COMPARISON OF NUMERICAL MODELS ON RESEARCH OF STATE AT FIRST IMPOUNDING OF A ROCKFILL DAMS WITH AN ASPHALT CORE

Ljupcho Petkovski, Professor, PhD, Ljubomir Tanchev, Retired Professor, PhD, Stevcho Mitovski, Teaching Assistant, MSc

University Ss Cyril and Methodius, Civil Engineering Faculty – Skopje, Republic of Macedonia

ABSTRACT

The response of a rockfill dam with an asphalt core on static loading state is complex issue, in highest part not describable by physical laws, but is estimated with various numerical models. Including of models instead of laws implies that for analysis of same dam type, the models (based on different approximations) are not mutually excluded, but on the contrary, are contributing on better understanding of the prototype behaviour. The dam stress state at reservoir first impounding stage can be analysed by two numerical models. The basic difference between these models is the boundary condition on application of the hydrostatic pressure. The first model relies on the correspondence of the effective stresses from the displacements, approximated by superposition of three effects: softening of the submerged granular material, its alleviation and hydrostatic pressure on the core wall. The second model is based on the fact that effective stresses are difference of the total stresses and water pressure in the cavities of the rock material (according to hydrostatic laws). In this case, the change of the total stresses results from the additional load from increase of the volume weight and the external hydrostatic pressure along the upstream face, while the displacements are influenced by the rock material stiffness reduction due to decreasing of the effective stresses. Comparison of the two different numerical models on research of the stress state at embankment dams at reservoir first impounding stage is illustrated by results of the static analysis of dam Konsko in Republic of Macedonia – a rockfill dam with asphalt core with height of 80 m, currently in design stage.

1. Introduction – critical loading states at embankment dams

Static stability of the upstream and downstream slope of embankment dams is assessed under critical loading conditions during construction and service stage. Critical loading states at earth and rockfill dams for the upstream slope are states after dam construction and rapid drawdown of the water level, while for the downstream slope, state after dam construction and at full reservoir. The shape of the potential sliding surfaces at rockfill dams cannot be predicted. In the matter of fact, at these types of dams the slopes are stable even if they are constructed with slope equal to the repose slope of the rock material, because there is no occurrence of pore pressure [1]. The safety of the thin water impermeable element of artificial material, and by that and safety of a rockfill dam as a whole, depends on the deformations. Therefore for estimation of the stability of rockfill dams, the deformations in the specified critical loading states are of crucial meaning.

The state after dam construction is short lasting state of loading and necessary safety coefficient of the slope is F=1.3. This state is especially important at earth and earth-rock dams, where analysis with effective stresses in time domain should be applied, respectively, by following the realistic loading chronology, while at rockfill dams because there is no occurrence of pore pressure, the time domain is irrelevant, thus a more simplified analysis is applied, using total stresses. During dam construction, the loadings from machinery for compaction, as well as the loads from the upper layers, presses the medium air-water, which is filling the pores. In that way, during construction of embankment dams pore pressure develops in the cohesive material. The pore pressure value during construction is variable, because parallel with growth of the pore pressure, there is also a process of its dissipation [2]. The development of the pore pressure at earth materials during construction depends of the following factors: (a) humidity at embedment, (b) compressibility, which is in dependence of the loading, (c) water permeability, (d) construction schedule and (e) drainage structure type.

The state of full reservoir has two typical phases: (1) state after rapid impounding and (2) state of long-term full reservoir. The first phase is short lasting loading state of the structure with hydrostatic pressure along upstream face of the low permeable earth material (or water impermeable element of artificial material) and necessary slope safety coefficient is F=1.3. The second phase is long lasting loading state of the structure with established steady seepage flow through the earth material (or hydrostatic pressure at rock material) and necessary slope safety coefficient is F=1.5. The state of reservoir rapid impounding takes place before the seepage process through the earth cohesive material has started. Then the hydrostatic pressure from the upstream face acts as additional external load, that causes increase of the consolidation pore pressure, generated during dam construction. At long lasting of constant high headwater level, through the earth material (core of an earthfill dam and shell of an earth dam) it is established steady seepage flow. The material below the seepage line is fully saturated and exposed to seepage pore pressure. The pore pressure (h) at any point of the seepage zone is determined as difference of the potential (H) and elevation head (Z). The potential pressure is determined from the flow net equipotential lines, according to the interpolated hydrodynamic flow net. The state of full reservoir is especially important because it is pre-earthquake state with highest potential hazard on the downstream valley and it is used for confirmation of the dam seismic resistance [3, 4].

