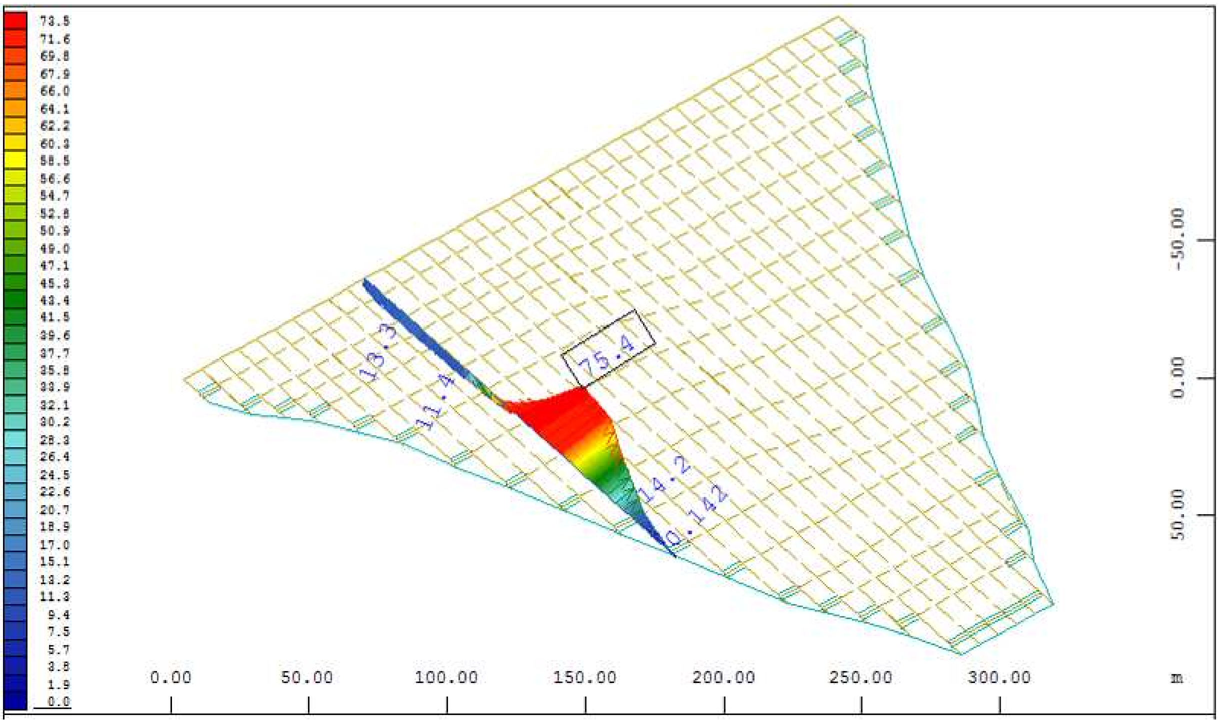


Figure 33. Face deflection after dam construction, section A, D=(0.0 ÷ 75.4) mm.

