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EDITORIAL - Preface to Volume 7 Issue 1 of the Scientific Journal of Civil Engineering (SJCE)

Todorka Samardzioska EDITOR – IN - CHIEF

Dear Readers,
Scientific Journal of Civil Engineering (SJCE) was established in December 2012. It is published bi-annually and is available online at the web site of the Faculty of Civil Engineering in Skopje (www.gf.ukim.edu.mk).

This Journal welcomes original works within the field of civil engineering, which includes: all the types of engineering structures and materials, water engineering, geo-technics, highway and railroad engineering, survey and geo-spatial engineering, buildings and environmental protection, construction management and many others. The Journal focuses on analysis, experimental work, theory, practice and computational studies in the fields.

The international editorial board encourages all researchers, practitioners and members of the academic community to submit papers and contribute for the development and maintenance of the quality of the SJCE journal.

As an editor of the Scientific Journal of Civil Engineering (SJCE), it is my pleasure to introduce the First Issue of VOLUME 7.

This year we celebrate significant jubilee – 80 years of dam engineering in Republic of Macedonia. Planning, design, construction and maintenance of dams are among the most responsible engineering works. Perhaps our planet has been unfairly named Earth, because 2/3 of it is covered with water. Water is the source of life, it creates and destroys. The hydraulic engineers are professional who contribute to its quality and rational use.

The first dam in R. Macedonia, arch dam Matka, at the canyon of Treska River, near city of Skopje, was built in 1938. Construction of this dam and its appurtenant structures marks the beginning of dam engineering in Republic of Macedonia. At present, in 2018, we can proudly state that dam constructors in R. Macedonia are worthy heirs and prolongers of the noble work of the first dam designers, dating from 1938. Confirmation of such statement is the fact that 45 dams with regional importance and over 110 small embankment dams with local importance are the key pillar of the present water economy infrastructure. With over 150 built dams of basically all types (embankment and concrete dams, gravity and arch dams) categorized as “large dams” by ICOLD criteria, Republic of Macedonia, proportionally to its size, is located right at the top of dam engineering in Europe.

This issue includes one introductory text and eight articles in the field of modelling, planning, construction and testing of different dams. The first text, written by the President of the Macedonian Committee on Large Dams, gives a chronological overview on the development of dam engineering in Republic of Macedonia. All other papers, originally presented at the 4th International Congress on Dams, in September 2017 in Struga, have been updated herein.
Wishing you nice summer and successful completion of the academic year!

Sincerely Yours,
Prof. Dr. Sc. Todorka Samardzioska
July, 2018

Impressum

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80 YEARS OF DAM ENGINEERING IN R MACEDONIA

This year we celebrate a significant jubilee – 80 Years of Dam Engineering in R Macedonia. Back in 1938, the first dam in R Macedonia was built – Matka arch dam. By construction of the dam and the appurtenant structures, at exit of the canyon of river Treska in nearby of the city of Skopje (figure 1), was created reservoir Matka (figure 2). By completion of the construction works on the hydropower plant located in the base of the dam, the power use of river Treska commenced. The Matka dam Design was prepared by academician Miladin M. Pecinar (1893-1973), figure 3, one of the pioneers in the development of contemporary "Hydraulic Engineering" in Yugoslavia. At this occasion, here below a brief overview of the rich biography of academician Pecinar is presented, which by his noble work has indebted in great deal the Civil Engineering profession in R Macedonia.

Figure 1. Canyon Matka on the river Treska in 1935, before dam construction

Figure 2. Downstream face of Matka dam, first dam in R Macedonia, built in 1938

Miladin Pecinar finished elementary school in the village of Ljubish, his birth place, and graduated in high school in Uzhice in 1912. He continued his education in Belgrade at the Technical Faculty, department of Civil Engineering. In March 1918, he took a break from the studies in Belgrade and has transferred to the Applied School for Civil Engineers in Rome. After the end of World War I, in 1919 Miladin continued its studies in Belgrade, and in graduated in 1921. The first years after graduation was employed at the Ministry of Civil Engineering, in the General directorate for water. In 1925 Pecinar created its own bureau for designing of water structures, where as he designed the following hydropower plants that were later constructed: Perukachko Vrelo on river Drina (1927), Chechevo (1929), Novi Pazar (1930), St. Andreja with the arch dam Matka (1938), Temshtica (1939), Crn Timok (1940) but also and other hydropower systems, that were not built. In that period, in several mandates, Pecinar was vice president of the Association of Yugoslavian engineers and architects.

After World War II, Pecinar was employed at the Ministry of Civil Engineering and the Yugoslav hydro meteorological facility. In 1946 he became president of the Yugoslav section of the International Commission on Large Dams (ICOLD). In 1948 he was elected as professor at Chair of Hydraulic structures at Civil Engineering Faculty within the Technical University in Belgrade. As most appreciated expert in field of hydrotechnics before, during and immediately after the World War II, Pecinar was elected as very first professor on the course “Hydraulic structures”. On the XI World Conference on Energy in 1957, he was general rapporteur on the topic of the complex use of water resources. In 1959 he was elected as writing member for the Serbian Academy of Science and Arts (SASA) and later in 1963 was elected as Academy full member. In 1960, he wrote the book “Hydraulic structures – dams”, containing very few formulas and plenty sketches and drawings, that happens to be the most favorable way for transferring of his great individual experience in designing and building of these most complex civil engineering structures. He got retired in 1963 as full professor at the Civil Engineering Faculty in Belgrade.

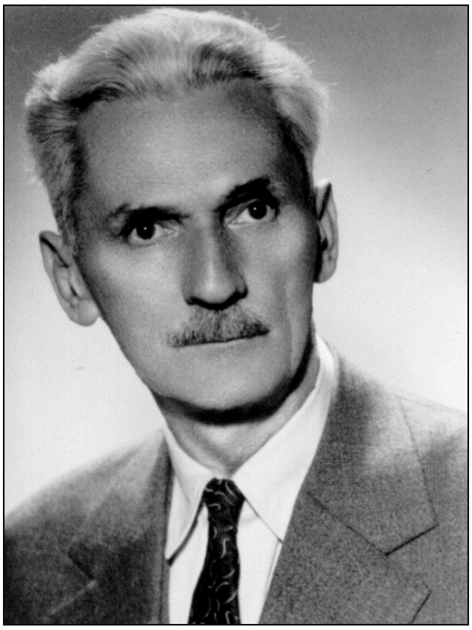

Figure 3. Academician Miladin M. Pecinar, (1893-1973)

It is here by to be noted that Pecinar for each hydropower plant that has designed, also designed and the appurtenant structures of the dam as well and hydropower derivations. Aside his ingenuity and enormous hydrotechnical talent, the greatness of academician Penicar was also in combining experts of different profiles thus creating both compatible and economically optimized structures, confirmed by Matka dam – a unique type of structure worldwide by many parameters. Accordingly, for the most important dam in R Macedonia – Matka dam, the best experts at that time from various fields were in charge. Namely, the static stability analysis was made by Miodrag Marinkovic (later a professor at the Civil Engineering Faculty in Belgrade), supervisor for concrete works was Djordje Lazarevic (later a professor at the Civil Engineering Faculty in Belgrade and full member of the Serbian Academy of Science and Art), Pavle Vukicevic was contractor of the dam (later a consultant at company Energoprojekt, Belgrade). Such approach by Pecinar resulted in building the Matka dam as "Penna Beff' type, second construction of such dam in Europe (Denia dam in Spain was the first). In addition, Matka dam was the highest arch dam in the Kingdom of Yugoslavia, but also and the boldest arch dam in Yugoslavia in XX century, with slender coefficient equal to 0.054.

At present, in 2018, we can proudly state that dam constructors in R Macedonia are worthy heirs and prolongers of the noble work of academician Pecinar, dating from 1938. Confirmation of such statement is the fact that the key pillar of the our present water economy infrastructure are 45 dams with regional importance, 4 of which are tailings dams, and over 110 small fill (embankment) dams with local importance. With over 150 built dams of basically all types (embankment and concrete dams, gravity and arch dams) categorized as "large dams" by ICOLD criteria, shows that R Macedonia, proportionally to its size, its located right at the top of dam engineering in Europe. It should be noted that the most significant water structures are designed and built by domestic companies, which is the best proof that in this period of eight decades was created well-known and respected Macedonian hydrotechnical school. The central spot in the progress and improvement of the widely respected Macedonian school for Dam Engineering holds the Chair of Hydraulic Structures at Faculty of Civil Engineering in Skopje, created by establishment of the Technical Faculty in Skopje in 1949. The greatest merits for the development of the Chair of Hydraulic Structures belong to the following: Chair founder, Prof. Bratislav Subanovic, that lectured the first classes in courses "Utilization of Water Power" and "Hydraulic Structures" in the so far away 1950 and was the head of the Chair until 1965; his heirs, Prof. Mihajlo Serafimovski (retired since 1987), leading by great number of applicative works and designs, Prof. Nikola Durned (retired since 2001) and Prof. Dr. Ljubomir Tanchev (retired since 2010) – a person with the greatest scientific contribution to the Chair and a professor that I had the privilege to be my teacher in the "world of dams".
According to the dynamics of large dams construction in R Macedonia, regarding the 45 hydro-systems with regional importance (figure 4), we can divide three periods with different intensity of construction of dams: (1) the period of 60-ies of XX century or "gold period" for dam construction, (2) the last decade of XX century – period of great stagnation, in which period are built very few small fill dams and (3) first two decades of XXI century – period of intensifying dam construction by various and also new dam types. By the chart on figure 4, it can be stated that R Macedonia has a solid tradition and continuity in designing and building dams that is required for proper knowledge transfer from one generation of hydrotechnics engineers to another and maintenance of high quality of work of the engineering companies in the field of Dam Engineering.

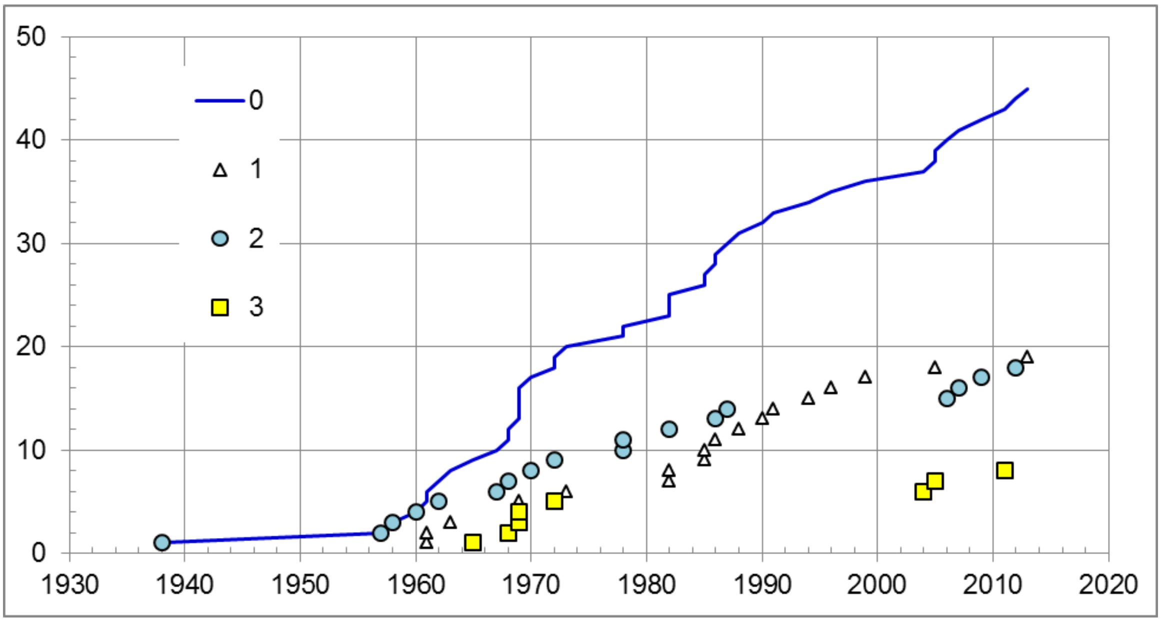

Figure 4. Dynamics of construction of large dams in R Macedonia, for 45 hydro-systems of regional importance. (0) in total, (1) low, H < 30 m, (2) medium, H < 80 m, (3) high, H < 150 m

The latest built dam in R Macedonia is double curved arch dam St. Petka on river Treska, with structural height of 64 meters, constructed in 2012, figure 5.

Figure 5. St. Petka arch dam on river Treska, constructed in 2012

In the past period in R Macedonia practically all dam types are built, in correlation that is common worldwide. According to the material type for dam construction (figure 6), 11 are concrete (24.4%) and 34 are embankment dams. Regarding the concrete dams, according to the structure, 8 of them are arch dams, 2 are massive dams and 1 is buttress dam (Prilep dam). In case of fill dams, according to the local material, equally are constructed - 17 earthfill and 17 rockfill dams. From the rockfill dams, mostly represented are earth-rock dams (impermeable element of natural clay material) and only 2 are rockfill dams (with artificial impermeable element). Such dams are Loshana dam, constructed in 2006 (with geomembrane facing) – first of such type in ex-Yugoslavia, and Knezhevo dam (figure 7), built in 2011 (with asphaltic core), first of such type in southeastern Europe. These two dam cases in most eclectant manner show the boldness and inventiveness of the present generation of hydrotechnics professionals in R Macedonia.

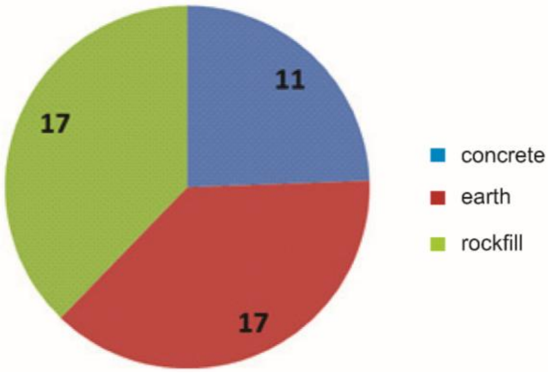

Figure 6. Division of large and important dams built in R Macedonia, according to the material type for construction

Figure 7. Rockfill dam Knezevo with asphalt core, on river Zletovska, built in 2011 with structural height of 82 m

Figure 8. Upstream view of Kozjak dam, highest earth-rock dam in R Macedonia

The highest dam creating water reservoir in R Macedonia is earth-rock dam Kozjak on river Treska, built in 2006 with structural height of 126.0 m, figure 8.However, the highest fill dam in R Macedonia is tailings dam Topolnica of mine Buchim, Radovish, completed in 2015, with crest-to-downstream-toe height of 141.2 m. According to the structural height of the important and large dams in R Macedonia, figure 9, we have mostly low dams (less than 30 m) and medium high dams (30 to 80 m) – all in all, total of 19 and 18 respectively, and high dams (80 to 150 m) are total of 8 (or 17.8%), while extremely high dam (height more than 150 m) is not yet constructed in R Macedonia.

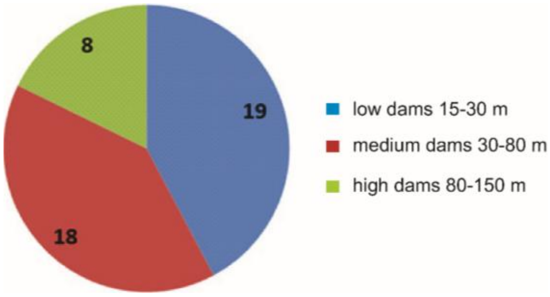

Figure 9. Division of large and important dams in R Macedonia, according to height


Become a student of the Faculty of Civil Engineering and a part of the impetus that creates and build the world! Step in the world of the successful people, because even the longest roads start with the first step. You will spend a part of your youth with us, and the youth is expensive to be misspent in vain. Your choice is an exceptional profession, for people who do believe in themselves, profession that requires prompt and courageous decisions. This profession will provide you with great privileges: your actions will remain an eternal record in the space and in the time being.
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COMPARISON ANALYSIS OF THE BEHAVIOR OF ROCK-FILL DAMS WITH CLAY CORE UNDER THE EFFECT OF THE VARIATION OF WATER LEVEL IN THE RESERVOIR

By application of advanced numerical methods, the behaviour of embankment dams is successfully analysed, almost for all typical states of static loading. The occurrences, not fully clarified during the static analysis of some types of embankment dams, are caused by the water effect. Such occurrences include: (a) hydraulic fracturing phenomenon of the coherent material (earth dams and earth-rock dams) during full reservoir with established steady seepage and (b) appearance of softening and weakening (or the latest term collapse settlement) of the rock material at the upstream dam shell in case of rock-fill dams with core, during the rapid filling of the reservoir. The topic of the research, whose results are included in the paper, is clarification of the stress-strain state for rock-fill dams with central waterproof element at variation of the water level in the reservoir. This issue, stated in the dam engineering in 70's of the last century, was analysed in several occasions, but the general conclusion is that the effect of water saturation of the rock material is not fully explained. In this paper are presented results from the comparison analysis of the variation of the water level of the reservoir, formed by construction of Kozjak Dam, on the river Treska, a tributary of the river Vardar. It is a rock-fill dam with slightly inclined clay core, with structural height of 130.0 m, the highest dam in the Republic of Macedonia. The first filling of the reservoir took place in 2003-2004 and the dam behaviour was monitored by surveying methods and by installed instruments in the dam body.