The state of rapid reservoir drawdown is short lasting loading state and accepted design criteria for the necessary safety coefficient is F=1.2÷1.3. Initial state for the stage of rapid emptying of the reservoir is stage of steady seepage, for which in the earth material was generated seepage pore pressure. Drawdown of the water level causes change in the pore pressure of the earth material. It should be noted that lowering of the reservoir level in period of few days to few weeks is rapid compared with the slow hydrodynamic process of water leaching from the saturated earth low permeable material. Similarly, as the stage during construction and after construction (before reservoir first impounding) occurs pore overpressure, that initiates consolidation process (manifested by decreasing of the pore pressure, increasing of the effective stresses and settlements). The difference of the consolidation processes at these two stages relies on the factor that causes the pore overpressure. During construction, the pore overpressure is caused by the loading of the upper layers (phase progressing of the embankment), while the change of the boundary hydraulic conditions at the rapid drawdown in the reservoir are generating the pore overpressure.

2. Contemporary methods on static analysis of embankment dams

Methods on static analysis of stress-deformation state of embankment dams are divided on classical and contemporary (also the term “advanced” is used) [5]. According to the classical methods, started to develop at beginning of XX century, the dam stability is evaluated by slope safety coefficients against sliding (Ks). Methods of this group are called Limit Equilibrium Methods and their common lack is that they don’t calculate the stress-deformation state.

The contemporary methods are composed of structure and foundation modelling (discretization) and application of numerical methods of finite differences or finite elements. The development and application of the contemporary numerical methods dates from the seventh decade of the XX century, connected with informatics development and with the pioneer work in this area by Zienkiewicz and Clough. By application of the contemporary methods, satisfactory results of the stress-deformation state are obtained and the dam stability is estimated through stability factor (Fs), depending on the realized stresses. The improvement of the computation technology (hardware and software) towards end of the XX century, conditioned these methods more and more to supersede classical methods in application in the engineering practice. So, today the classical methods are actual from educational point of view and eventually at some preliminary analysis of embankment dams.

The essence of application of methods for estimation of the dam stability, consisting of determination of safety coefficient (Ks) or stability factor (Fs) against slope sliding is to calculate the minimum value of these coefficients. The sliding surface of certain dam loading state, for which is obtained the minimum value of KS or FS is called critical sliding surface. It should be noted that there is no exact methodology, by which geometry of the critical sliding surface will be determined. Therefore, great number of potential sliding surfaces are analysed (200÷300), for the both slopes and for each typical loading state. The choice of number, shape and geometric parameters of the potential sliding surfaces in the engineering practice is still intuition problem for the designer of the embankment dam.

First attempts to solve the problem of determination of stress-deformation state of embankment dams by application of theory of elasticity and plasticity are made in the third decade of the XX century. These attempts had serious lacks due to the inability in the calculations to introduce the improper geometry, stage construction in layers and non-linear behaviour of local materials. Specified lacks are overcome by structure modelling (discretization with finite elements) and application of numerical (approximate) method of finite elements.

At prototype modelling is done physical approximation of the continuum that is being replaced by finite elements connected in nodes. From mathematical aspect, the application of finite element method comprises of numerical solving of the algebra equations. It is a great number of equations that can only be solved by use of calculation processors. Therefore, the appearance, development and application of finite element method at embankment dams emerges in same time with development of the computer technique. By analysis of the embankment dam with application of finite element method are determined stresses and deformations in the model, thus explaining the behaviour of prototype with complex geometry and heterogeneous composition.

3. Specific aspects of numerical models for static analysis of embankment dams

The powerful numerical finite element method (or finite differences method) is a strong tool applied by engineers for solving of the tasks in the field of continuum mechanics. However, we should have in consideration that application of contemporary numerical methods has purpose to explain certain phenomena and processes and to define more precisely the solution, but the designer must take responsibility on results interpretation. If the results do not match with values obtained by engineering reasoning, then it should be discarded and the mistake should be detected in order to be eliminated due to its repetition. The mistakes in application of numerical models can arise due to: (a) improper input data, (b) not enough precise discretization of the real system, (c) non-adequate constitutive law and (d) numerical problem (if iteration procedure is applied).