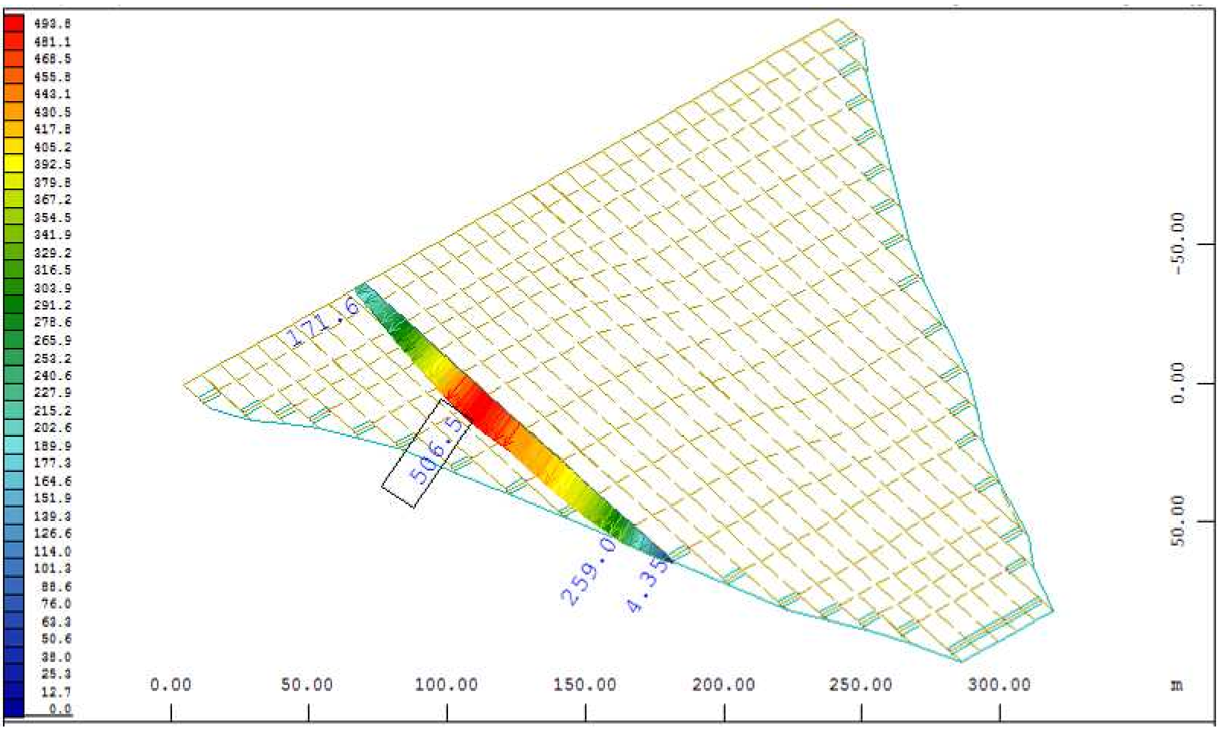


Figure 34. Face deflection after reservoir impounding, section A, D=(0.0 ÷ 506.5) mm.