Keywords: rock-fill dam, clay core, water level

1. INTRODUCTION

By applying contemporary numerical methods, based on Finite Element Method (FEM), the mathematical models for successful research of the behavior of fill dams are developed, almost for any typical state of static loading. The only occurrences that are yet to be fully clarified at static analysis of fill dams are those caused by the water effect. These include the possible occurrence of hydraulic fracturing phenomenon in coherent materials (within earth and earth rock dams) at the stage of full reservoir through established steady seepage flow, and the possible softening and weakening (or collapse settlement) of the rock material in the upstream dam shell in case of rock-fill dams with core during the stage of reservoir rapid filling.

The objective of this research is to clarify the stress-deformation state of rock-fill dams with central impermeable element during variation of the water level in the reservoir. This issue, raised in dam engineering since the 1970s, was investigated on several occasions, but all attempts ended by stating that the modelling was not properly done and according to the latest knowledge in this field. The general conclusion is that the saturation effect of the rock material is still present. The aim of this paper is to contribute to the selection of most favourable numerical models for simulation of the response of embankment dams with shells of rock material and core of coherent material under the effect of variation of the water level in the reservoir. To accomplish the specified goal, a comparison of few models has been done, differing by the constitutional laws for the stress-strain dependence. The contribution of this research refers to the recommendations of more advanced numerical model for study of the stress-deformations state in case of rock-fill dams with core during the stage of rapid filling of the reservoir.

2. THE WATER EFFECT IN CASE OF STATIC RESPONSE OF EARTH ROCK DAMS WITH CORE

The research on water effects on earth-rock dams subjected to static loading was limited until the 1970s, due to the application of the classical methods, based on Theory of elasticity and Theory of plasticity. Therefore, these methods were gradually replaced by advanced methods – Finite element method (FEM), developed in the last three decades of the twentieth century, thanks to the pioneering work in this field by Zienkiewicz and Clough (ICOLD 1978). The improvement of the calculation technique (hardware and software) in the beginning of the 21th century has enabled the advanced numerical methods, based on the FEM, to fully replace the classical methods in the engineering practice. The focus of the current research is to clarify the response of rock-fill dams with earth core under the effect of variation of water level in the reservoir. The variation of water level causes various effects on the stress-strain state of the fill dams. In the following paragraphs, a short overview of water effect for typical loading states of the structure during construction and in service period is given.

During rock-fill dam construction, water effects are relevant for the zones of coherent material, and can be modelled using FEM. The coherent materials are applied in layers with optimal humidity, meaning that the pores are fully filled with water. By increasing the load (through placing of the upper layers) at the first moment, the full load is accepted by the pore water and pore pressure occurs. During time, a pore pressure dissipation occurs by leaching of the water. Such slow hydrodynamic process of pore pressure release, followed by an increase of the effective stresses and material settlement is called consolidation. The FE simulation of the development (increase and dissipation) of the pore pressure is performed according to two concepts: analysis of total stresses and analysis of effective stresses. According to the first (simplified) method, a consolidation pore pressure, caused by the change of the total stresses, can be generated by application of total stresses or in non-drained conditions in case of low permeable coherent materials. The stress-deformation state, as well as the development (generation and dissipation) of the pore pressure, are most realistically determined by analysis of the effective stresses, meaning that the coherent material is treated in drained conditions. In the response of the structure according to the second approach, three components are included: (1) mechanical and elastic properties, (2) hydraulic properties (seepage coefficient and volume content of the water) and (3) time factor, or dynamic of dam construction.

For the state of reservoir filling, the water effect is most simply manifested in case of rock-fill dams with facing made of artificial material. This is a case where the water is outside of the dam body and acts as external pressure. Far more complex is the response of the rock-fill dams with diaphragm and earth rock dams with core of coherent material, due to the water effect. At stage of rapid filling of the reservoir, the water rapidly fills the pores of the rock material in the upstream dam shell and causes the following effects: (1) softening of the submerged non-coherent material, causing additional settlements, (2) alleviation of the permeable material (increase of the total stresses and pore pressure and decrease of the effective stresses) and (3) action of force due to the hydrostatic pressure along the upstream face of the core/diaphragm.

Similar water effects, but with opposite action, occur in the state of rapid drawdown of the water level in the reservoir. The initial state for the stage of rapid drawdown of the reservoir is the state of steady seepage, for which pore pressure generated seepage in the earth material. Lowering of the water level in the reservoir causes change of the pore pressure in the earth material. It should be noted that the lowering of the reservoir water level during a period of few days to few weeks is rapid, compared to the time consuming hydrodynamic process of slow leaching of the water from the saturated low permeable earth material. Similar to the stage after dam construction (before first filling of the reservoir), at the stage of rapid drawdown of the reservoir excess pore pressure occurs, thus initiating consolidation process (manifested by decrease of the pore pressure, raise of the effective stresses and settlements). The difference in the consolidation processes for both stages consists in the factor that causes the excess pore pressure. During construction, the excess pore pressure is caused by the loading of the upper layers, while at the stage of reservoir rapid drawdown, the variation of the boundary hydraulic conditions generates the excess pore pressure.

For the state of full reservoir, with established steady seepage through the earth material, the stresses distribution depends on the following factors: dam composition and geometry, material parameters and dam type. In case of fill dams, there is regular occurrence of stress transfer, thus enabling unloading of some elements and overloading of others. If in a particular zone, the value of the pore pressure exceeds the value of the total normal stresses, a zone with negative effective pressure occurs. It can lead to hydraulic fracturing, manifested by appearance of fissures in the coherent material. Most common reason for occurrence of hydraulic fracturing is the non-uniform settlement of zones by materials of different stiffness properties, where the softer material "hangs" on the stiffer material and thus transferring part of its stresses. The next potential reason is distinct unevenness and different inclinations in the foundation of the coherent material, causing uneven settlements and zone of shattered and unloaded material. According to Penman researches from 1975, the occurrence of the hydraulic fracturing at earth materials is possible if the pore pressure in within the interval of maximal and minimal total normal stress. According to other authors, for occurrence of the fissuring of coherent material, beside the negative effective normal pressure, an appropriate non-homogeneity of the material is required, thus referring to the finding that such phenomena is not fully clarified yet.

From the above specified short overview of the water effect at the response of dams for typical loading states in static conditions, it can be concluded that for research of the fill dams behaviour (during construction and in service period) standard numerical models are distinguished. Such models are successfully applied in case of various dam types and almost for all loading states [Petkovski L., Tanchev L., Mitovski S., 2007]. For most of the dams (fill and concrete) for the reservoir filling state are obtained displacements in downstream direction, which in fact is intuitively expected by the researchers. It was one of the reasons why in the past were adopted minor curving of the dam crest in layout, placing the convex face towards the reservoir, even in the case of fill dams, even though they are gravitational structures. Such solutions for earth-rock dams with central or slightly inclined core in cross section and with curved shape in layout with convex face towards the reservoir were very popular in the middle of the 20th century. Most likely, the designer's prediction was that the downstream horizontal displacement, under influence of the reservoir filling would cause additional compaction of the local material that would provide improved safety of the fill dams.

By more precise measurements of the horizontal displacements in the interior part of the dam, for the state of reservoir first filling, in case of rock-fill dams with core/diaphragm is registered unusual occurrence in the cross section axis. Such occurrence, registered by inclinometer, was noted by Marshal and Ramirez in 1967, and referred on the first filling of the reservoir at El-Infiernillo Dam, Mexico [ICOLD, 1986, Bulletin 53]. Such occurrence is manifested by upstream horizontal displacement of the dam during first filling of the reservoir up to 50% of the height. The further raising of the water level in the reservoir causes downstream displacement in the lower part up to 30% of the dam height, while the crest is still displaced upstream. By reaching the normal water level in the reservoir, the dam axis is displaced in downstream direction, with maximal intensity at 50% of the dam height.

Such unusual bidirectional horizontal displacement at the crest in dependence of the water level in the reservoir results from the complex water effect in the upstream shell of the rock-fill dams with core. The first attempt for analysis of the displacement phenomena at rock-fill dams with core during first filling was by Nobari and Duncan in 1972, where following components were superposed: water load, softening and weakening of the fill material. The softening of the local material by the saturation process is confirmed with three-axial testing for dams Oroville (USA) and Beliche, (Portugal), and in the numerical multistage experiment should be modelled by variation of the stiffness and strength parameters.

The further research (modelling and measuring) of the non-usual bidirectional horizontal displacements of rock-fill dams with core, for the stage of first filling, systemized in the publications of the most authoritative institution in dam engineering worldwide [ICOLD, 1988, Bulletin 94], point out that such phenomena is not yet clarified in full. Such issue, raised in the dam engineering in the 70-ties of the 20th century was analyzed in several occasions [ICOLD, 1994, Numerical analysis of dams, Volume III] and it was stated that it is not modelled properly [ICOLD, 2001, Bulletin 122]. In the latest publications, the terms “softening and weakening” are most often replaced by “collapse settlement” [ICOLD, 2013, Bulletin 155], but the effect of the saturation of the rock material is still present.

3. ANALYSIS OF THE STATE OF RAPID FILLING OF EARTH-ROCK DAMS WITH CORE

The state of stress in the dam body for stage of first filling of the reservoir can be analysed by numerical models, differing by: (a) constitutive law for the dependence stress-strain and (b) the boundary condition on where to apply the hydrostatic pressure – the upstream slope of the dam or upstream face of the waterproof element. If the boundary condition is upstream slope of the dam, a far more realistic picture is obtained for the distribution of the maximal normal stresses (total and effective), that is the base for all further structural (static and dynamic) analyses of the fill dam. Therefore, the purpose of the current research is to contribute at the choice of most favourable numerical model for simulation of the stress-deformation state in case of earth-rock dams with shells of rock material and central waterproof element, under influence of the reservoir filling. In order to achieve the specified aim a comparison of the specified particular models is done.

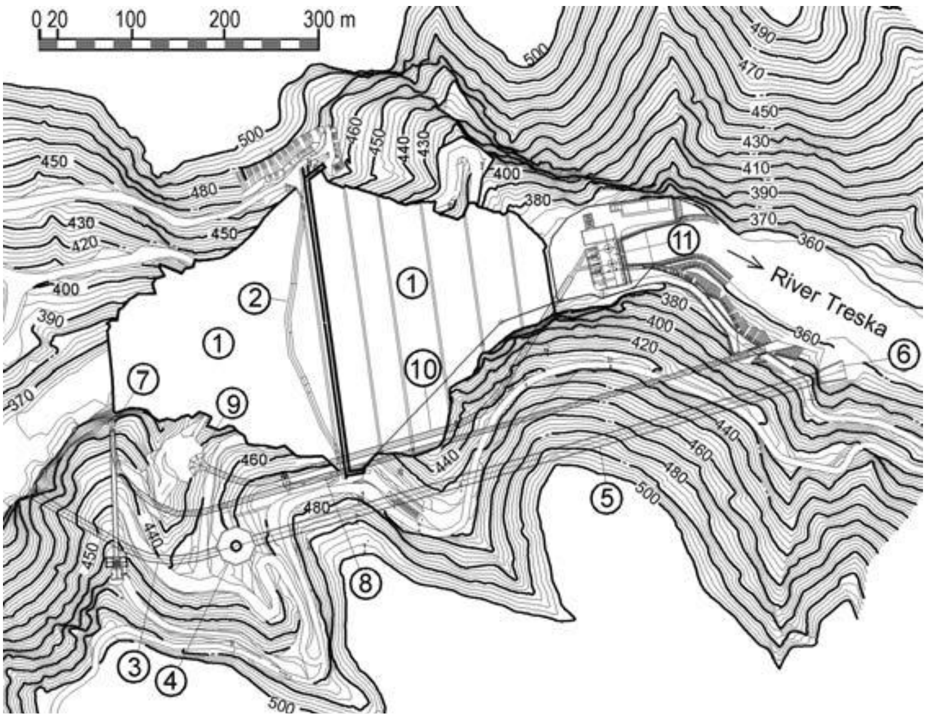

Figure 1. Layout of the hydraulic scheme with Kozjak Dam. (1) Dam body; (2) grouting gallery; (3) diversion tunnel; (4) morning glory (shaft) spillway; (5) spillway tunnel; (6) flip bucket; (7) intake structure of the bottom outlet; (8) bottom outlet tunnel; (9) intake structure of the power plant; (10) water supply tunnel for the power plant; (11) tail raise channel

The comparison of the different numerical models for research of the stress state in case of fill dams during the stage of reservoir first filling is illustrated by the results from static analysis of dam Kozjak, on river Treska, right tributary of the river Vardar, Republic of Macedonia (Fig. 1 and 2). It is a rock-fill dam with slightly inclined clay core, with structural height of 130 m, the highest dam in Macedonia. The dam was constructed in 2000, the first filling of the reservoir took place in 2003-2004 and the dam behaviour was monitored by surveying methods and by installed instruments in the dam body.

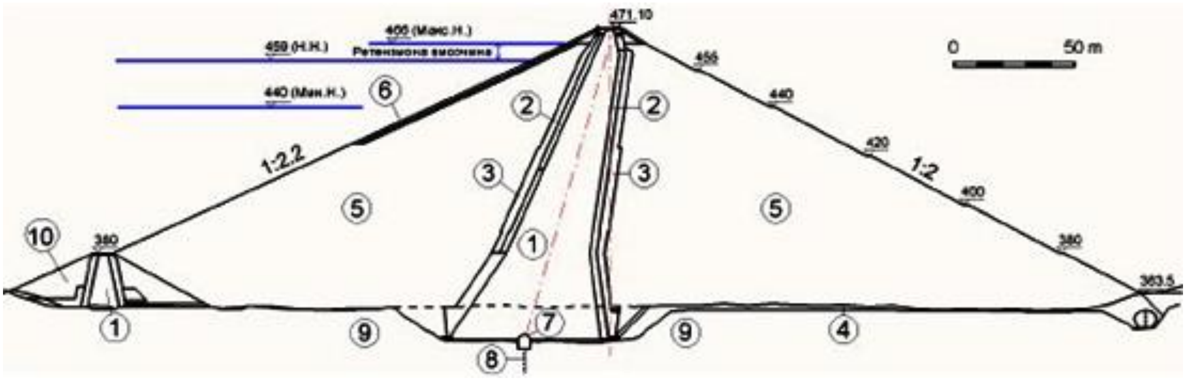

Figure 2. Main cross-section of Kozjak Dam, section no. 17. (1) Clay core; (2) first transition zone; (3) second transition zone; (4) river deposit; (5) rock-fill shells (limestone); (6) arranged slope protective stones; (7) grouting gallery; (8) grout curtain; (9) rock foundation (limestone); (10) gravel in the upstream cofferdam

4. NUMERICAL MODEL CALIBRATION

For calibration of the numerical model for analysis of the state of first filling are used data from technical monitoring of the dam for construction state and consolidation process of the dam. The respective strength parameters are adopted by using numerous data from control laboratorial testing of the placed local materials. The elastic parameters, by variable elasticity modulus with change of the effective stresses, are set by meeting the criteria on minimization of the difference between the measured and simulated values. The following key measured values are registered within the dam monitoring, directly after dam construction: maximal settlement of 1.3 m, maximal pore pressure at dam core foundation (400-700) kPa and crest settlement, caused by consolidation, shortly before the first filling, of 0.1 m. The typical cross section no. 17, representative for plane static analysis, is discretized with grid of 947 finite elements, connected in 947 nodes (Fig. 3).

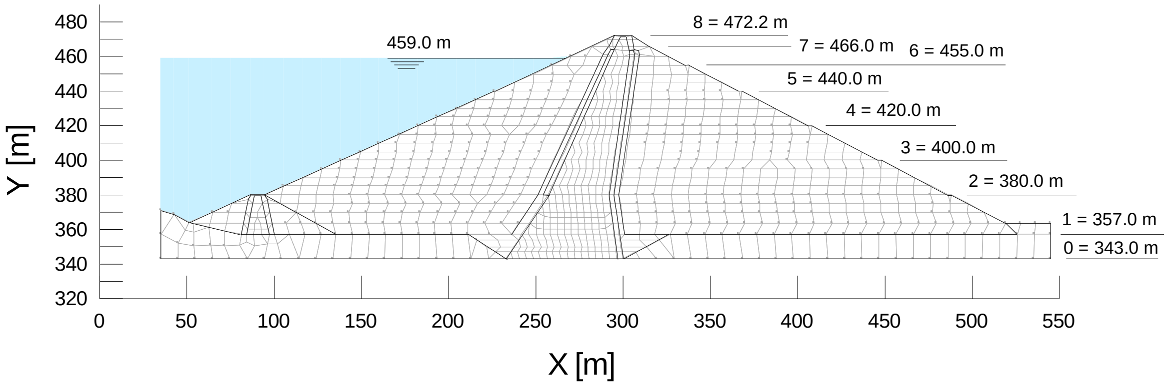

Figure 3. Numerical model for cross section no. 17, discretized by finite elements

For obtaining of the initial state before reservoir filling, a model with effective stresses is applied, by coupling mechanical and hydraulic response of the structure in real domain. The state after dam construction is simulated in 28 loading increments for period of 920 days (Fig. 4).