At modelling of embankment dams and conveying of numerical experiments on structure response in critical loading states, a general rule should be followed – the task should be solved in several stages, by gradual increase of the model complexity. In the static analysis of embankment dams the application of this general rule would mean to start with model of initial stage (simplified geometry, more zones with similar material parameters should be replaced with one zone, linear constitutive law, analysis with total stresses). When the result from the initial model will be obtained, a transition to the following stage can be done, by gradual increase of the complexity, and the most important – by interpretation of the variations of the results, caused by each level of increased complexity. In that way a simulation model for behaviour of the embankment dam at the end stage can be done, composed of complex geometry, heterogeneous medium, non-linear plastic constitutive law and analysis with effective stresses [6].

The embankment dams are structures constructed in layers. The size of the cumulative settlements in the layers (state after dam construction) depends primarily on: (a) load of the upper layers, and (b) distance of the stiff non-deformable foundation [7]. If we analyse state after construction of embankment dam on rock foundation, then engineering reasoning refers on the fact that zero vertical displacements will appear at: (a) foundation, that is non-deformable, and (b) dam crest, due to non-existence of loading on the highest layer. It means that for this loading state of the structure, it is obvious that maximal cumulative settlements will appear at intermediate part of the structure and the necessity of modelling with finite element method is to define more precisely the isolines values and distribution of the vertical settlements. However, if by finite element method dam stage construction in layers is not simulated, but is simulated instantaneously construction (typical for numerous civil engineering structures), then it will obtained that maximal settlements appear at dam crest – not corresponding with the engineering reasoning.

Independently that on first glance, the water seems as simple element in the nature, the theory indicated, and the practice confirmed, that the most complex phenomena at embankment dam behaviour are due to the water effect. For dam construction state, the water effect is actual only in case of zones of coherent earth material, so at rockfill dams for this state there is no structural problem influenced by this effect. Cohesive materials are embedded in layers at optimal humidity and by their compaction, the pores are fully filled with water. At loading increase (construction of upper layers) in first moment the full load is accepted by the pore water, and pore pressure develops. During time, by leaching of the pore water, its dissipation takes place. This slow hydrodynamic process of freeing of the pore pressure, followed by growth of the effective stresses and material settlement is called consolidation. Simulation of the development (raise and dissipation) of the pore pressure by application of the finite element method is carried out according to two approaches: analysis with total stresses and analysis with effective stresses [8]. According to first (simplified) approach, by application of total stresses or non-drained conditions, in low-permeable (cohesive) materials can be generated consolidation pore pressure, caused by change of the total stresses. The stress-deformation state and development (generation and dissipation) of the pore pressure, most realistically are determined by analysis with effective stresses that is when cohesive material is treated in drained conditions. In such a way, in structure response, according to the second approach, are included three components: (a) mechanic and elastic properties, (b) hydraulic properties (seepage coefficients and volume content of water), and (c) factor of time (schedule of dam construction).

At state of reservoir rapid impounding, the water effect in simplest way is manifested at embankment dams with facing of artificial material. It is a case when the water is outside of the dam body and acts as external pressure. The consequences of the water action in this case are: (a) increase of the normal stresses in upstream dam shell, thus conditioning growth of the material shear strength, contributing to dam stability, (b) in the dam body occurs horizontal displacements (directed downstream) and vertical displacements (maximal settlements in the intermediate part of facing and eventual raising with maximal value at downstream slope in the upper part), and (c) normal deflection in the facing (important for the estimation of the stability and dimensioning of the water impermeable element). Far more complex is the response of rockfill dams with internal core and earth-rock dams with cohesive core on the water effect. For the state of reservoir rapid impounding, the water rapidly fills the rock material cavities in the upstream dam shell and causes the following impacts: (a) softening of the submerged non-cohesive material in the upstream shell; as consequence of this additional settlements occurs, (b) alleviation of the permeable material (increase of the total stresses and pore pressure and decrease of the effective stresses), and (c) action of a force due to hydrostatic pressure along the upstream core face.