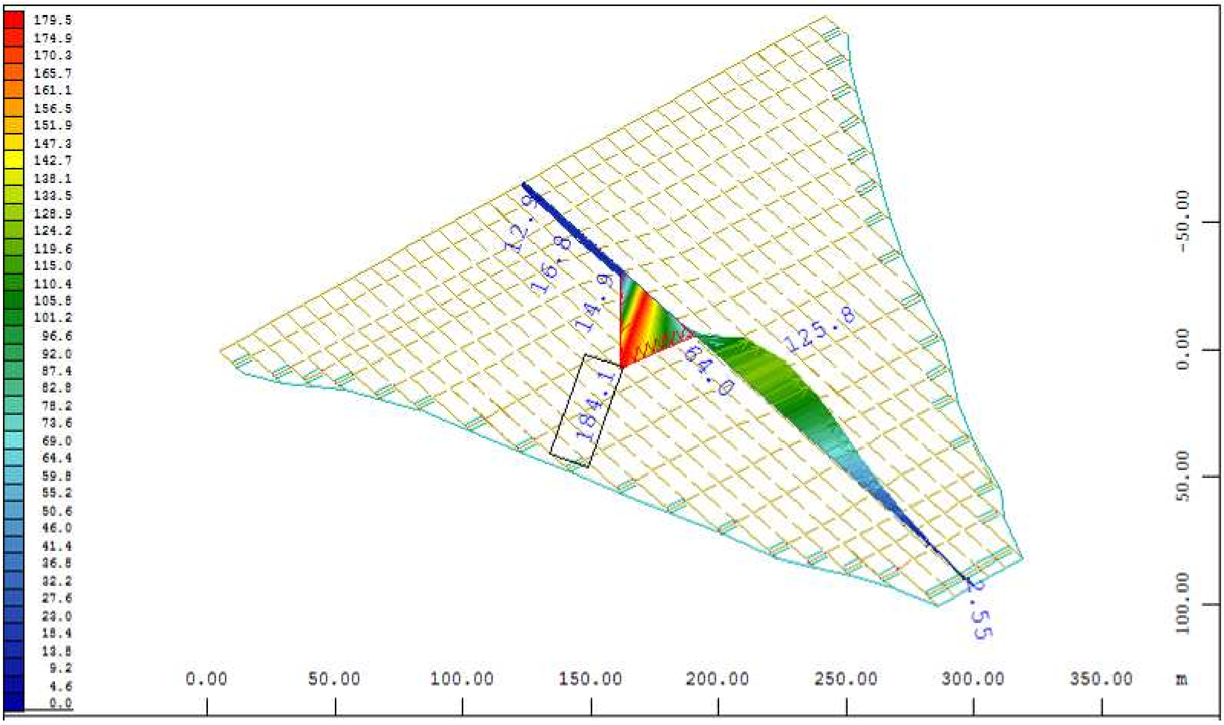


Figure 35. Face deflection after dam construction, section B, D=(0.0 ÷ 184.1) mm.

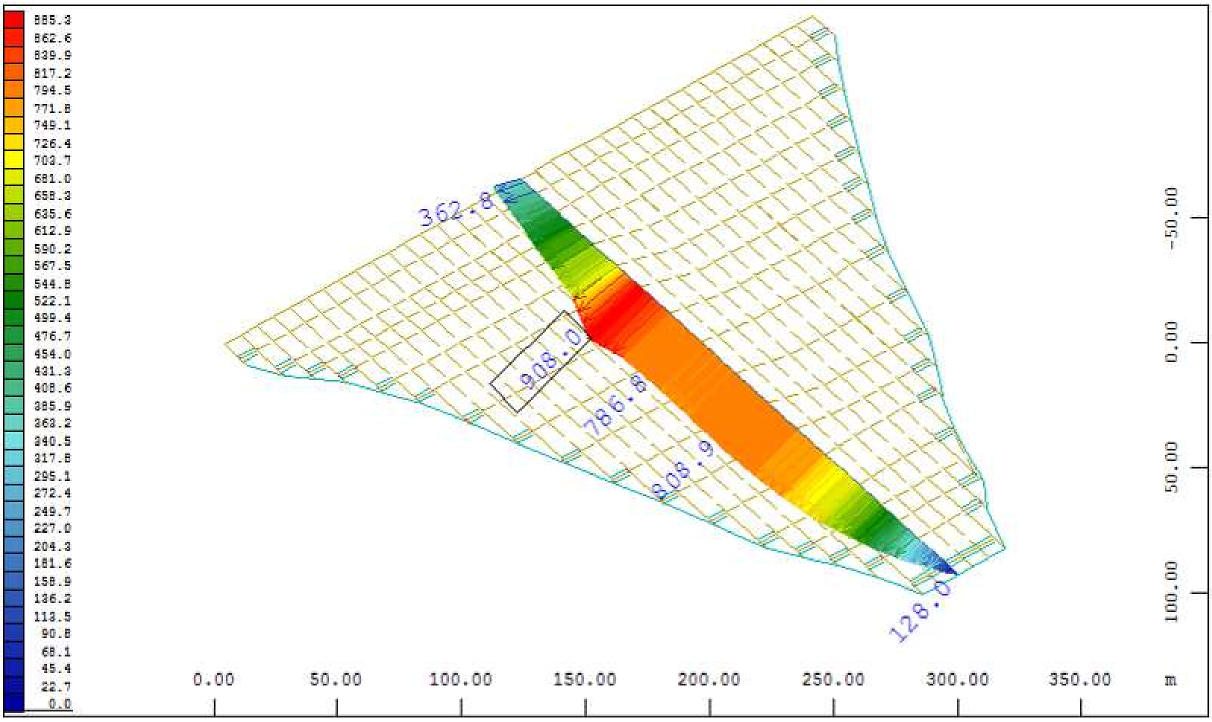


Figure 36. Face deflection after reservoir impounding, section B, D=(0.0 ÷ 908) mm.