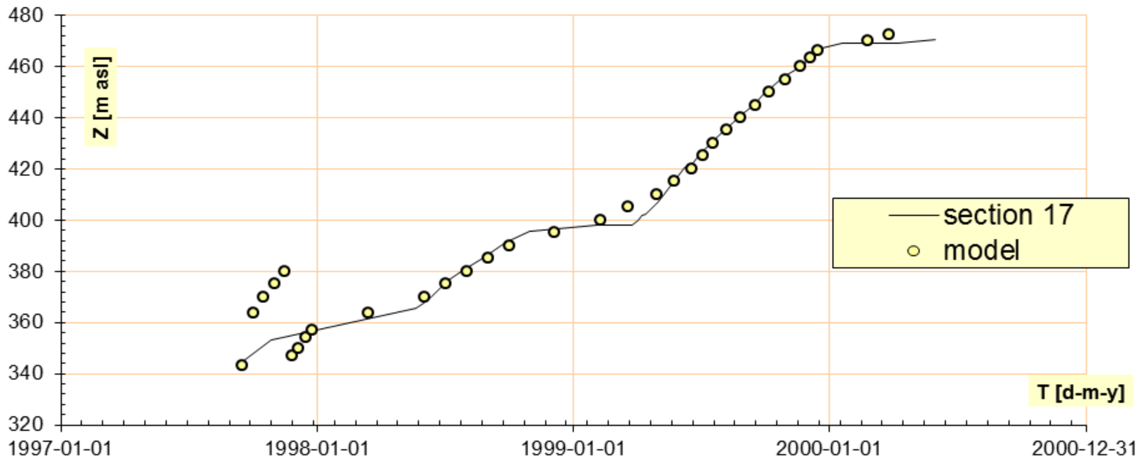

Figure 4. Dam construction simulation

The consolidation state is simulated in one loading increment, by 14 calculation exponential steps, for period of 1,043 days. From the distribution of: settlements after construction (Fig. 5), pore pressure after construction (Fig. 6) and incremental consolidation settlements (Fig. 7), it can be concluded that the model calibration is properly done.

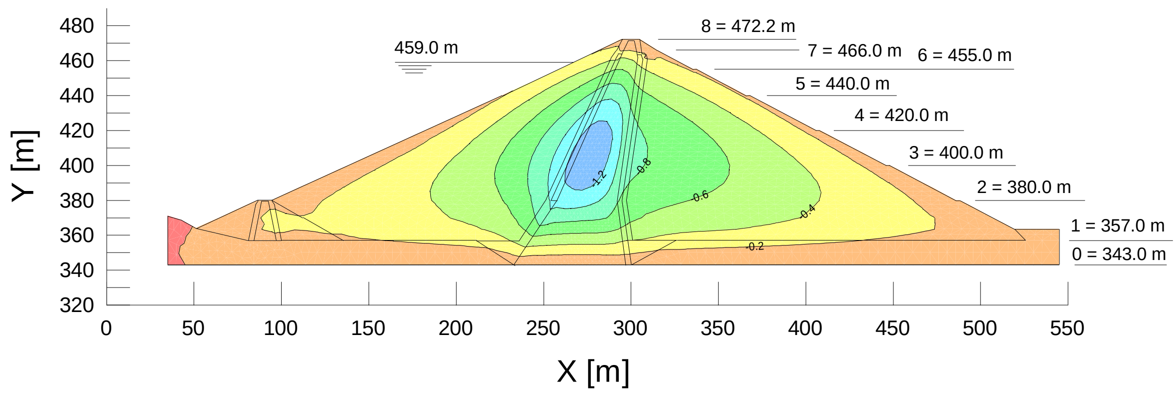

Figure 5. State after dam construction, Ydisplacement (-1.338) - (+0.012) m

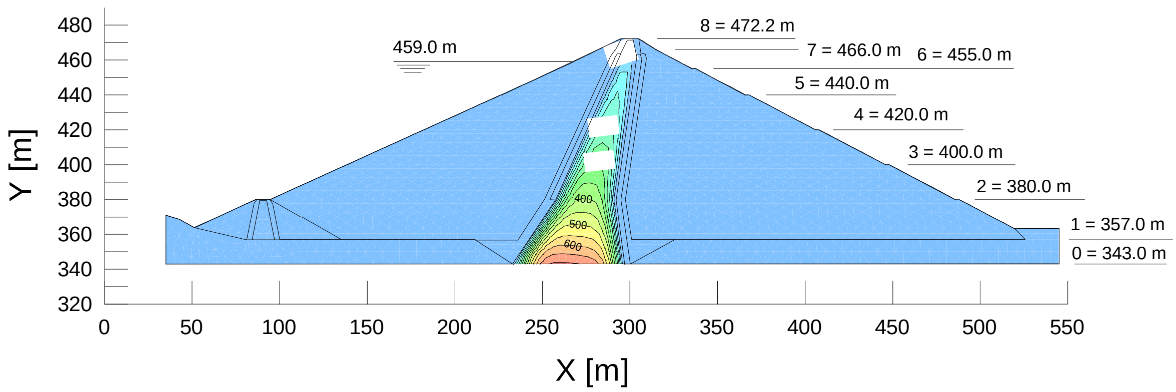

Figure 6. State after dam construction, Pore water pressure, (-0.0) - (+710.5) kPa

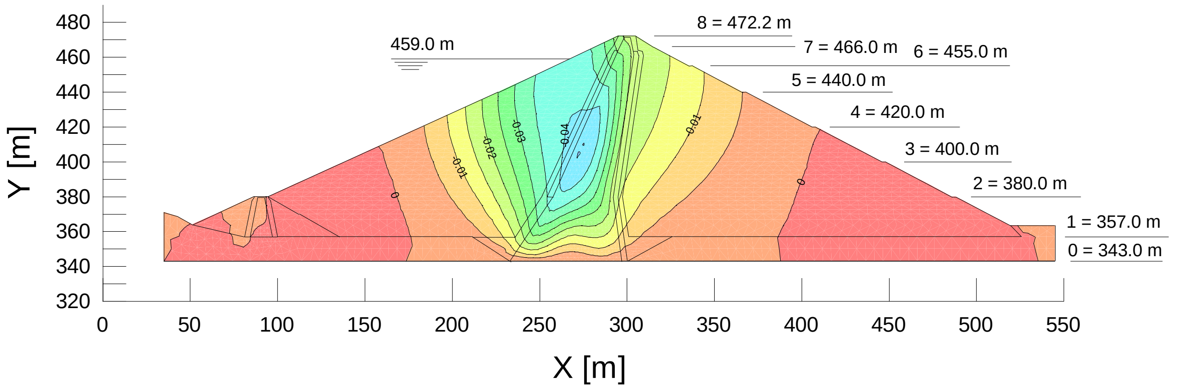

Figure 7. State upon consolidation, dY displacement (-0.045) - (+0.001) m

5. ANALYSIS OF THE STATE OF FIRST FILLING OF THE RESERVOIR

By using the calibrated model, the state of first filling is simulated in period of 516 days, by linear increase of 15 calculation steps of the water level from 343.0 m asl to 459.0 m asl (Fig. 8). The generation and dissipation of the pore pressure in the three typical stages (Fig. 9), by total lasting of 2,479 days, mostly matches with the measured values, that is contributing the conclusion regarding the regularity of the numerical experiment.

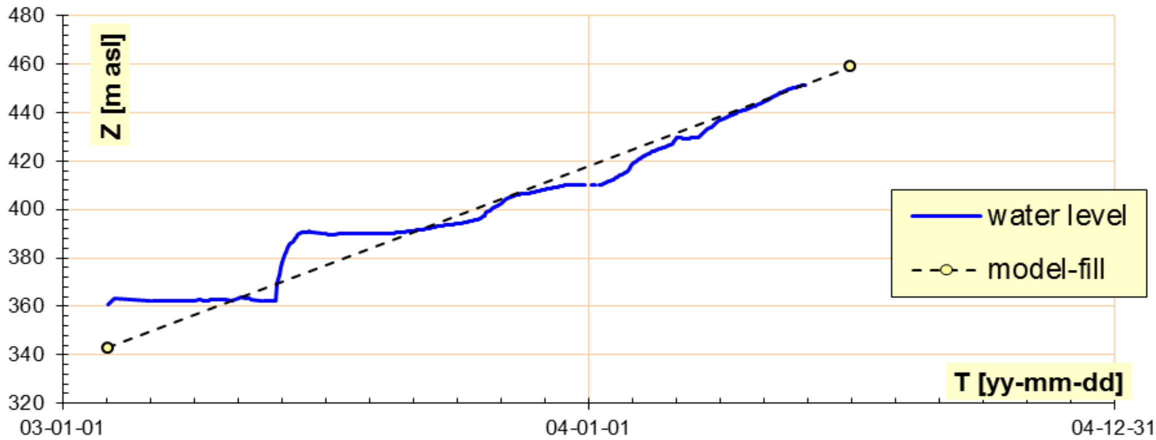

Figure 8. Simulation of the reservoir first filling

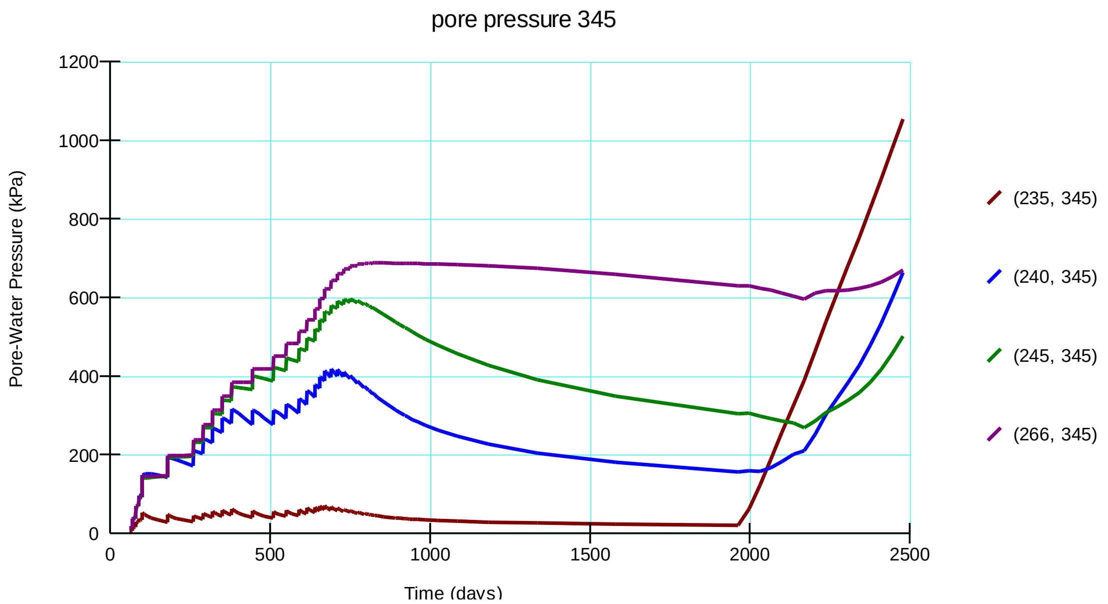

Figure 9. Generation and dissipation of the pore pressure at core foundation, at elevation 345.0 m asl, with coordinates X = {235, 240, 245, 266} m

The unusual bidirectional displacements of the central waterproof element, in dependence of the water level, are displayed for the upstream and downstream face of the core, Fig. 10 and Fig. 11. By the pattern and distribution of the horizontal displacements, it can be concluded that the response of the earth rock dam during first filling is properly simulated, and that the state of the effective stresses upon reservoir filling (Fig. 12) can be successfully applied for analyses of the following static and dynamic loadings of the structure.

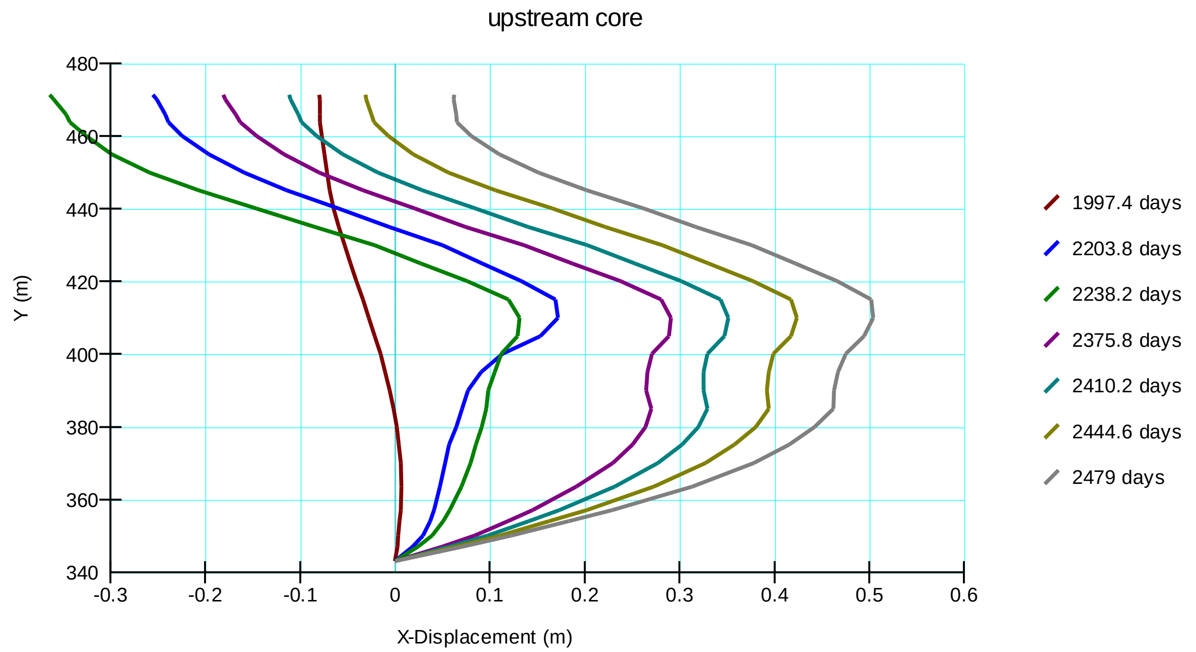

Figure 10. Development of horizontal displacements along upstream face of the core during first filling of the reservoir

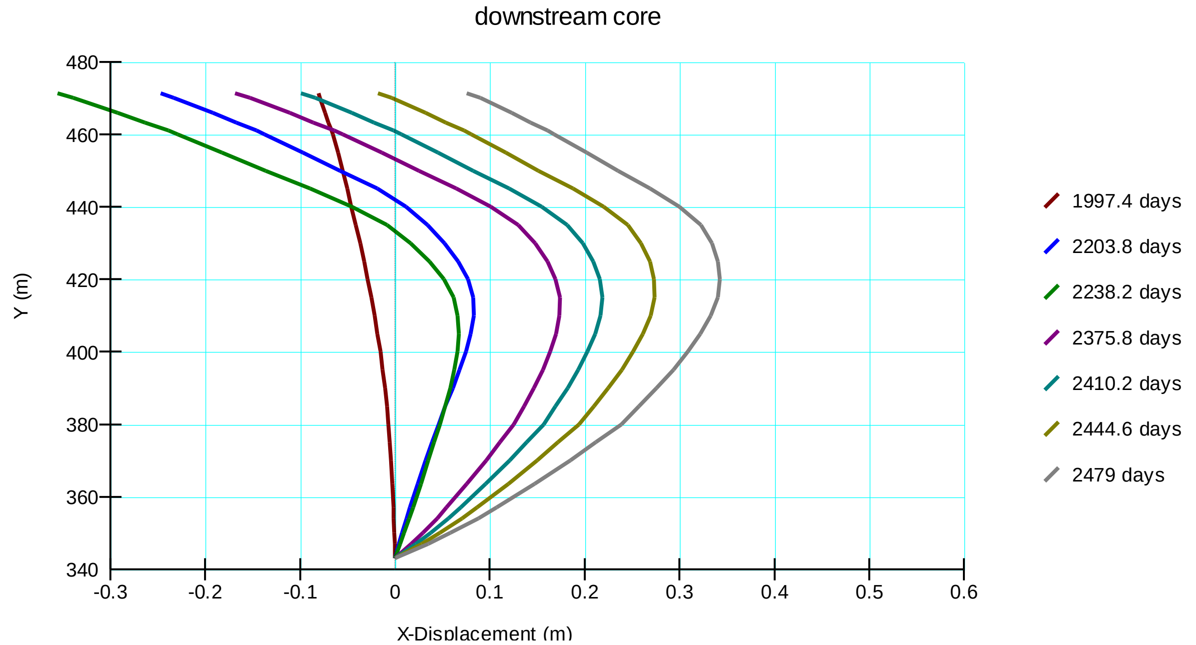

Figure 11. Development of horizontal displacements along downstream face of the core during first filling of the reservoir

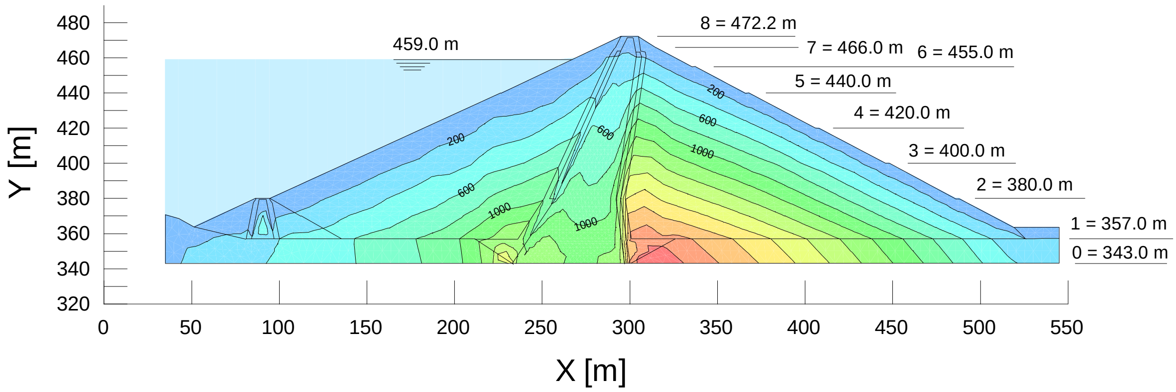

Figure 12. Distribution of effective vertical stresses, (10.14) - (+2,590) kPa after reservoir filling

The change of the total stresses results from the additional load from the increase of the volume weight and external hydrostatic pressure along the upstream slope, while the lowering of the stiffness of the rock material, due to the lowering of the effective stresses, is affecting the displacements. The effective stresses are difference of the total stresses and neutral water pressure (according to the laws of hydrostatic) in the pores of submerged materials upstream of the core. The softening of the material is actually caused by the lowering of the elasticity modulus, due to lowering of the effective stresses. The reason why in reality, by water saturation of the upstream shell (or by lowering of the effectives stresses) raising does not occur (elastic response) is the superposition of at least three effects: (a) increased stiffness at unloading, (b) downstream displacement caused by the basic load – hydrostatic pressure and (c) occurrence of "collapse settlement". The third effect is manifested by settlements of the coarse material after submerging in water, due to the decrease of its strength parameters, crushing of the grains edges - phenomena intensively researched in the last two decades [Alonso et al., 2005; Oldecop and Alonso, 2007; Roosta and Alizadeh, 2012]. The results for the partial horizontal displacements (Fig. 9 and 10), where in the axis of the inclined core, the maximal horizontal displacement is approximately 40 cm in the intermediate part of the dam, and at crest approximately 10 cm, according to the pattern are similar with obtained values by monitoring of earth-rock dams with central core and are appropriate to the numerical analysis of rock-fill dam Konsko (Gevgelija, Republic Macedonia) with asphalt core [Petkovski L., Tančev L., Mitovski S., 2013].