At full reservoir state, with established steady seepage flow through the earth material, the stress distribution depends on the following factors: dam composition and geometry, material parameters and dam type. At embankment dams stress transfer regularly appears. As a consequence, some elements are unloaded and other overloaded. If in certain zone the pore pressure value exceeds the value of the total normal stresses, a zone with negative effective pressure is obtained. It can cause hydraulic fracturing, usually manifested with occurrence of cracks in the cohesive material. The most common reason for occurrence of hydraulic fracturing is non-uniform settlement of material zones with different stiffness properties, where the softer material are "hanging" on the stiffer material (arch effect), thus transferring them a part of their stresses. Next potential reasons are violations and different slopes in the foundation of the cohesive material, that can cause non-uniform settlements and zone of bulk and unloaded material. According to Penman, the appearance of hydraulic fracturing at earth materials is possible if pore pressure is in the interval between maximal and minimal total normal stress $\sigma3_{max} < Pw < \sigma1_{max}$. According to other authors, on the occurrence of cracking of the cohesive material, beside the negative effective normal pressure, is necessary and adequate material non-homogeneity, referring on the statement that this phenomena is still not fully clarified.

4. Analysis of the impounding state at rockfill dams with an asphalt core

From the specified review on modelling of embankment dams under action of static loads it can be concluded that so far research on this issue introduced standard numerical models applied for typical loading states for different embankment dam types. Only in the case of reservoir impounding of rockfill dams with a core made from artificial material, there are two approaches, whose advantages and lacks are analysed in the further text. The stress state of the dam at first impounding stage can be analysed by two numerical models. The basic difference of these models is the boundary condition on application of the force of hydrostatic pressure. The comparison of the different numerical models on research of the stress state at rockfill dams at first impounding stage is illustrated with the results of the static analysis of dam Konsko (Fig. 1) in Republic Macedonia, a rockfill dam with asphaltic core with height of 80 m, elevation of rock foundation 470.0 m.a.s.l. and dam crest at 550.0 m.a.s.l., currently in design stage.

In the model no. 1, reservoir Konsko impounding up to normal water level is applied in three increments: up to elevation 496.0 m.a.s.l., 520.0 m.a.s.l. and 546.0 m.a.s.l. appropriately. The numerical analysis is done with application of the program Sofistik [9]. By the applied approach in this model, the effective stresses should be in correspondence with the displacements, obtained with superposition of three effects: softening of the granular material in the upstream shell, alleviation and force of hydrostatic pressure on the core. The first effect is simulated by reduction of the angle of internal friction (up to 1.5 degrees) and stiffness material parameters in the saturated zones, respectively with reduction of the elasticity modulus approximately for 20% and by application of additional load in 5% of the self weight of the zones above saturation line. On Fig. 2, the effective stresses σ'_1 after reservoir impounding are obtained with application of the second effect (alleviation simulated with Archimedes force directed upwards), but with reduced intensity and full horizontal hydrostatic pressure (the third effect), that leads to horizontal displacements along the asphalt core height, displayed on Fig. 3, and partial vertical displacements, displayed on Fig. 4.

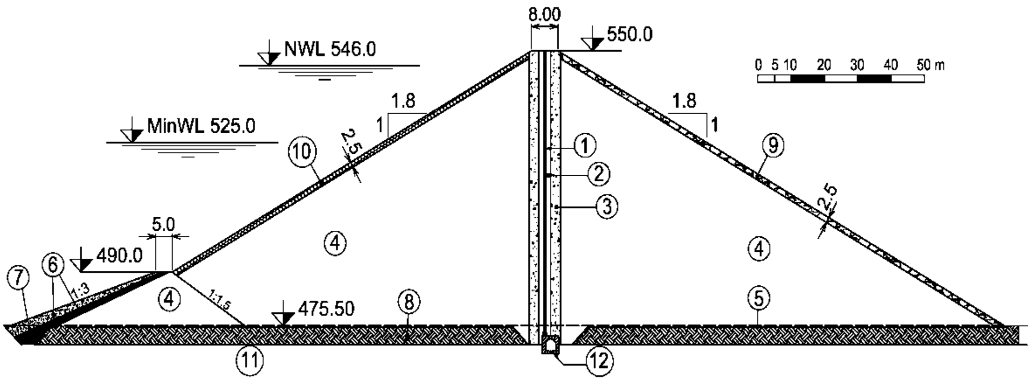

Figure 1 Cross section of dam Konsko; (1) asphalt core, (2) fine transition, (0-60) mm, (3) coarse transition, (0-250) mm, (4) rockfill, grains to 700 mm, (5) removed humus layer, (6) cofferdam filter from river gravel, (7) cofferdam clay facing, (8) river deposi