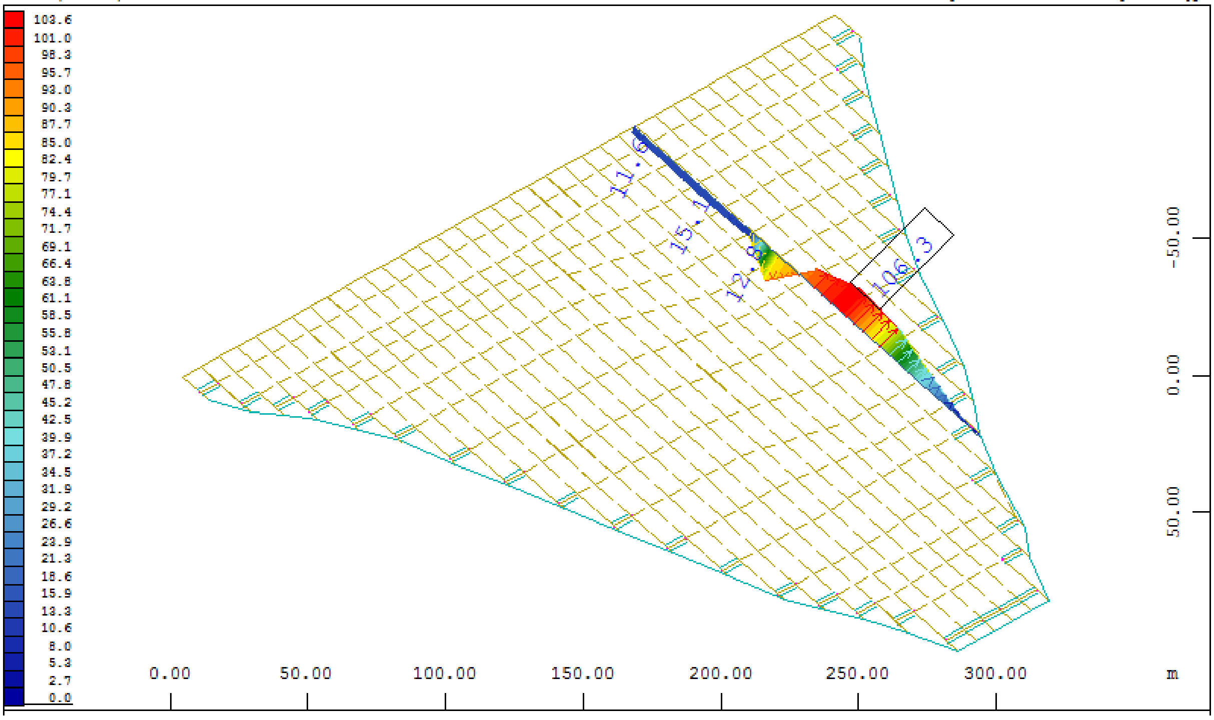


Figure 37. Face deflection after dam construction, section C, D=(0.0 ÷ 106.3) mm.