6. CONCLUSION

The response of fill dams to the action of static loads is a complex issue that in most cases cannot be solved by physical law, but is assessed by numerical models. The inclusion of models instead of laws means that for analysis of a dam, the models (based on different approximations) are not mutually excludible but in contrary, they contribute to a better understanding of the prototype behavior. By comparison of the results from the considered models (elastic and non-elastic, with constant and variable elasticity modulus) for the behavior of the earth rock dams with slight inclined core, during the stage of reservoir first filling, the following two observations can be outlined. First, by applying elastoplastic model with variable elasticity modulus in dependence of the effective normal stresses are obtained patterns of bidirectional displacements in the axis of the waterproof element, verified by monitoring of real structures. Second, by applying of boundary hydraulic condition along the upstream slope of the dam, a realistic distribution of the maximal main normal stresses (total and effective) is obtained, which is the base for all further structural (static and dynamic) analyses of the fill dam. Namely, this concept for displacements is in correlation with the calculated stresses, meaning they result from the change in the effective stresses, as difference between the total stresses and neutral water pressure.

ACKNOWLEDGEMENTS

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REFERENCES

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

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

[3] ICOLD, 1978, Bulletin 30, Finite elements methods in analysis and design of dams, Paris, France

[4] ICOLD, 1986, Bulletin 53, Static analysis of embankment dams, Paris, France

[5] ICOLD, 1988, Bulletin 94, Computer software for dams - Validation, Paris, France

[6] ICOLD, 1994, Numerical analysis of dams, Volume III, Theme B1 "Evaluation of pore pressure and settlements of an embankment dam under static loadings", Paris, France

[7] ICOLD, 2001, Bulletin 122, Computational procedures for dam engineering - Reliability and applicability, Paris, France

[8] ICOLD, 2013, Bulletin 155, Guidelines for use of numerical models in dam engineering, Paris, France

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

[10] Petkovski L., Tančev L., Mitovski S., 2007. 06 "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

[11] Petkovski L., Tančev L., Mitovski S., 2013.10 "Comparison of numerical models on research of state at first impounding of rock-fill dams with an asphalt core", International symposium, Dam engineering in Southeast and Middle Europe - Recent experience and future outlooks, SLOCOLD, 16-17.10.2013 Ljubljana, R.Slovenia, ISBN 978-961-90207-9-1, Proceedings, 106-115

[12] Roosta M. R., and Alizadeh A. 2012. Simulation of collapse settlement in rock-fill material due to saturation. International Journal of Civil Engineering, Vol. 10, No. 2, June 2012, pp.93-99.

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STRUCTURAL VIBRATION MEASUREMENT IN DAM MONITORING

Hydropower dams are, due to their operation on turbines and hydrodynamic effects of water continuously subjected to dynamic loading. In this paper, we present a study of structural vibrations using a laser Doppler vibrometer (LDV). Using this measuring gauge velocities are directly obtained, which is a considerable advantage to the accelerometer based measurements. Our aim is to research the impact of dynamic loading due to regular operation of dams. In Slovenia, we are increasingly faced with the problem of aging of dams. At the same time, we are also faced with changes in the environment, especially with variability in time-dependent loads such as new patterns of operation with turbines, with several starts and stops on a daily basis. The in situ structural response measurements are performed on Brežice dam. We have chosen several experimental points on the structure of the dam, where vibrations are captured on a regular basis. The LDV enables measurements of surface velocities in time domain. Since we are interested in analysis of eigen frequencies, measured data is transformed from time to frequency domain for further analysis. Investigation on Brežice dam started already during the construction and will continue in the future. With this study, we aim to present the advantages of structural vibration monitoring of the concrete used as a part of regular structural health monitoring of dams.

Keywords: Optical measurements, vibrometry, concrete gravity dam, measurements

1. INTRODUCTION

Recently a new pilot study to monitor vibrations on massive concrete is implemented on HPP Brežice. This research for the first time introduces the Laser Doppler Vibrometer as an experimental equipment used for vibration monitoring on dams. The device enables non-contact and continual data acquisition at preselected surface points. The vibrometer records surface velocities, which is a considerable advantage to the accelerometer based measurements. Application of vibrometer has also other advantages. Firstly, measurements are not demanding; we just have to find the position from where the laser beam has unhindered access to the object, the device is also portable and does not require any special installation on site.

In this paper we present the methodology and equipment used for experimental work. Various types of time-dependant loads are constantly present on hydro power plants. These loads represent irregular excitation patterns that consist of various dominant frequencies among which the ones close to the eigenfrequencies of the structure are of major concern. These excitations could be the cause for damages on the structure, noticed on some dams during regular inspections. The aim of this study is introduce the advantages of the proposed methodology to be further used as a part of a regular structural health monitoring of Slovenian dams. With the regular observation of vibrations on structures it is possible to monitor the aging process, and detect changes before they are visible on the surface.

1. DESIGN OF THE INVESTIGATION

1.1. HYDRAULIC POWER PLANT BREŽICE

Brežice HPP (Fig. 1) is a fifth hydro power plant in the chain of six hydropower plans on the Lower Sava River. It is a run-of-river type of power plant with a limited storage capacity, which will enable partial compensation of the discharge once the hydro scheme on Sava will be finished. The dam is a combined type of dam; the central concrete-gravity part of dam consists of powerhouse and five overflow sections. The two earthen embankments with maximum height of 9.5 m are connecting the riverbanks to the 160 m long concrete part. Structural height of cross-section of the concrete dam varies, maximum structural height is 36.5 m in the cross section of the powerhouse, and spillway section reaches maximum height at 16 m respectively. The spillway section consists of five 15 m wide overflow sections, each with a maximum discharge capacity of 1,000 m^3/s. Each spillway is installed with a segment gate with a flap for fine level regulation [1].

Figure 1: HPP Brežice

In the power station three vertical Kaplan turbines with a design rated power of 45 MW are installed with estimated annual production of 161 GWh. Rated flow rate is to be 500 m^3/s

and a hydraulic head of 11 m. This is a newly built dam, the construction began in April 2014 and should be finalized by the end of the year 2017. Powerhouse is currently in phase of trial power production with all three aggregates [1].

1.2. LASER DOPPLER VIBROMETER

Laser Doppler Vibrometer is used for non-contact measurements of surface vibration velocities in the frequency range between 0 and 22 kHz. The device is an optical transducer, it works on the basis of optical interference and Doppler effect, it detects the frequency shift of back scattered light from a mowing surface. One of the main advantages of the vibrometer is that the device is portable and easy to use. Once we establish visible contact with the vibrating surface on the distance of maximum 30 m, the recording can begin [2,3]. One example of a measurement of HPP is presented in Figure 2.

Figure 2: Example of a measurement with the Laser Doppler Vibrometer

2. THE DESIGN OF THE FIELD INVESTIGATION

The surface under the investigation has to deflect laser light. In order to improve deflection deflective tiles were installed on all experimental points. Our experiment launched in April 2016, roughly two years after the beginning of the construction works on site. We established a first set of experimental points on several locations in the powerhouse and in the spillway section of the dam. The first set consisted of 4 points (Fig. 3):

- two in the engine room: in the middle of the engine room wall located between aggregate 1 and 2 (S2), and at the upper part of the engine room wall located next to the aggregate 1 (S1);
- two points in the first stilling basin: one in the end and one in the middle of the wall (P1, P2).

These points served to record the initial state of dam. We recorded the response of the dam triggered with the construction work and also at rest.

During trial run operation we continued with our investigation. To the set of initial investigation points new points were added (Fig. 3):

- one point to the outside part to each of the three turbine shafts (A1, A2, A3);
- one point on a pillar next to the turbine 2 (ST1);

- one point on the side pillar in the engine room (ST1);
- two points on the wall between the 4th and 5th stilling basin (P4, P5);
- two points on the wall between the 3rd and 4th stilling basin (P6, P7).

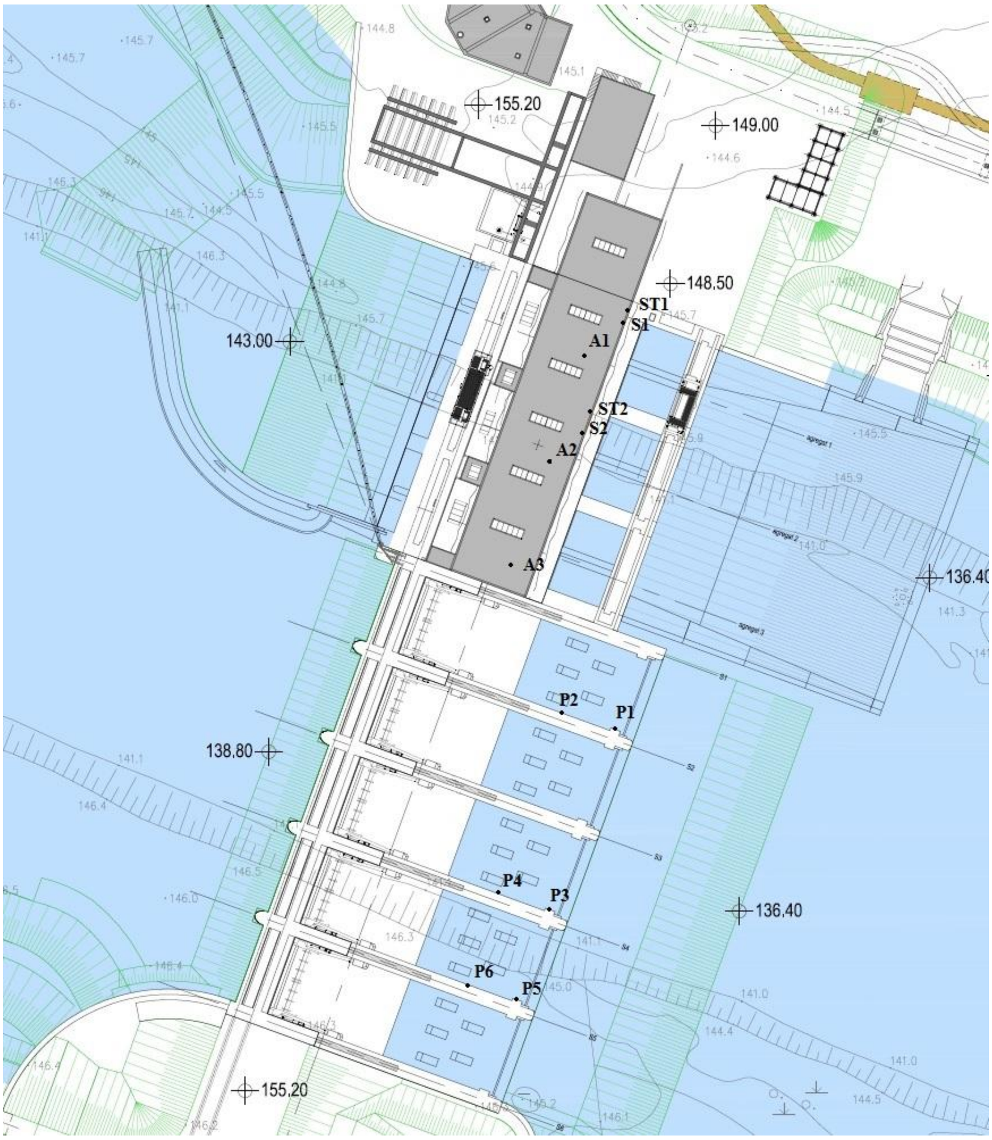

Figure 3: Layout of the experimental points

Currently there are 13 experimental points to monitor vibrations allocated around the structure of the Brežice dam. The investigation is still ongoing; our aim is to monitor the dam also during regular operation.

3. EXECUTION OF THE INVESTIGATION AND ANALYSIS

In this paper we will present measurements recorded during different events on the dam. First example in Figure 4 is a time-history of surface velocities at rest recorded on position P1. We can observe very small magnitudes and the presence of higher frequency oscillations.

The next example presents a time-history of a stilling basin wall triggered with the construction work, this measurement was taken on May 7th 2016. On that day work on the construction site was focused on installation of the components for the hydraulic gates. Example is presented in Figure 5 and shows an increase in the vibration amplitudes that were caused with the construction work. Velocity amplitudes rise up to the 0.08 m/s.

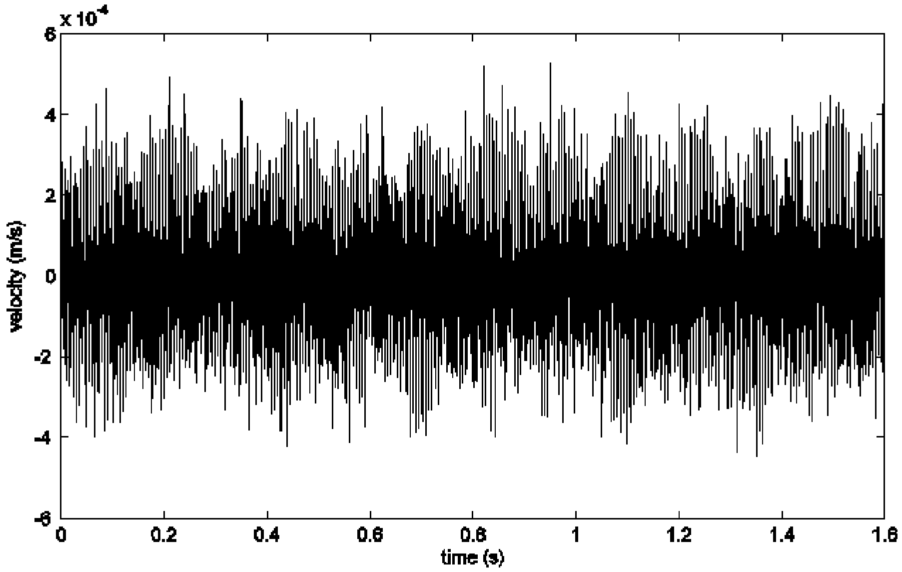

Figure 4: Typical time history of velocities at the stilling basin side wall

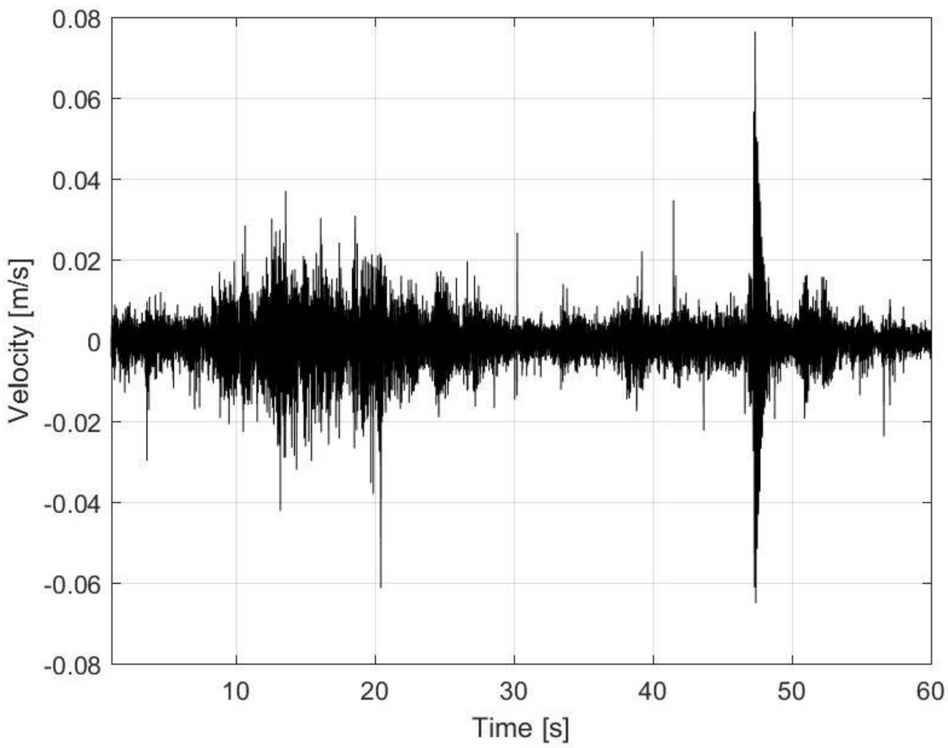

Figure 5. Time history of velocities at the stilling basin sidewall during construction work

Next figure 6 presents the recording in the powerhouse, on the position (S2), this recording was also taken on May 7th 2016. Since the majority of work on the dam was concentrated on the spillway sections, as expected the response of the powerhouse wall has smaller magnitudes in comparison to the measurements in the spillway section.