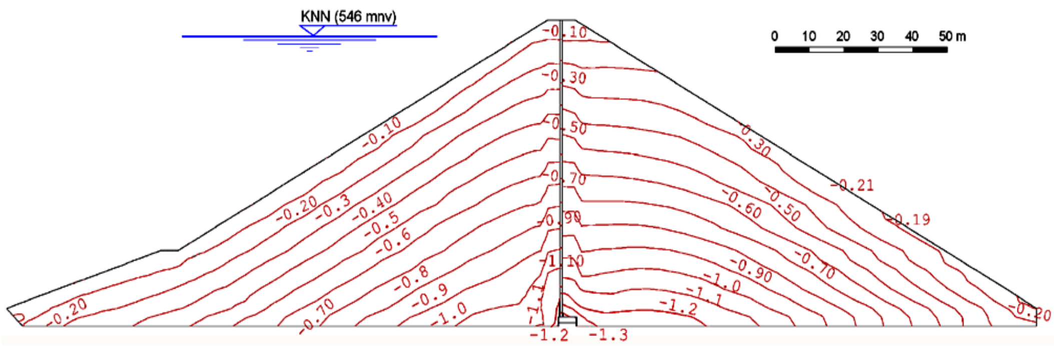

Figure 2 Isolines of maximal main effective normal stresses σ`1 at reservoir impounding: [MPa], equidistance 0.1 MPa; (–) denotes compression

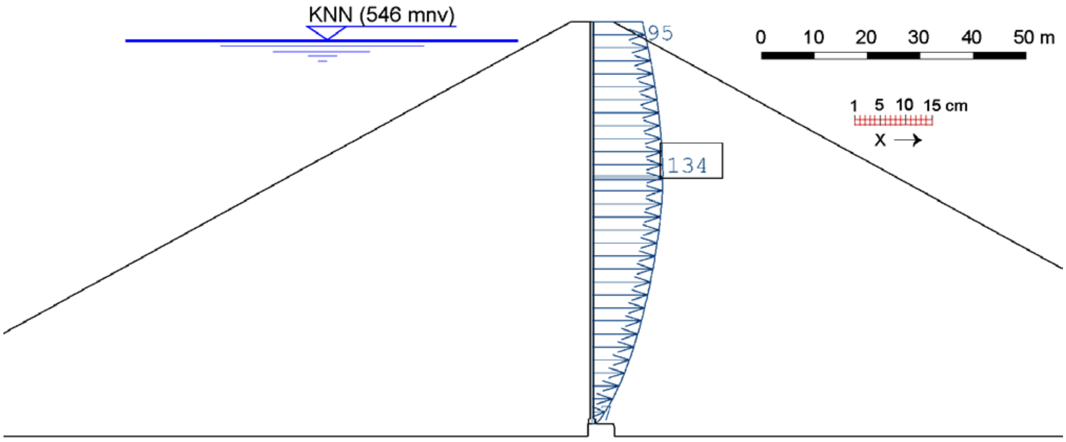

Figure 3 Horizontal displacements per asphalt core axis, at reservoir impounding; dimensions [mm], (at core axis, maximal value of 134 mm at 60% of the dam height, in the crest value of 95 mm)

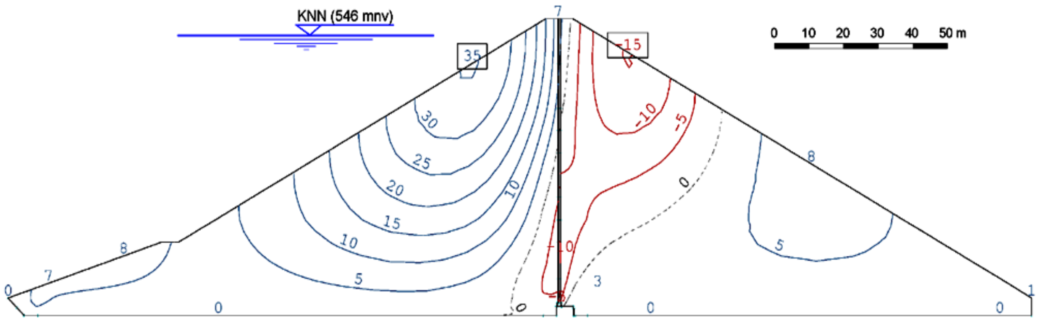

Figure 4 Isolines of partial vertical displacements in dam cross section, at reservoir impounding; dimensions [mm], equidistance of 5 mm; (+) displacements downward (settlements) ; (–) displacements upwards (raising)