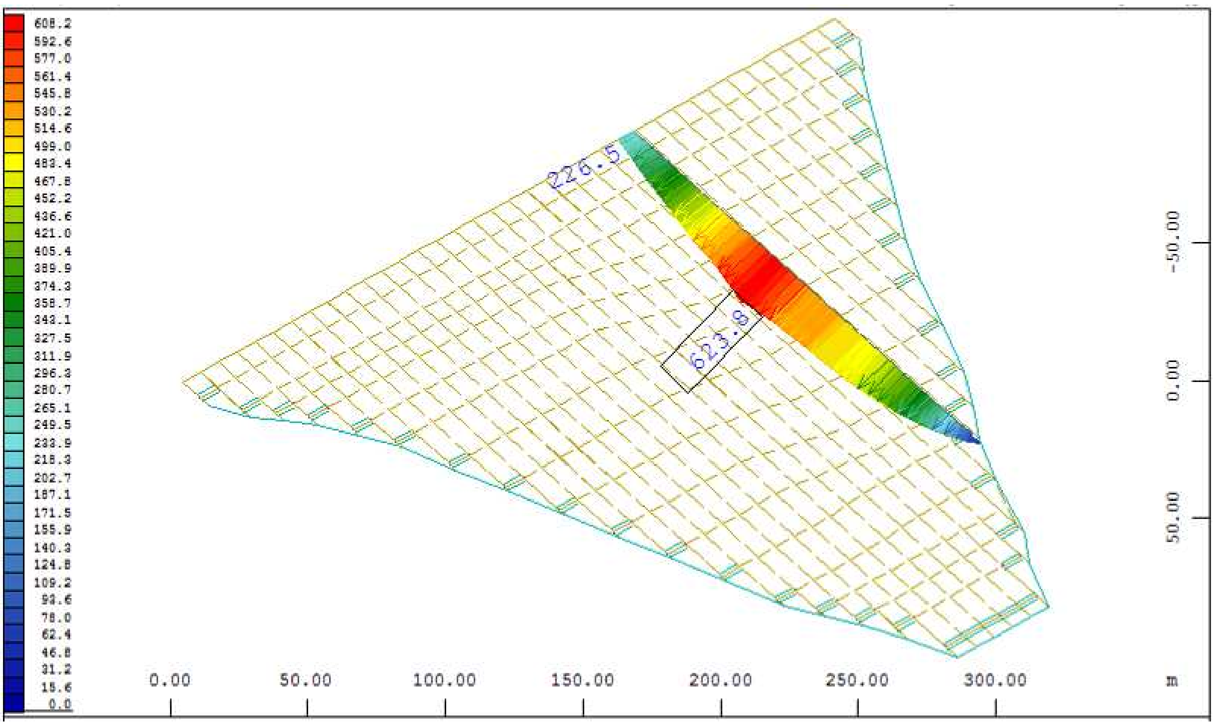


Figure 38. Face deflection after reservoir impounding, section C, D=(0.0 ÷ 623.8) mm.

CONCLUSIONS

From the performed analysis following main conclusions could be drawn out:

→ Program package SOFiSTiK is powerful tool for complex three-dimensional analysis of dams. It has rich possibilities for modeling of the dam body, and also possibilities for application of different constitutive laws, as well as for complex load influences.

→ From the analysis of dam behaviour for the loading states (state after dam construction and state after reservoir impounding), obtained values and distribution of the dam settlements and vertical stresses are usual for this type of dam. The maximal vertical settlement is in the intermediate part of the dam, located approximately at 60% of the dam height, with value 2.138 m at point HS B11, while the measured value at the same point is 2.32 m. This shows good accordance between these values. Maximal vertical stresses are at the bottom, in the central part of the, with value of 2.36 MPa.

→ From the comparison of the measured values for surface settlements for lake level at EL 2075 (Fig. 16 from the given data) and obtained values from the analysis (only for the upstream side, Fig. 28 from the analysis) it can be noticed that the diagram and the values of the settlements are very similar and also shows good accordance.

→ Up to now, with calibration of the numerical model, using some of the advanced features of the program SOFiSTiK, it was possible to explain some of the results obtained by the performed measurements. But, to explain the complete behaviour of the dam including the crack pattern, it is necessary to do additional improvement and calibration of the already complex numerical model with the measured data.