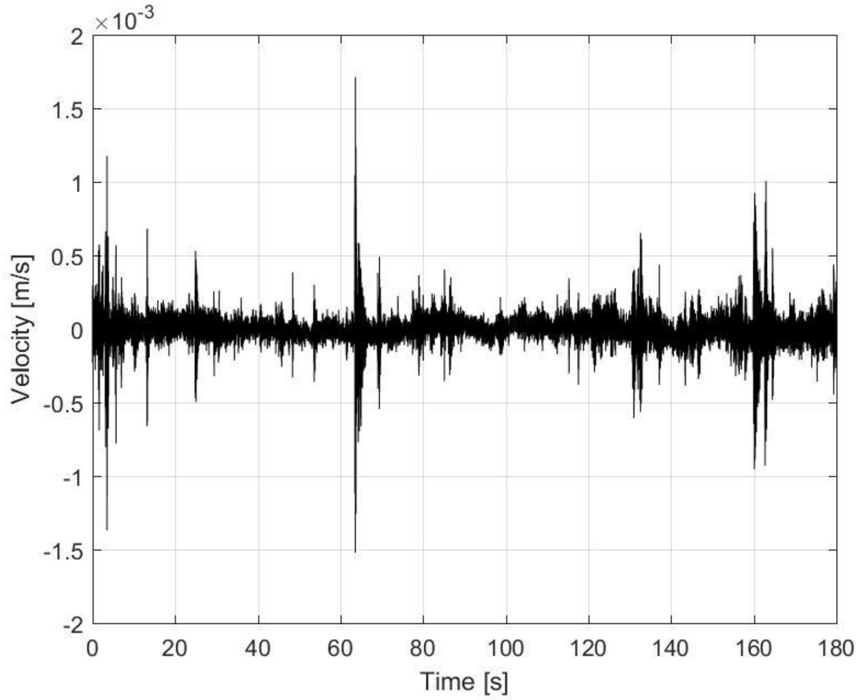

Figure 6: Time history of velocities at the wall in the powerhouse during construction work

On August 16th, 2017 the impoundment behind the dam reached nominal operational level at 153 m a.s.l. On August 21st, 2017 diagnostic test for hydro mechanical equipment began, alongside these testing also vibrations of the structure of the dam are recorded. Figure 7 presents a time-history measured during alteration of power on a turbine, the response is measured on the position A3. An increase in velocity amplitudes is evident, the velocities rise up to 0.5 m/s.

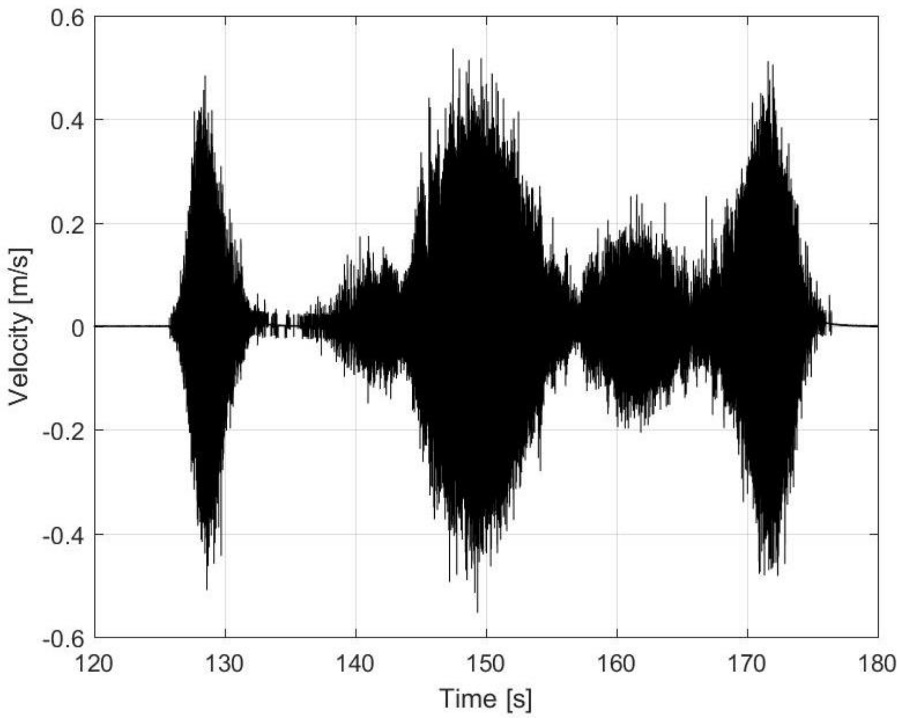

Figure 7: Vibration during on position A3 during the manoeuvring with the turbine, adjustment of power

4. CONCLUSIONS

In this paper we presented the implementation and some preliminary results of the experiment on HPP Brežice. The presented results gave us the insight of the dynamic properties of the structure before it becomes fully operational. The experimental data was obtained by non-contact measurements using laser vibrometers. The measured time history reveals the influence of the construction work on the structure resulting in much higher amplitudes as for the cases where structure vibrates naturally. Despite their massive construction, the hydraulic power plants are sensitive to dynamics loads. Therefore, it is important to monitor dynamic properties of dams as a part of a regular structural health monitoring process.

Our investigation is still ongoing. Future investigation will reveal the effect of the water flow on the structure of the dam. With analysis in the frequency spectrum we will also be able to identify operational maneuvers that are crucial from the structural point of view. With the regular vibration monitoring it is possible to monitor aging process of the structure, with our investigation we established the initial or reference state, and with the continuation we will be able to monitor the aging of the Brežice dam.

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REFERENCE

[1] HESS (2016) Hidroelektrarne na Spodnji Savi, Ltd. - Hess d.o.o. Available at: http://www.he-ss.si/eng/ (Accessed: 5 February 2017).

[2] Nassif, H. H., Gindy, M. and Davis, J. (2005) 'Comparison of laser Doppler vibrometer with contact sensors for monitoring bridge deflection and vibration', NDT & E International, 38(3), pp. 213–218. doi: 10.1016/j.ndteint.2004.06.012.

[3] Polytec (2016) 'Portable Digital Vibrometer PDV-100 High Resolution Digital Velocity Measurement', pp. 1–3.

19 - 20 Октомври, 2018 год.
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Скопје, Република Македонија
Skopje, Republic of Macedonia

Трибина на тема:

80 години од инженерство за брани во Р Македонија

Tribune on topic:

80 Years of Dam Engineering in R Macedonia

Здружение Македонски комитет за големи брани
Macedonian Committee on Large Dams

member of
член на

Меѓународна комисија за големи брани
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VERY SIMPLIFIED SEISMIC RESPONSE EVALUATION OF AN ASPHALT CORE ROCK-FILL DAM – ITS POSSIBILITIES AND LIMITS

Despite of the intensive development of sophisticated dynamic analysis methods and their implementation in the field of Dam Engineering in the last decades due to the explosive increase of the computational power of modern computers, the pseudo-static equivalent-force approach is still being further developed due to its ability to provide fast and simple estimation of some response parameters of the dam under earthquake excitation. This statement is proved by some recent publications in this field continuing to focus the attention of the practicing engineers on such as far as possible simple tools yet providing realistic results.

The present work deals with the example application of such simple pseudo-static response analysis method to a Bulgarian rock-fill dam with asphalt concrete core. The procedure applied was developed in a series of works by Dr. Max A.M. Herzog for embankment dams under basic operational loads as well as for seismic excitation. The obtained results are compared with the results from sophisticated dynamic analysis procedures for the same dam carried out with well-established finite element analysis software. Based on these comparisons, conclusions are drawn about the applicability of the used simplified method for the case of seismic loading on rock-fill dams with bituminous cores.

Keywords: rock-fill dam, seismic response, simplified method

1. INTRODUCTION AND PROBLEM FORMULATION

In the 21st century, the computational power even of the personal computer systems is already remarkably high, and these systems become more easily affordable. On the other hand, sophisticated methods for static and dynamic analyses of complicated civil engineering systems with complex physical interactions are continuously being developed, and their software implementations become more intuitive and user-friendly. This situation can be observed especially in the field of Dam Engineering where the mentioned overall development already allows for much more realistic modelling of complex physical phenomena such as: the strongly non-linear material and structural behaviour of the dam, dynamic soil-structure and fluid-structure interactions, liquefaction, seepage problems.

Despite of the intensive development of both sophisticated dynamic analysis methods and their hard- and software implementation in the field of Dam Engineering in the last decades, simplified methods for cheap, fast and yet reliable assessment of key response parameters of the dam structure to decisive loads and impacts are further developed and used for independent control and comparison purposes. As examples in this connection, the works [6, 7] should be mentioned. They are dedicated to the computation of particular response parameters of the dam (fundamental period, settlement). Special attention is continuously paid to the pseudo-static equivalent-force method still being further developed due to its ability to provide fast and simple estimation of some response parameters of the dam under earthquake excitation. Here, a series of works by M. Herzog should especially be mentioned, a small part of which is used and cited further below [1-4].

In general, the application of more or less simplified methods can be in many cases important for:

- preliminary assessment of key dam response parameters for various load and impact conditions and orientation about the further computational proceeding based on the obtained results in this way;
- independent qualitative control of the results from more sophisticated models / analyses.

In this presentation, an application of a very simplified calculation procedure is shown to the case of a Bulgarian rock-fill dam with asphaltic concrete core currently under construction. The procedure applied follows in general the approach developed in the above mentioned works of M. Herzog [1-4]. The particular case of this dam was selected mainly due to two reasons. On the one hand, the mentioned simplified calculation procedures have been applied in the corresponding sources to earth- and rock-fill dams. It would be indeed interesting how such an approach holds for a rock-fill dam with asphaltic concrete core. On the other hand, a technical design developed by a renowned consultant already exists based on thorough static and dynamic non-linear analyses with real physical parameters of the fill zones obtained from extensive laboratory studies. Thus, the possibility for a comparison with the results of such sophisticated computations also exists which would allow assessment of the applicability of the discussed simplified approach.

It should be noted here that relatively few references discuss rock-fill dams with asphaltic concrete cores compared to the conventional rock-fill dams. Most of them present either particular projects, as for example [8], or discuss particular issues of the design and construction of this type of dams, however, without going deeper into detail regarding computational procedures (which is of course understandable). In this connection, the milestone report [5] should be noted containing extensive information about practically all main issues related to such a project.

2. CASE STUDY AND IMPLEMENTATION OF A SIMPLIFIED PROCEDURE

In the following, an application of the mentioned simplified approach presented mainly in the works [1-4] to a Bulgarian rock-fill dam with asphaltic concrete core is presented.

2.1 PRESENTATION OF THE DAM

The considered dam is in fact no typical rock-fill dam. Its body consists in fact of a crushed rock, even with similar physical parameters as those of the filters. Further below, average mechanical parameters of the case (crushed rock / ballast fill dam) with asphalt concrete core were used. The design and site investigation works of the dam began in the 80s of the last century. In 2001, the construction of the partly built dam (up to about one third in height) and appurtenant facilities was suddenly stopped. Currently, attempts are made for completing the dam and setting the reservoir and its facilities in operation. Due to the large time gap, new site investigations and re-design of the dam and all facilities were carried out by an internationally renowned consultant who won the tender. The typical cross-section of the dam with both zones – the already built one and the upper one to be newly constructed is shown in Figure 1.

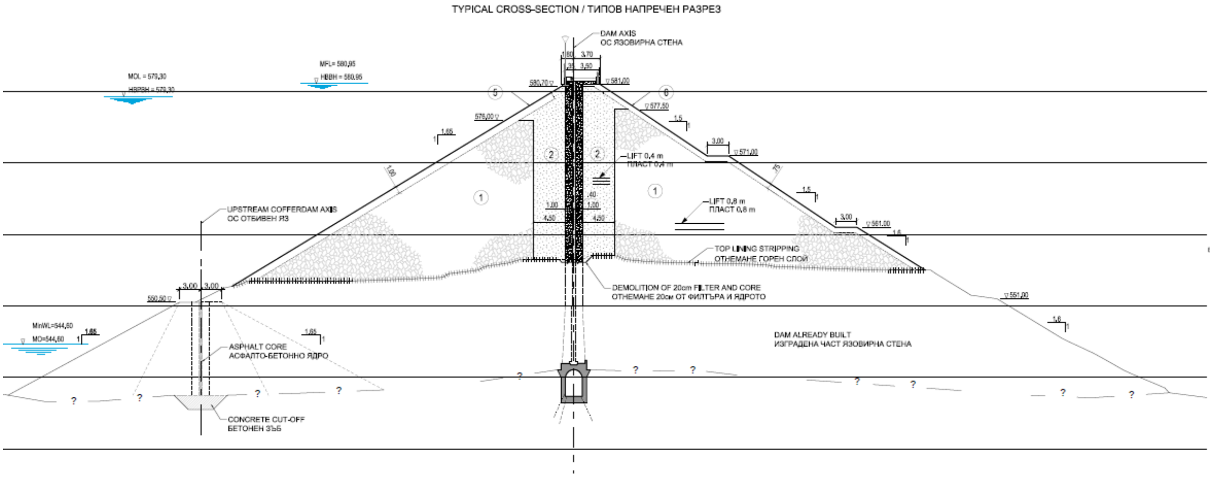

Figure 1. Typical cross-section of the dam

The main design parameters of the dam are as follows:

Type: (crushed) rock fill dam with asphalt concrete core

Height: 47,15 m (maximal, from grouting gallery base to parapet wall crest)

Crest length: 200 m

Crest width: 4,25 m

Gross capacity of the reservoir at maximum operating level: 3841000 m^3

Total volume of the dam: 295500 m^3

Spillway: side channel spillway with stepped chute and stilling basin on the left bank.

According to the requirements of the Bulgarian national code for design of hydraulic structures, the dam was designed as Class II structure. According to the Bulgarian national code for the design of buildings and facilities in seismic regions [10], the site is in a zone of intensity level VII with peak ground acceleration (PGA) of 0,1g corresponding to a return period of 1000 years.

2.2 OUTLINE AND APPLICATION OF THE SIMPLIFIED CALCULATION PROCEDURE

In general, two main lines can be identified in the simplified calculation procedure for earthquake-induced response assessment of the dam according to the works [1-4]. These are:

- calculation of a compound seismic coefficient which can be further used for obtaining of a pseudo-static equivalent-force by multiplication of the weight of the dam (or part of it);
- approximate calculation of some key seismic response parameters of the dam, such as fundamental frequency of the dam, horizontal and vertical earthquake-induced settlement etc.

In the following, the composition of the generalized seismic coefficient according to [3, 4] will be shortly presented. Particular further details are presented in other sources from the series of works by the same author on this problem. Firstly, the fundamental period T_1 (fundamental frequency, respectively) of the dam has to be evaluated. According to [4], the fundamental frequency of the dam can be approximately calculated in (Hz) as:

$$f_1 = \frac{5,6}{\sqrt{w_{stat}}} \tag{1}$$

where w_{stat} is the maximal transversal static deflection of the dam under the horizontal action of its own dead weight, substituted in (cm). Of course, $T_1 = 1/f_1$, and w_{stat} can be obtained [4] as:

$$w_{stat} = \frac{\gamma H^2}{4G} \tag{2}$$

with H being the dam height, γ – the unit weight, and G – the shear modulus.

The generalized seismic coefficient consists of 4 components with the following meaning of the corresponding multipliers M_i:

- response parameter of the seismic excitation:

$$\mathrm{M}_1 = \frac{T_1}{T_b} \qquad (3)$$

where T_b is the upper bound (with respect to period T) of the plateau of the design response spectrum in the particular case.

- influence of the foundation:

$$\mathrm{M}_2 = \left(\frac{2}{\log E_f}\right)^2 \qquad (4)$$

where E_f is the modulus of elasticity of the foundation in (MN/m^2).

- influence of the ductility, i.e. of the relation of the deformation at collapse to the deformation at the limit of linearity (proportionality). This multiplier can be written in the form:

$$\mathrm{M}_3 = \frac{3}{F_d} \qquad (5)$$

where the value of the factor F_d can be read from a table given in [3] depending on the dam type.

- accounting for resonance effects, i.e. for possible activating of the fundamental frequency of the dam by the seismic excitation. For this multiplier, an upper limit value is recommended to be set as:

$$\mathrm{M}_4 = 10 \qquad (6)$$

Thus, the generalized seismic coefficient finally gets the form:

$$\mathrm{C}_s = \frac{a_E}{g} M_1 M_2 M_3 M_4 \qquad (7)$$

where a_E is the design PGA for the dam site.

One more multiplier is further introduced as well to account for the hydrodynamic dam-reservoir interaction. However, since it has to be separately applied to the hydrostatic load and is not directly related to the above presented seismic coefficient it will not be discussed further herewith.

Besides the seismic coefficient, furthermore simple relations and even rules of thumb for some other important parameters of the seismic dam response are introduced in the mentioned sources. These parameters are the maximal horizontal and vertical displacements, values of the dynamic modules as well as the above already mentioned fundamental period / frequency.