In the model no. 2 (Fig.. 5), the impounding of reservoir Konsko is simulated in ten increments. The numerical analysis is done with application of program Sigma [10]. In this case, the change of the total stresses results from additional load of the increase of the volume weight and the external hydrostatic pressure along the upstream slope, and the displacements are influenced by rock material stiffness reduction due to reduction of the effective stresses. By this approach, the effective stresses (Fig. 6) are difference of the total stresses and water pressure (according on hydrostatic laws) in cavities of the submerged materials upstream of the core. The change in stresses results from the additional load by growth of the volume weight (from natural to saturated) and from material softening caused from elasticity modulus decrease at reduction of effective stresses (far bigger then value of 20% adopted in model no. 1).

The reason why in realistic case by saturation of the upstream shell (or by reduction of the effective stresses) does not occur rising (elastic response), displayed on Fig. 7, is superposition of at least three effects: (a) increased stiffness at unloading, (b) downstream displacements under action of the basic load – hydrostatic pressure (Fig. 8), and (c) occurrence of so called “collapse settlement” (previously term “material softening” was used). The third effect is manifested by settlement of the coarse material after submersion with water due to reduction of its strength properties, crushing of the coarse edges and etc., phenomena intensively researched in the last two decades [11, 12, 13]. The reason why in the model no. 2 are obtained higher downstream horizontal displacements of the core, compared with the upstream slope of the dam, although the hydrostatic load is applied along the upstream slope, are the reduced effective stresses in the upstream slope that conditioned stiffness reduction of the saturated materials. This effect does not appear at rockfill dams with facing and therefore at these type of dam the maximal horizontal displacements (by application of the same model) are obtained at the upstream dam slope.

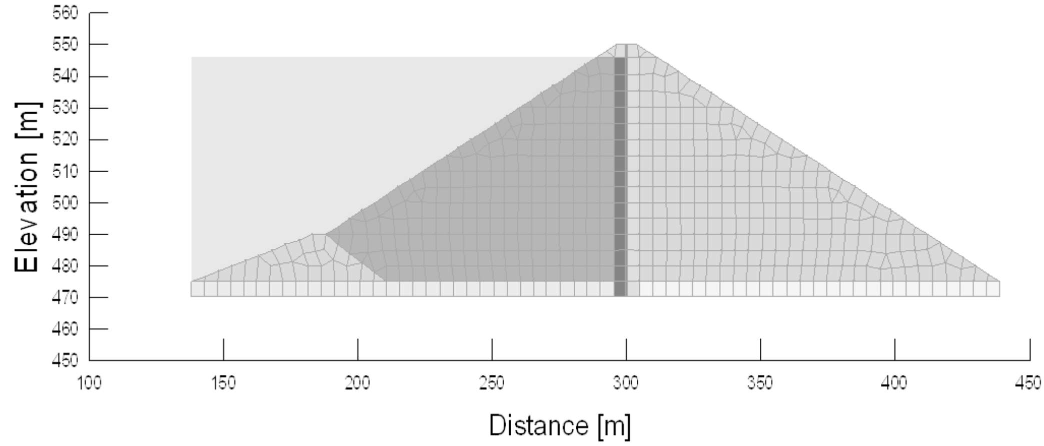

Figure 5 Model no. 2, discretization of dam Konsko by 534 elements and 588 nodes, first impounding at elevation 546 m.a.s.l.