The earthquake-induced displacements of the dam body to be expected are of particular interest. The following relations are proposed for them [1-4], respectively:

$$\mathrm{w}_{\mathrm{dyn,h}} = \frac{a_{E,h}}{g} w_{stat} \qquad (8)$$

where $w_{dyn,h}$ is the maximal dynamic horizontal (transversal) crest displacement, and w_{stat} is to be substituted after Eq.(2);

$$\mathrm{w}_{\mathrm{dyn,v}} = \frac{a_{E,v}}{g} \frac{\gamma H^2}{2E} \qquad (9)$$

with $w_{dyn,v}$ the maximal dynamic vertical crest settlement.

It would be reasonable, and hence recommended, to use the dynamic values of the corresponding deformation modules in Eqs.(8, 9). Reversibly, these relations can be used for obtaining the average dynamic deformation modules of a dam if measured values of the seismic crest displacements exist.

2.3 RESULTS AND DISCUSSION

In the case of the considered dam in particular, we start the application of the outlined approach with calculation of the fundamental frequency / period of the dam. According to the submitted by the Designer results from the laboratory investigations of the dam materials, the unloading-reloading deformation module is 63 MN/m^2. Further below, we used these values as an approximate average for the dam body without taking into account the mechanical parameters of the asphalt concrete core, i.e. latter's contribution to the deformation behaviour of the dam. Thus, the shear modulus is 26,25 MN/m^2. The average unit weight is 21 kN/m^3. With these values, the maximal horizontal transversal static crest displacement according to Eq.(2) is 44 cm. By means of Eq.(1), the fundamental frequency of the dam is calculated as 0,844 Hz, i.e. the fundamental period is 1,184 s.

For comparison, by means of the empirical relations given in [9], the fundamental period of the dam is calculated as 1,187 s. There is some discrepancy between these values and the computed fundamental period by the Designer by means of the sophisticated FE-model 1,104 s. However, the difference between the obtained values and the interval for the fundamental frequency proposed by the rules of thumb in [2] is much larger – 0,291 s to 0,357 s. This observation just emphasises the need for great caution and independent comparison when any strongly simplified relations are used.

In a next step, the components of the presented above generalized seismic coefficient are calculated as follows:

- response multiplier from Eq.(3):

$$\mathrm{M}_1 = \frac{0{,}36}{1{,}184} = 0{,}30 \qquad (10)$$

- influence of the foundation from Eq.(4):

$$\mathrm{M}_2 = 0{,}396 \approx 0{,}4 \qquad (11)$$

with modulus of elasticity of the foundation E_f = 1500 MN/m^2 according to the site investigation results;

- influence of the ductility:

$$\mathrm{M}_3 = 0{,}43 \qquad (12)$$

with F_d = 7 as read from the table given in [3] for the considered dam type;

- multiplier for resonance effects:

$$\mathrm{M}_4 = 10 \qquad (13)$$

Thus, the generalized seismic coefficient gets the value in the considered case:

$$\begin{aligned} \mathrm{C}_s &= \frac{a_E}{g} M_1 M_2 M_3 M_4 = \\ &= 0{,}1 * 0{,}3 * 0{,}4 * 0{,}43 * 10 = \\ &= 0{,}1 * 0{,}516 = 0{,}0516 \end{aligned} \qquad (14)$$

with horizontal design PGA for the dam site a_E = 0,1g.

For comparison, calculation of a generalized seismic coefficient according to the Bulgarian national seismic code [10] was performed. If only the fundamental period is taken in to account, the following relation holds:

$$\begin{aligned} \mathrm{C}_s^* &= k_c C R \beta(T) = 0{,}1 * 1{,}5 * 0{,}25 * 0{,}8 = \\ &= 0{,}1 * 0{,}3 = 0{,}03 \end{aligned} \qquad (15)$$

Such comparison could only serve general orientation about the results from the quite different approaches since the meaning of the single multipliers is here complete different: in fact, k_c = a_E/g, C = 1,5 represents the importance class of the structure, R = 0,25 is the reciprocal value of the corresponding ductility factor, and the dimensionless function β(T) gives the shape of the design response spectrum with respect to three groups of soil conditions. The used here value is according to the actual conditions (foundation in rock) and fundamental period.

The difference between the results is obvious. As it can be clearly seen, when applying simplified approaches, one should be cautiously aware of all assumptions made as well as of the meaning of every single parameter used.

Finally, the maximal earthquake-induced crest displacements can be calculated:

- The maximal dynamic horizontal (transversal) crest displacement for the design excitation with PGA = 0,1g is 4,4 cm according to Eq.(8). For comparison, the computed horizontal seismically induced displacements by the Designer by means of a sophisticated non-linear FE-model are 14 cm + 4,5 cm (initial + residual value) with maximal amplitude of 7 cm. Although any direct comparison would be impossible and simply non-professional, the accuracy of the dynamic displacement is obviously not bad.
- The maximal dynamic vertical crest settlement for the design excitation with PGA = 0,1g is 3,7 cm according to Eq.(9). For comparison, the computed horizontal seismically induced displacements by the Designer are 22 cm + 5 cm (initial + residual value) with maximal amplitude of 6 cm. Also here, under the above formulated assumptions, the accuracy of the dynamic displacement is fully acceptable, too. If as usual according to [10], the maximum vertical excitation is assumed to be 2/3 of the maximal horizontal one, the maximal vertical crest displacement will become 2,48 cm instead of 3,7 cm.

2. CLOSURE

The above illustrated application of a strongly simplified approach to the seismic response of a rock-fill dam with asphaltic core or to determination of some particular parameters of the dam structural behaviour can be highly efficient with respect to the possibility to obtain in a very short time orientation about quantitative values of key parameters of the dam and its response to different loads and impacts. Such results are especially useful when the obtained values are realistic. In many cases, the latter one can be proved by means of comparison with measured values or with results from rigorous computations with much more realistic sophisticated models of the system.

However, in many cases such comparison possibilities simply do not exist, and just in such cases, obtaining a preliminary realistic assessment is especially important. In such cases the application of heavily simplified approaches can become a problem. In this connection, the following considerations should be taken into account:

- Despite of the theoretical (although based on simple mechanical models) justification of every component of the compound seismic coefficient in the pseudo-static equivalent-force approach or of the relation used for a particular physical effect, the obtained results should be treated with great caution.
- The use of an integral seismic coefficient in the pseudo-static equivalent-force method is a matter of global approach, and the particular calculation procedure should always be applied in its consistency and completeness with respect to the particular assumptions and effects accounted for. The emphasized caution needed here is due to the ambition to describe at the end all substantial physical phenomena of interest by means of a single number.
- In general, it is different when simplified relations are applied to particular response parameters (for example: earthquake-induced crest settlement), however, in such cases one should be quite clearly aware of the assumptions and features of the mechanical system model justifying the used relation(s) as well as of the assumptions and features of the impact describing model.
- Rich experience in the field is inevitably required for any application of any similar simplified approach.
- Last but not least – although simplified approaches are most commonly used for independent control purposes parallel to the use of sophisticated computational procedures, their application also needs an independent control. If measurements and / or sophisticated computational models are not present, an independent comparison with similar cases, data from literature sources etc. will be inevitable.

REFERENCE

[1] M. Herzog: „Closed formulae for earthquake-induced deformations in dams“, Water Power & Dam Construction, January 1985, pp. 33-34

[2] M. Herzog: „Eigenschwingungsdauer sowie dynamischer Gleitmodul von Erd- und Steinschüttdämmen“, Bautechnik, 69, Heft 9, 1992, pp. 515-517

[3] M. Herzog: “Zutreffende Erdbebenbemessung von Talsperren mit einem verbesserten statischen Ersatzlastverfahren”, Österreichische Wasserwirtschaft, Jahrgang 44, Heft 1/2, 1992, pp. 15-22

[4] M. Herzog: “Der Bruch des Sheffield-Dammes in Santa Barbara beim Erdbeben am 29. Juni 1925 im Rückblick”, Österreichische Wasserwirtschaft, Jahrgang 44, Heft 7/8, 1992, pp. 206-208

[5] K. Höeg: „Asphaltic concrete cores for embankment dams. Experience and practice“, Statkraft, Veidekke, Norwegian Geotechnical Institute, 1993

[6] R. B. Jansen: „Estimation of embankment dam settlement caused by earthquake“, Water Power & Dam Construction, December 1990, pp. 35-40

[7] Zhen-Zhong Shen, Zhi-Ying Xu: „Shear vibration of 3-D inhomogeneous earth dams in triangular canyons“, Paper No.584, 11th WCEE, Acapulco, Mexico, 1996

[8] T. Valstad, P. B. Selnes, F. Nadim, B. Aspen: „Seismic response of a rockfill dam with an asphaltic concrete core“, Water Power & Dam Construction, April 1991, pp. 22-27

[9] И. Б. Папазчев, Хр. Б. Абаджиев, Л. В. Шейтанова, Д. Н. Тошев: „Ръководтсво за проектиране на хидротехнически съоръжения”, Техника, София, септември 1991 (In Bulgarian: I. B. Papazchev, Chr. B. Abadzhiev, L. V. Sheytanova, D. N. Toshev, “Manual for design of hydraulic structures”. Tehnika, Sofia, September 1991).

[10] Наредба № РД-02-20-2 от 27 януари 2012 г. за проектиране на сгради и съоръжения в земетръсни райони, Министерство на регионалното развитие и благоустройството, София, България, 2012 (In Bulgarian: Ordinance № РД-02-20-2 of 27th January 2012 on the design of buildings and facilities in seismic regions, Ministry of regional development and public works, Sofia, Bulgaria, 2012)

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MANAGEMENT OF MULTIPURPOSE WATER STORAGE RESERVOIR IN FLOOD WATER REGIMES

As a consequence of climate changes the extreme hydrological phenomena occures, in terms of spatial and temporal variability. In such conditions – of more frequent and extreme high water regimes and reduction of low water – the water storage reservoirs should be used for active flood protection as well as for improving the low water regimes. The possibilities for better flood control in the multipurpose water resources system, including the water storage reservoirs, are analyzed in the paper. Case study described in the paper refers to the Trebišnjica River Hydrosystem (Herzegovina). As a part of the improvement of management process a mathematical simulation model was developed. Model proposes the most favorable operational rules for operation with water gates at the bottom outlet and spillways of the dam, according to the criterion of achieving the best transformation of the flood wave. The main goal is the best possible protection of the Trebinje Town from destructive influence of the flood waves.

Keywords: Flood control, water storage reservoirs, mathematical modeling

1. INTRODUCTION

One of the important purposes of the water storage reservoirs is flood protection. That purpose becomes more important under conditions of climate changes, whose main characteristic is greater irregularity of precipitations, with shorter but more intensive raining periods and longer periods of droughts, especially for rivers with small catchment areas.

In the design phase of water storage reservoirs designers always define some additional volume for flood protection. But full flood control effects of reservoirs (and overall water resource system) can be achieved after implementation of the operative management model, where one of the modules is minimization of water flows in the river downstream of the reservoir. Now days, such management models are usual part of water resources system projects, but 20 or 30 years ago that area was at the infant stage and such parts of projects did not exist [1]. As a consequence, a lot of existing reservoirs in Serbia and the wider region do not have such models, and management in period of high water flows is usually performed on the base of experience, or previous calculations. This management approach limits the management options, and usually all available reservoir capacities are not used [2]. In some cases inadequate management can generate worse flood than it would be in natural conditions. It is especially dangerous in the case of dams with gated spillways.

That is why it is very important to implement operative and flexible water reservoir management models [5], which can calculate the consequences of some management rules or find the optimal management rules for defined goals and limitations.

2. BACKGROUND AND STUDY AREA

The Trebišnjica Hydrosystem (Eastern Herzegovina, Bosnia and Herzegovina) is one of the most complex water resources systems in the region [7]. Construction of the system started 60 years ago and it still lasts. The backbone of the system is Trebišnjica River, the largest sinking river in Europe. Its catchment area is highly karstified with numerous ponors, springs, estaveles and underground connections with different capacity. The population of this area has been struggling with water for centuries. In cold period of the year (period of high precipitation when underground conduits are saturated), local people struggle with floods. Beside high water flows, in that period, all karst poljes are turned into lakes. In summer period, with low or without precipitation, they struggle with droughts.

With the construction of the complex Trebišnjica Hydrosystem in Eastern Herzegovina region, the rechearge of the largest infiltration zones was reduced to specific, short-lasting, hydrological periods or completely bloked. Water is stored in reservoirs and transported through the tunnels and channels from higher elevated parts of catchment areas down to the sea. Along its route water is used for power production, irrigation, water supply and number of secondary benefits. Duration of flood events in karst poljes is limited and locally eliminated.

The Hydrosystem Trebišnjica consists of seven dams, six reservoirs, six tunnels (with total length of 57 km), four channels with total length of 74 km, and seven hydropower plants with total instaled capacity of 1069 MW.

Due to its complexity, the construction of the system has been divided into three phases. The I phase of the system (the most economical part) includes the Grančarevo dam, with HPP Trebinje I and the Gorica dam with HPP Dubrovnik and HPP Trebinje II (Fig. 1). This is the „backbone“ of the entire hydrosystem and it was completed in 1975. The management model described in the article is made for this part of the system.

The main element of the system is Bileća Reservoir, created by 123 m high Grančarevo Dam, with total water storage volume of 1280 · 10^6 m^3, active volume of 1100 · 10^6 m^3 and normal operating level 400 m a.s.l. There are two lateral spillways with radial gates for flood discharges, with maximal capacity of 874 m^3/s and two bottom outlets with maximal capacity of 266 m^3/s. HPP Trebinje I, with three Francis turbines of installed flow 3 × 70 m^3/s, and installed power 171 MW, is located in the immediate proximity of the dam body.

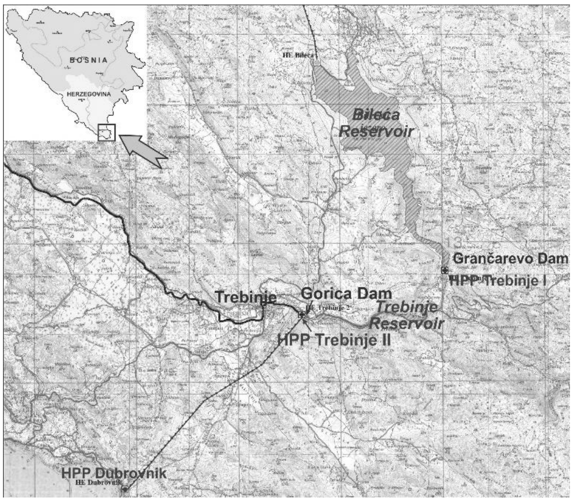


Figure 1. Trebišnjica Hydrosystem (I phase)

The gravity concrete Gorica Dam (33.5 m high) is situated 13.5 km downstream from Grančarevo dam. It formes Trebinje Reservoir, with total storage volume of 15.6 · 10^6 m^3 and normal operating level 295 m a.s.l. There are two spillways in the centre of the dam, with radial gates, with maximal capacity of 412 m^3/s and two bottom outlets with maximal capacity of 800 m^3/s. Two intakes for the HPP

Dubrovnik are located at the left bank in the immediate proximity upstream of the dam body. The HPP Dubrovnik is underground HPP, with 16.5 km long head race (tunnel and penstock) and two Francis turbines with installed flow 2 × 45 m^3/s and installed power 2 × 108 MW. In the left bank immediately downstream of the dam is HPP Trebinje II, with one Kaplan turbine (installed flow 45 m^3/s, minimal flow 12 m^3/s). Ecological flow [3] that has to be provided downstream of the Gorica Dam is 8 m^3/s. There is also outlet for discharging guarantied flow downstream of the dam.

The Trebinje Town is situated 4 km downstream from the Gorica Dam. One of the important purposes of the system is flood protection of the Trebinje Town. This purpose becomes more important and complex in conditions of climate changes, when flood waves are more frequent and intensive as well as because of uncontroled construction of the area in the immediate vicinity of the river. That is why the Trebinje is now endangered with flows greater than aproximately 400 m^3/s, the flow that occurs once in 1.25 years (80% probability), while earlier that boundary flow was much higher, round 750 m^3/s.

Due to this situation a mathematical simulation model for management in flood periods was developed (Flood wave mitigation model). Model proposes the most favourable management rules the gated spillways and bottom outlets of the dam, according to the criterion of achieving the best mitigation of the flood wave. The main goal is the best possible protection of the Trebinje Town from destructive influence of the flood waves. This model is a part of complex Trebišnjica River Management Model, designed to improve all aspects of the system's effectivity and the operational management capabilities of reservoirs and HPPs on Trebišnjica River. Main task is optimisation of management to achive optimal hydropower production, the best efects of flood protection and the best protection of Trebišnjica river ecosystem in period of low water flows.

3. HYDROLOGICAL REGIME IN THE STUDY AREA

Hydrological regime of the investigated area is very irregular, as a consequence of irregular precipitations (with maximal values in cold period of the year, late autumn and winter) and highly karstified area with numerous underground conduits, that reduces the period of flood wave concentration.

In order to create a flood mitigation model, it was necessary to analyze the flood waves that occur in the reservoirs. It should be mentioned that one of the characteristic of Bileća Reservoir is that the reservoir submerge its own springs aproximately 75 m. That is why it is not posible to measure the inflow into the reservoir, as the inflow is completely underground. That data can be obtained indirectly, by calculations based on data on the change of the reservoir volume and flow that is discharged downstream.

Hydrological analises for Bileća Reservoir for period 1957-2006 indicates that high water flows are frequent in period from 15 October to 15 Mart (Fig. 2), with specially high frequency in the period from 1 November to 31 February. During this period, depending on the water level in Bileća Reservoir, it is highly probable that the priority task of system management will be switched from the energy optimisation regime to the regime of flood protection and mitigation of flood waves.

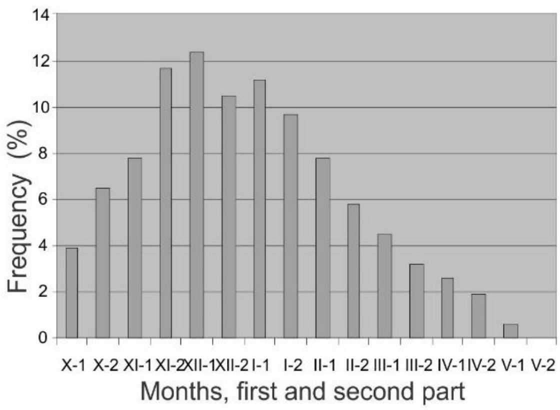


Figure 2. Frequency of flood waves that inflow into the Bileća Reservoir

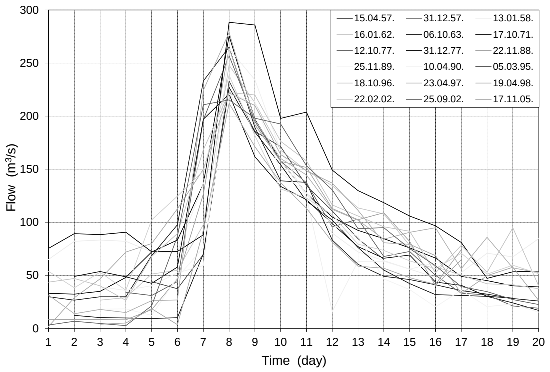
In its final stage, Trebišnjica River Managament model will have prediction module (part), with task to predict flood waves on the base of different inputs (mainly the precipitation, flows and levels in piezometers). To complete that part of the model it is necessary to equip the entire catchment area with measuring stations with automatical transfer of the measured data to the central database, where necessary calculations will be performed. This is complex task (time and economicaly very demanding). Untill that part of the model is completed, input flows are defined on the base of detailed hydrological analyses of all flood waves (waves with maximal flow over 200 m^3/s) recorded in period 1957-2006. Some general parameters of those waves are:

- period of flood wave concetration is from 1 to 2 days,
- base flow varies from 50 m^3/s to 150 m^3/s,
- maximal gradients of flow increase are over 300 m^3/s in a day,
- recession period usually last 6 to 8 days,
- gradientof flow decrease is 100 – 150 m^3/s in a day.

Also there is a possibility of occurance a few successive waves.

On the base of described analyses flood waves are divided into three groups:

1. small waves, with maximal flow less than 300 m^3/s – those waves can easily be mitigated in Bileća Reservoir without any aditional management measures. This type of waves are not included in the model;



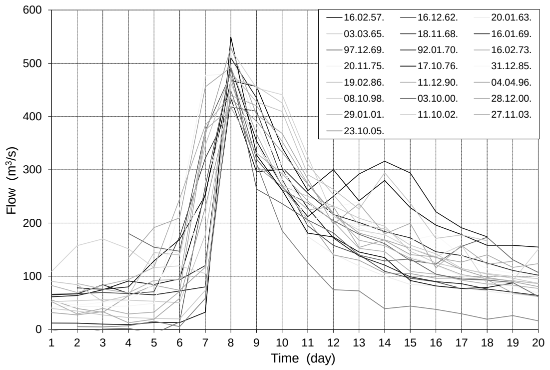


Figure 3. Medium and high flood waves for Bileća Reservoir

2. medium waves, with maximal flows from 300 m^3/s to 400 m^3/s (Fig. 3) – those waves were used to define sinthetic flood waves named 'S' – midium waves in model and has to be tested by the Flood wave mitigation model;
3. high waves, with flows over 400 m^3/s (Fig. 3), used to define sinthetic flood waves named 'V', that has to be tested. If such waves occures the entire system switch into emergency flood protection regime, which implies the obligation of all HPPs to continiously work with it's installed flows, since the flood protection is a prior management requirement.

Inflow into the Trebinje Reservoir (lateral inflow between Grančarevo and Gorica dams) is also mostly underground and cannot be directly measured. Analyses of inflows into the Grančarevo and Trebinje reservoirs indicated that very good correlation between these flows can be established:

$$Q_{mdot} = 0{,}000648545 \cdot Q_{dot}^2 + \\ + 0{,}0836931 * Q_{dot} + 1{,}159062$$

where

Q_{mdot} – lateral inflow into the Trebinje Reservoir

Q_{dot} – inflow into the Bileća Reservoir

On the base of this equation inflow into the Trebinje Reservoir can be easily calculated.

But, during the exploatation of the system, extremely unfavourable lateral inflows into the Trebinje Reservoir were noticed. The reason is the underground karst catchment area of the Sušica River (the biggest tributary of Trebišnjica River). That is why lateral inflow in model can be also defined using synthetic flood waves, with maksimal flow in the range from 200 to 300 m^3/s (Fig. 6). This is more complex task because Bileća Reservoir has to mitigate flood waves more significantly in order to enable mitigation of such a large wave in the Trebišnjica Reservoir.

4. FLOOD WAVE MITIGATION MODEL

Management model for flood mitigation is simulation model with main task to give the most favourable management solution for Bileća and Trebinje resevoirs in flood periods, satisfying certain management rules, with goal to protect Trebinje Town from flooding (flow should be less than 400 m^3/s) and to achive maximal mitigation of flood wave in reservoirs. Model is based on the calculation of balance equations, first for upstream Bileća Reservoir, and then for the Trebinje Reservoir.

The system management strategy in the conditions of high water inflows completely differs from management strategy in regular operation conditions, when main request for system is optimal production of HPPs as well as satisfying all other users of the system. In flood conditions system management and operation of HPPs have task to mitigate flood waves in the best possible way. It is assumed that HPPs in that period work with their full available capacity. Specialy important is operation of HPP Dubrovnik, as it redirects water from the Trebišnjica River toward the Adriatic Sea.

Schematic view of the system is presented at Fig 4. The system consists of two subsystems: Bileća Reservoir and Trebinje Reservoir.

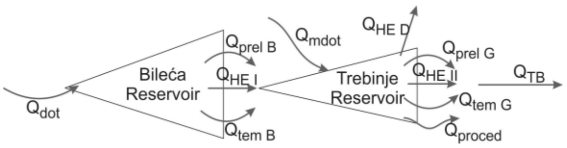


Figure 4. Schematic view of Trebišnjica Hydrosystem (I phase)

Input parametars for model: • inflows into the Bileća Reservoir (Q_{dot}), • lateral inflows into the Trebinje Reservoir (Q_{mdot}), • number of available turbines in HPP Trebinje I, • operation regime of turbines in HPP Trebinje I (optimal or maximal), • water level in Bileća and Trebinje reservoirs at the begining of flood period (H_{B0} and H_{T0}, respectively), • water level in Bileća Reservoir that should be achieved at the end of flood period, • maximal water level in Bileća Reservoir that should not be exceed, • water level in Bileća Reservoir when spillway gates start opening.

Output parametars of the model: • water levels in Bileća and Trebinje reservoirs (H_B and H_T) as time series for flood period, • management of spillway gates at Grančarevo and Gorica dams, • management of Grančarevo and Gorica dams bottom outlets, • water flows downstream of Gorica dam (flows through Trebinje Town) as time series for flood period (Q_{TB}).

Paramerers calculated in the model: • water flow through spillways ($Q_{prel\ B}$) and bottom outlets ($Q_{tem\ B}$) at Grančarevo dam, • turbine flow in HPP Trebinje I ($Q_{HE\ I}$), • water flow through spillways ($Q_{prel\ G}$) and bottom outlets ($Q_{tem\ G}$) at Gorica dam, • turbine flow in HPP Trebinje II ($Q_{HE\ II}$) and HPP Dubrovnik ($Q_{HE\ D}$), • leakage flow from Trebinje Reservoir (Q_{proced}).

As it was earlier mentioned, untill prognostic model for the system is not incorporated into the model, the inflow into the Bileća Reservoir (Q_{dot}) can be defined by selecting flood wave from the synthetic flood wave bases. There are two groups of waves ('S' – medium and 'V' – large) and four types in each group (Fig. 5). Concentration periods of flood waves are 1, 1.5 or 2 days and period of recession 8 ÷ 9 days. Maximal flows are up to 450 m^3/s for medium waves and up to 700 m^3/s for high waves.

Lateral inflow into the Trebinje Reservoir (Q_{mdot}) can be defined in two ways:

1. The automatic procedure (usually applied) - inflow is determined on the base of the correlation dependence described earlier.

2. Choice of a special synthetic wave of lateral inflow - In very rare cases the lateral inflow can be greater than the inflow generated by the correlation dependence. If such a hydrological situation is predicted, lateral inflow can also be selected from another set of synthetic hydrograms named 'MS' (3 types of hydrograms, Fig. 6).

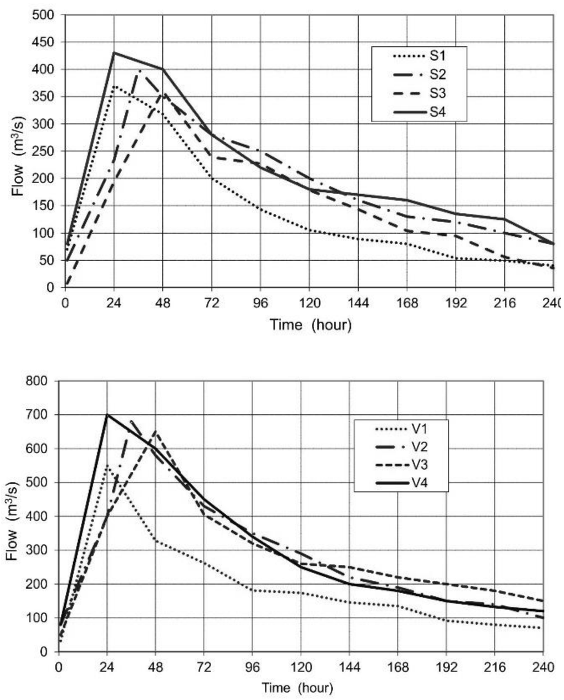


Figure 5. Synthetic flood waves for Bileća Reservoir

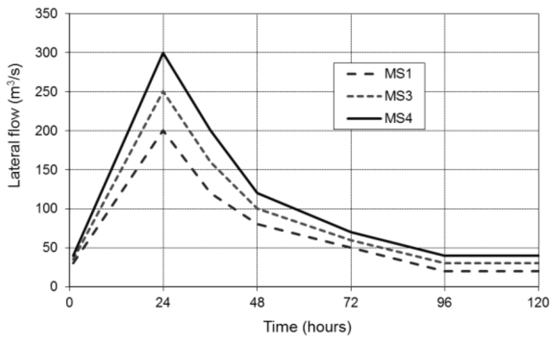


Figure 6. Synthetic flood waves for lateral inflow into the Trebinje Reservoir

4.1 SUBSYSTEM 'BILEĆA RESERVOIR'

The management for Bileća Reservoir is determined using basic balance equation and certain management rules:

$$\frac{\Delta V_{B,i}}{\Delta t} = \overline{Q}_{dot,i} - \overline{Q}_{HE\,I,i}(H_{B,i}) - \overline{Q}_{prel\,\mathrm{B},i}(H_{B,i}) - $$
$$- \overline{Q}_{tem\,\mathrm{B},i}(H_{B,i})$$
$$H_{B,i} = f(V_{B,i})$$

where:

- $\Delta V_{B,i}$ - change of the volume in the i-th interval of time: $\Delta V_{B,i} = V_{B,i} - V_{B,i-1}$

- Δt - time interval, which is constant in the model: Δt = 1 čas

- $\overline{Q}_{*,i}$ - mean flow value during the time interval. It is assumed that degree of openness of gates is constant during the time interval. The change in the flow depends only on the water level in the reservoir $H_{B,i}$

$$\overline{Q}_{*,i}(H_{B,i}) = \frac{Q_{*,i}(H_{B,i}) + Q_{*,i-1}(H_{B,i-1})}{2}$$

Water flow downstream of the Grančarevo Dam (inflow into the Trebinje Reservoir):

$$Q_{B\_izl}(t) = Q_{HE\,I}(t, H_{B,i}) + Q_{prel\,\mathrm{B}}(t, H_{B,i}) +$$
$$+ Q_{tem\,\mathrm{B}}(t, H_{B,i})$$

Since the flows QHE I, Qprel B and Qtem B depend on the water level in the reservoir, it is necessary to carry out an iterative calculation for determining the flow, that is, the volume and the water level for each time step.

The calculation is performed as iterative procedure until a satisfactory solution is obtained:

- In the first iteration, it is assumed that the spillway gates and bottom outlets are closed, so that entire discharge is equal to turbine flow of HPP Trebinje I. If maximal water level in the Bileća Reservoar does not exceed the input maximal level for a given period of the year, the flood wave can be completely accepted (mitigated/caught) in the reservoar, without any additional discharge.

- If the water level exceeds input maximal value it is necessary to discharge additional quantity of water, first through the spillway, and then if necessary through bottom outlets. The radial spillway gates openes when water level achieves elevation defined as input value/parametar. This water levels are one of the results of this model (Fig. 7) and are described later in the article. The opening step is 10 cm. It is allowed to open one step every hour, and maximal opening is 3m.

- If the flood wave can not be evacuated this way bottom outlets has to be opened. Both outlets open simultaneously, due to the dissipation of energy above the waterfall. The opening step is 10%, and it can be opened once in an hour.

- The calculation continues until such management is achieved that the maximal water level in the reservoir is under defined maximal value.

- Closing of the gates is performed in a similar manner and at the same water levels in the reservoir as opening. First the gates of the bottom outlet are closed, and then the spillway gates. The steps for cloasing are the same as for opening (10% for bottom outlet gates and 10 cm for spillway gates). The gates cloased one step each time when water level in the reservoir falls below the defined water level and it continues until gates completelly cloase.

4.2 SUBSYSTEM 'TREBINJE RESERVOIR'

The schematic view/representation of the subsystem 'Trebinje Reservoir' is shown at figure 4. The management rules for the Gorica Dam (the 'G' in the equations) are determined using basic balance equation and certain management rules, similar as for the Bileća Reservoir:

$$\frac{\Delta V_{G,i}}{\Delta t} = \overline{Q}_{mdot,i} + \overline{Q}_{HE\,I,i} + \overline{Q}_{prel\,\mathrm{B,i}} + \overline{Q}_{tem\,\mathrm{B,i}} - Q_{HE\,D}$$
$$- Q_{HE\,II} - \overline{Q}_{prel\,\mathrm{G,i}}(H_{G,i}) - \overline{Q}_{tem\,\mathrm{G,i}}(H_{G,i}) - \overline{Q}_{proced,i}(H_{G,i})$$

$$H_{G,i} = f(V_G, i)$$

where:

- $\Delta V_{G,i}$ - change of the volume of Trebinje Reservoir in the i-th interval of time:

$$\Delta V_{G,i} = V_{G,i} - V_{G,i-1}$$

- Δt - time interval, which is constant in the model: $\Delta t = 1$ čas

- $\overline{Q}_{*,i}$ - mean flow value during the time interval. It is assumed that degree of openness of gates is constant during the time interval. The change in the flow depends only on the water level in the Trebinje Reservoir $H_{G,i}$.

$$\overline{Q}_{*,i}(H_{G,i}) = \frac{Q_{*,i}(H_{G,i}) + Q_{*,i-1}(H_{G,i-1})}{2}$$

Water flow downstream of the Gorica dam (Q_{TB}):

$$Q_{TB}(t) = Q_{HE\,II} + Q_{prel\,\mathrm{G}}(t, H_{G,i}) +$$
$$+ Q_{tem\,\mathrm{G}}(t, H_{G,i}) + Q_{proced}(t, H_{G,i})$$

These flows has to be determined in iterative calculation for each time step because they depend on the water level in the reservoir.

Since the Bileća Reservoir has the main role/task in flood waves mitigation (because of its large active volume), while the Trebinje Reservoir has the role of compensatory basin, simulation analysis has shown that the recommended water level in the Trebinje Reservoir during the flood protection period is 294 m a.s.l., and in periods of high probability of the appearance of large waves (especially large lateral inflow) recommended water level is 292 m a.s.l.

The iterative calculation is very similar to procedure described for the Bileća Reservoir, but some management rules are different.

- HPP Dubrovnik and HPP Trebinje II operate with constant turbine flow $Q_{HE\ II}$ = 45 m^3/s and $Q_{HE\ D}$ = 90 m^3/s;
- Leakage from the reservoir (Q_{proced}) depends on the water level in the reservoir and can be calculated on the base of functional relation.
- When it is necessary to discharge over 135 m^3/s the radial gates at the bottom outlets open (symmetrically), with opening step of 30 cm, until discharge of 350 m^3/s is achieved. Gates are managed with the condition that the level in the Trebinje Reservoir is maintained constant, at the level equal to level at the begining of flood period (H_{T0}). Such management provides flows through Trebinje less than 400 m^3/s.
- If it is necessary to discharge through bottom outlets over 350 m^3/s, a part of the flood wave is accepted by the free volume of the Trebinje Reservoir, without increasing opening of gates on bottom outlets.
- Further opening of the gates at bottom outlets continues if the level in the Trebinje Reservoir increases over 294 m a.s.l. or some other level recommended for periods of emergency events. Maximal discharge through bottom outlets is 800 m^3/s. In extremely rare cases, if it is not possible to evacuate the entire amount of water through the bottom outlets, opening of the radial spillway gates begins, with opening step of 10 cm every hour.