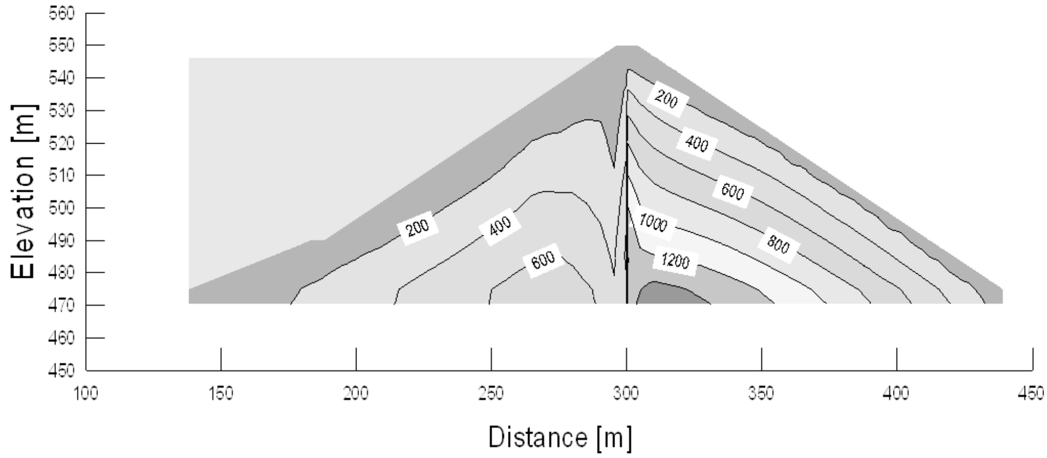

Figure 6 Maximal main effective stresses σ'1 after reservoir impounding up to level 546 m.a.s.l., value from 21.8 to 1,545.0 kPa

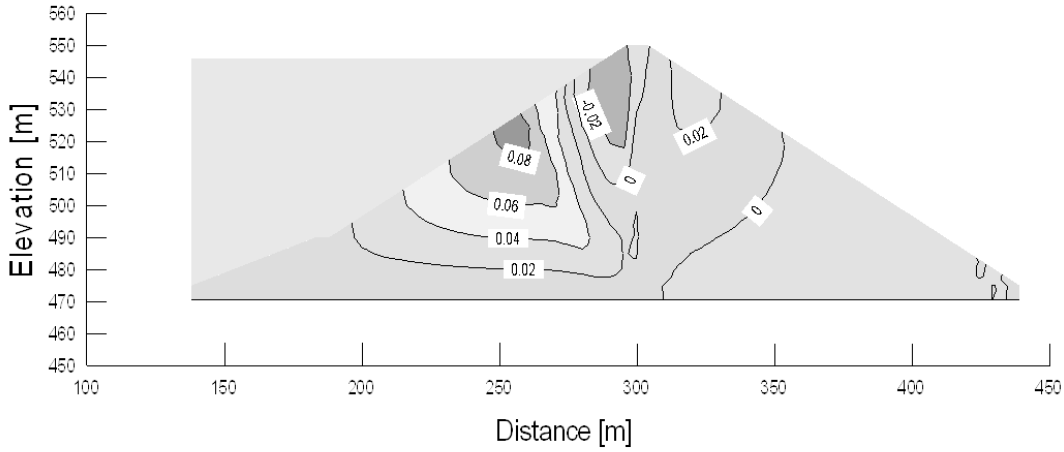

Figure 7 Vertical displacements at reservoir impounding up to level 546 m.a.s.l., from - 0.034 m (settlement) up to + 0.086 m (raising)

Picture 1

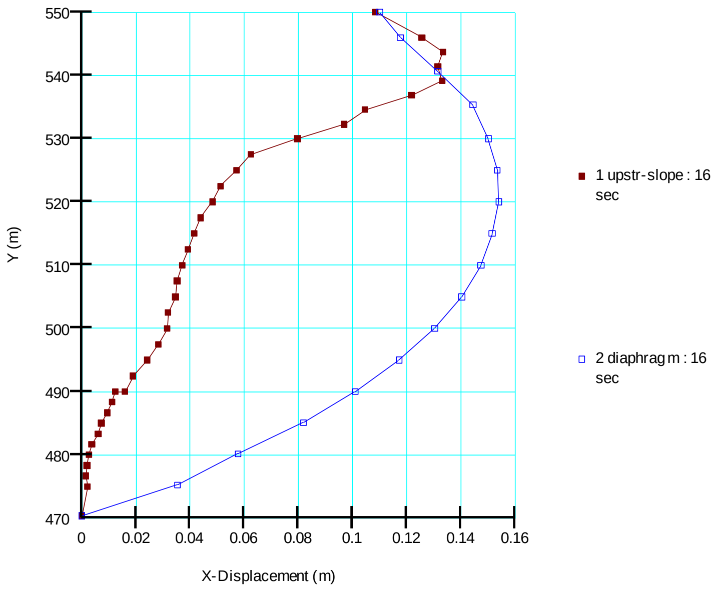

Figure 8 Horizontal displacements [m] along height of: (1) upstream slope and (2) asphalt core, at reservoir impounding up to elevation 546.0 m.a.s.l., (at core axis maximal value of 15.4 cm at 60% of the height, in the crest 11.0 cm)

5. Conclusion

The response of the embankment dam under action of static loading is complex issue, in largest amount not describable by physical laws, but is estimated by numerical models. Inclusion of models instead of laws implies that for analysis of same dam, the models (based on various approximations), should not be mutually excluded, but in contrary, they should contribute on better explanation of the prototype behaviour. By comparison of the results of the two applied models for behaviour of rockfill dam with asphalt core at first impounding stage, three following facts can be outlined. First, by analysis of the horizontal displacements along the core axis is noticed that in both models is obtained identically deformed shape, with maximal displacements around 0.2% of the dam height, located above the foundation at approximately 60% of the dam height, and reduced downstream displacement in the crest, estimated at 70% of the maximal horizontal displacement. Second, in case of the vertical displacements, in both models is obtained same maximal settlement, located in the upstream shell, at nearby of the reservoir normal water level, value of 0.05% of the dam height. And third, with model no. 2 is obtained more realistic display on distribution of the maximal main effective stresses, which is the basis for all future structural (static and dynamic) analysis of the dam. The degree of accuracy of the two different numerical models on distribution of the effective stresses in the upstream shell of rockfill dams with core wall at first impounding stage most appropriately can be confirmed by monitoring data for the total stresses and pore pressure at testing and service period of the dam.

6. Reference

[1] Tančev L., 2005. “Dams and appurtenant hydraulic structures”, Taylor & Frances, London, UK, 2005

[2] Numerical analysis of dams, ICOLD, 1994. Volume III "Evaluation of pore pressure and settlements of an embankment dam under static loadings", September, Paris, France

[3] Petkovski L., 2007. “Seismic Analysis of a Rock-filled Dam with Asphalitic Concrete Diaphragm“, 4th International Conference on Earthquake Geotechnical Engineering, 25-28 June 2007, Thessaloniki, Greece, paper #1261, CD-ROM

[4] Petkovski L., Ilievska F., 2010. “Comparison of Different Advanced Methods for Determination of Permanent Displacements of Tailings Dams in Earthquake Condition“,14th Europian Conference on Earthquake Engineering, 30-03 August 2010, Ohrid, R.Macedonia, paper #1511, CD-ROM

[5] Novak P., Moffat, Nalluri, Narayanan, 2001. "Hydraulic structures", London, UK

[6] Petkovski L., Tančev L., Mitovski S., 2007. "A contribution to the standardization of the modern approach to assessment of structural safety of embankment dams", 75th ICOLD Annual Meeting, International Symposium “Dam Safety Management, Role of State, Private Companies and Public in Designing, Constructing and Operation of Large Dams”, 24-29 June 2007, St.Petersbourg, Russia, Abstracts Proceedings p. 66, CD-ROM

[7] Petkovski L., Tančev L., 2004. "Basic principles of modern approaches to static analysis of earth filled dams", I Congress of the Macedonian Committee on Large Dams, October, Ohrid, Proceedings, p. 25-36

[8] Petkovski L., Tančev L., 1998. "Hydraulic and mechanical response of an earth dam during construction", VI International Symposium on water management and hydraulic engineering, Dubrovnik, Croatia, Proceedings Vol.2, p.239-248

[9] SOFiSTiK, 2010, Analysis Program, manual

[10] Geo-Slope SIGMA/W, 2012. "Stress/deformation analysis", GEO-SLOPE International Ltd., Calgary, Alberta, Canada

[11] Alonso, E.E., S. Olivella and N. M. Pinyol. 2005. A review of Beliche Dam, Géotechnique 55, No. 4, p.p. 267–285.

[12] Oldecop, L.A., Alonso, E.E. 2007. Theoretical investigation of the time-dependent behaviour of rockfill. Géotechnique, 57 (3), pp.289-301.

[13] Roosta M. R., and Alizadeh A. 2012. “Simulation of collapse settlement in rockfill material due to saturation”, International Journal of Civil Engineering, Vol. 10, no. 2, June 2012, pp.93-99.