If the simulations show that the levels in the Trebinje Reservoir will rise above 295 m a.s.l. the user of the program immediately receives information that the flood wave cannot be accepted and mitigated in Trebinje Reservoir. Then, the program 'Flood mitigation' is restarted, in order to examine whether additional emptying of the Bileća Reservoir can further increase its protective function.

On the basis of numerous simulations and analyses of frequency of flood waves (Fig. 2) possibilities of flood wave mitigation in the Bileća Reservoir during the flood period were analysed. As a result dijagram of recomended water levels is proposed (Fig. 7). Purpose of this diagram is to help in defining water levels in the Bileća Reservoir, which are also required as the input data for Flood wave mitigation program.

- Red zone is protected space reserved for the situations which limit the normal functioning of the system (e.g. problems with function on HPP Dubrovnik). If water level gets into red zone all necesary measures should be carried out to reduce it as soon as possible.
- Orange zone is defined as a zone in which flood waves are mitigated and discharge is performed according planned management rules, or management rules deffined by the Flood wave mitigation program.

- Yellow zone is defined as a zone with precautionary measures and enhanced surveillance.

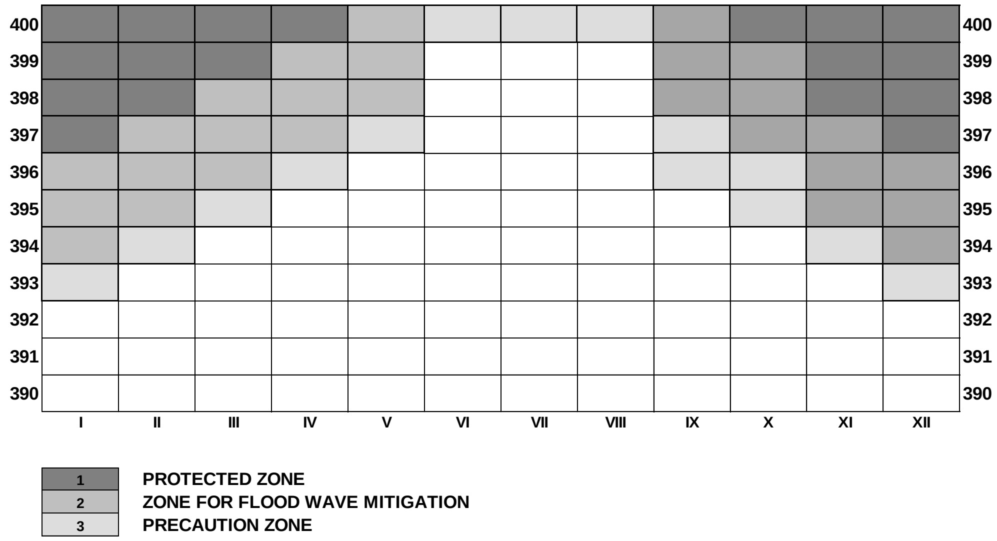


Figure 7. Dijagram of recomended water levels in Bileća Reservoir for flood period

In the normal working period diagram of recommended water levels in Bileća Reservoir differs from recommended levels in flood period. It is defined on the base of optimal management of hydropower plants according to the criteria of maximal energy production of the system. Calculations were performed for period 1956-2005 and one of the results was optimal water levels in Bileća Reservoir. These values were statistically analysed for each month. The probabilities of optimal water levels are presented in the graphical form (Fig. 8). Lower water level boundary connects the optimal levels of 10% probability of occurrence, while upper water level boundary refers to optimal levels of 90% probability of occurrence. This is very important diagram which should help in the management of Trebišnjica Hydrosystem. Water levels in Bileća Reservoir should be maintained within the defined boundaries.

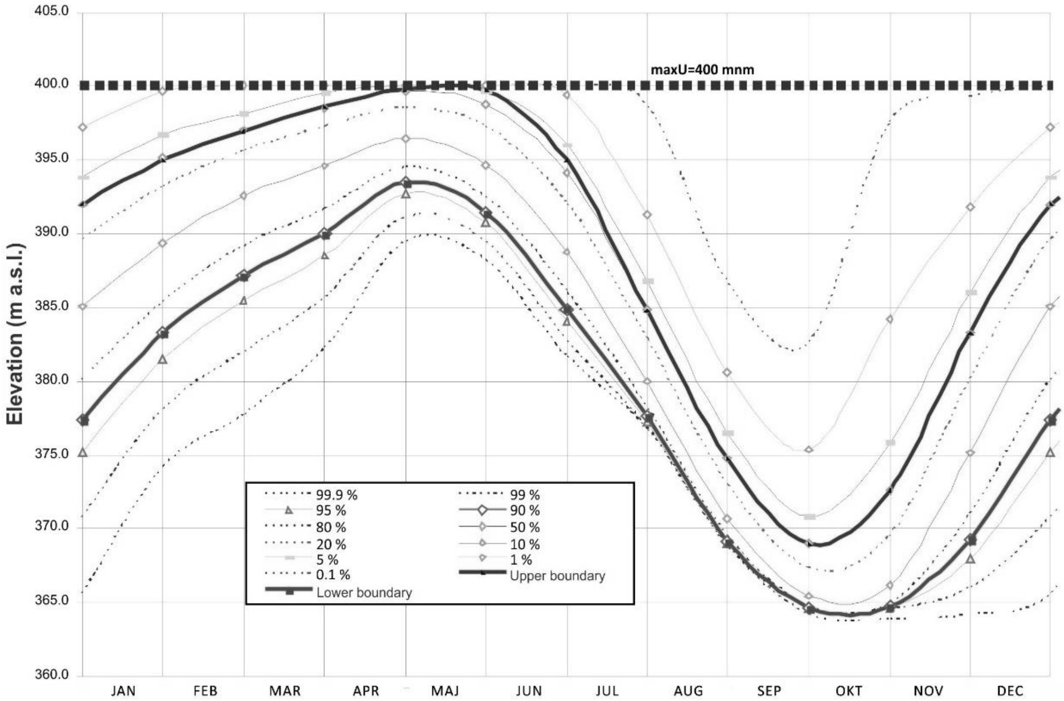


Figure 8. Dijagram of optimal water levels in Bileća Reservoir

5. CONCLUSIONS

Improvement of reservoir management using a real-time model allows significant increase of the effects of multipurpose systems. This improvement refers to:

- increasing the operability of the measuring and information system to be able for real-time management,
- creating the operative mathematical model for defining inputs into the system, and
- improving the decision algorithm.

For the system of two reservoirs and three hydropower plants on the Trebišnjica River, a Trebišnjica River Management Model has been developed. One of the important parts of this model is the Flood wave mitigation model. The model allows the user to obtain the most favourable management based on the data on the current state of the system according to the criterion of minimizing the maximal flows through the Trebinje Town. This is, in fact, a suboptimal solution for the simulated hydrological situation. Results of the model are rules for management of gates at spillways and bottom outlets of dams as well as the diagram of water flows through the Trebinje Town in flood period. The software is adaptive, so that management decisions can be constantly improved in accordance with obtaining new information on the shape and size of the input waves. In this way, the greatest mitigation of flooded waves is achieved, and the most successful active protection of Trebinje is realized in defined hydrological conditions.

In order to allow efficient management during the year, on the base of optimization analyses, diagram of optimal water levels in the Bileća Reservoir have been made. Also, on the base of simulations performed for different flood waves, diagram of recomended water levels in the Bileća Reservoir as well as recomended water levels for Trebinje Reservoirs (both reservoirs) are proposed. Adequate use of these diagrams can successfully compromise demands of different water users.

Further improvement of the model should be directed towards the development of the prognostic model. This is particularly important for the Flood wave mitigation model, because input flood waves would be flood waves predicted on the base of observed (measured) parameters instead of predefined synthetic waves used in current model.

It is very important to develop similar management models for other large water storage reservoirs, especially those situated upstream from settlements and urbanized river valleys. Management performed without previous control of management rules results in cautious management without the full use of all model performance.

REFERENCE

[1] B. Đorđević: „Water resources systems" (in Serbian), Naučna knjiga, Belgrade, 1990, 498p

[2] B. Djordjević: „Cybernetics in Water Resources Management", WRP, USA, 1993, 619p

[3] B. Đorđević, T. Dašić: „Method for determing the environmental flow downstream of the dams and water intakes" (in Serbian), Journal Vodoprivreda, Vol. 43, No. 252-254, 2011, pp.151-164

[4] B. Đorđević, T. Dašić: „Water storage reservoirs and their role in the development, utilization and protection of catchment", Spatium International Review, 2011, pp.9-15

[5] B. Đorđević, T. Dašić: „Expert system for operational planning and implementation flood control" (in Serbian), Journal Vodoprivreda, Vol. 47, No. 276-278, 2015, pp.187-202

[6] H. Kaladžisalihović, A. Bibović: „Numerical model of flood wave on river Neretva, FBiH" (in Serbian), Journal Vodoprivreda, Vol. 43, No. 252-254, 2011, pp.189-196

[7] P. Milanović: „Karst of Eastern Herzegovina and Dubrovnik Littoral", ASOS, Belgrade, 2006, 362p

[8] D. Pavlović, V. Vukmirović: „Statistical analysis of the short duration rainfall by the annual maxima method" (in Serbian), Journal Vodoprivreda, Vol. 42, No. 246-248, 2010, pp.137-148

[9] S. Prohaska, A. Ilić: New method for defining multiple coincidence of flood waves in complex river systems (in Serbian), Journal Vodoprivreda, Vol. 42, No. 246-248, 2010, pp.125-136

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ADVANCED GEOMEMBRANE TECHNOLOGIES IN NEW EMBANKMENT DAMS AND TAILINGS DAMS

Polyvinylchloride (PVC) geomembranes, with permeability much lower than traditional water barriers, have been used in embankment dams since 1959, for new dams and with covered geomembrane. The use of upstream exposed PVC geomembranes started at the beginning of the 1970ies, due to greater confidence acquired through using these relatively new synthetic materials, on dams having hard subgrades (concrete, RCC, asphalt concrete). Since the beginning of the new century, PVC geomembranes are used also on granular subgrades. Major assets of geomembranes in embankment dams are outstanding elongation properties allowing resisting settlements, differential movements, and earthquakes, which would destroy rigid water barriers such as concrete. Geomembranes are engineered against environmental aggression, and unlike clay cores are always available in the needed quantity/quality. The dam can be built on deformable foundations, with steeper slopes reducing the quantity of the fill, construction can be made in stages and completed in shorter times at lower costs. New patented anchorage systems employ extruded curbs with PVC anchor strips, or anchor trenches, or grouted or earth anchors. These techniques have been installed on more than 10 dams worldwide. This paper shows the new anchoring techniques, including a technology allowing installing geomembrane systems underwater, even with flowing water.

Keywords: Geomembrane, polyvinylchloride, waterproofing, fill dam, tailings dam, GFRD, GFED

1. INTRODUCTION

Geomembranes are prefabricated, practically watertight flexible synthetic materials used in hydraulic projects for more than half a century. They are engineered to ensure adequate properties that allow them sustaining the loads imparted during service; they are manufactured in the controlled environment of a factory, ensuring that the properties established by design are constantly maintained throughout the whole production lot by computer-controlled manufacturing processes, and constantly checked in the laboratory. At the job site, the properties, especially those crucial for good performance, such as tensile properties, are checked with standard methods to verify that they are compliant to specifications and have remained constant during transport and storage. Since properties of geomembranes are independent of weather conditions during construction, the characteristics of the geomembrane water barrier when the dam is completed are consistent and verifiable. In Concrete Face Rockfill Dams (CFRDs) or in dams with clay or asphalt concrete cores, the characteristics of the water barrier, on the contrary, are influenced by construction conditions, construction procedures, and workmanship.

Geomembranes are of particular interest when embankment dams are at stake. The major assets of geomembranes, in particular Polyvinylchloride (PVC) geomembranes that are by far the most used waterproofing material for large dams [2], are their outstanding elongation properties, which allow resisting settlements, differential movements, and seismic events, which would destroy more rigid water barriers such as concrete facings (CFRDs); the possibility of engineering them to resist environmental aggression, and their permanent, consistent quality over large quantities, enables high quality installations.

The seaming of PVC geomembranes enables the face to be one large continuous waterstop, eliminating the need for the multiple defence lines of waterstops needed for a CFRD. Furthermore, geomembrane connections to the ancillary concrete structures can be designed to accept large differential movements ensuring watertightness at the perimeter. A geomembrane facing allows for building the dam on highly deformable foundations, allows for steeper upstream face, which means less volume of fill, and a shorter, smaller diversion tunnel.

From a construction standpoint, geomembranes have the advantage of reducing construction times, constraints and costs. In CFRDs, the installation/construction of the reinforced concrete face slabs, and the placement of copper and PVC waterstops, can have considerable impact on the overall construction schedule. In dams with clay or asphalt concrete cores, the construction of the dam body and the construction of the central core are strictly related. There are constraints imposed by weather conditions, or by disruption in placement of the filter material, or in placement/compaction of the impervious core. This affects the overall rate of construction of the clay or asphaltic core dam. On the contrary, installation of a PVC geomembrane system is practically unaffected by weather. The geomembrane system does not need to be a serial task in the construction process, and its installation can be done in parallel with other activities, so that it is independent of most construction or operational constraints of the dam. The geomembrane can be installed when the dam is completed, or installed on the lower completed part of the dam while construction of the fill is ongoing in the upper part. Geomembrane installations offer the additional benefit that if there are floods during construction, the waterproofed lower part of the dam will be a barrier against the flood, increasing the safety of the project.

The complexity of the techniques needed to construct the waterproofing system must be taken into consideration when evaluating the time and costs of embankment dams. Inadequate placement of the waterstops and inadequate construction of the connections to the ancillary concrete structures are critical, and can have highly negative effects on the future performance of the dam. Especially when the main contractor has little or no experience in construction of such dams, adopting a geomembrane system is “forgiving” and will avoid the problems related to inadequate construction, which could require future time consuming and expensive repairs.

Geomembranes were first used in construction of new embankment dams (1959 in Europe and in Canada). In these pioneering projects, when synthetic materials were relatively new to dam engineers, the geomembranes were covered. In the 1970s, once confidence in PVC geomembranes’ behaviour and reliability had increased, a cover layer was no longer mandatory and exposed geomembranes began to be used. At present, the use of PVC geomembranes in new construction includes facings for embankment dams, RCC dams, tailings dams, external waterstops for peripheral and vertical joints in CFRDs, and external waterstops for joints between monolith blocks in RCC dams. In rehabilitation, all kinds of dams have been waterproofed around the world, in both dry and underwater installations. This paper focuses on upstream exposed geomembrane systems applied to new fill dams to provide watertightness at the upstream face. Such dams have become known as Geomembrane Facing Rockfill Dams (GFRDs) and Geomembrane Facing Earthfill Dams (GFEDs).

2. THE GFRD AND GFED: CONCEPT AND COMPONENTS

Carpi upstream geomembrane systems for Geomembrane Facing Rockfill Dams (GFRDs) and Geomembrane Facing Earthfill Dams (GFEDs) are based on the concept of providing a flexible watertight barrier that can elongate well beyond the maximum expected deformations of the dam body. The flexible water barrier is anchored to the dam face with a site-specific anchorage system as described in this chapter and in the case histories that follow, and at boundaries with a perimeter seal designed to resist differential settlements while maintaining watertightness.

2.1 THE CONCEPT

In these dams, the water barrier is a flexible synthetic composite geomembrane, also called “geocomposite”, placed at the upstream face in exposed position. The geocomposite is anchored against uplift over the face of the dam, and is sealed at all peripheries by a continuous seal, which is watertight against water in pressure at submersible boundaries, and against rain, snowmelt, and waves at crest. The design of the anchorage and sealing system is conceived to be highly deformable, to accommodate all foreseen settlements that can occur in the dam body, and differential movements between the deformable dam body and the rigid concrete appurtenances.

The presence of the upstream geomembrane allows adopting a very simple layering for the dam:

- Dam body: zoning is not strictly required, a single fill material can be used

- Drainage layer: depending on the design of the dam, generally the thickness of this layer can be reduced

- Base/anchorage layer: this single layer acts as support to the geomembrane and incorporates the face anchorage system; depending on the design of the dam, this layer can also solve the function of drainage layer

- Waterproofing liner: the geocomposite.

2.2 THE WATERPROOFING LINER

The main issue when selecting the type of geomembrane for a GFRD is the tensile behaviour. A study by Giroud & Soderman [1] comparing the behaviour of different types of geomembranes subject to differential settlements has shown that an appropriate combination of tensile strength and strain is essential. This optimal combination depends on the shape of the tension-strain curve of the geomembrane; the co-energy, which is the area between the tension-strain curve and the tension axis, quantifies the ability of a geomembrane to withstand differential settlements. The larger the area, the more performing will be the geomembrane in this respect. Plasticised PVC geomembranes tension-strain curve exhibit by far the largest area as compared to High Density (HDPE) and Low Linear Density Polyethylene (LLDPE) geomembranes. Consequently, the factor of safety in respect to stresses associated with differential settlements is significantly higher if a PVC-P geomembrane is adopted.

The waterproofing liner used in GFRDs and GFEDs is SIBELON® CNT, consisting of a thick high quality, specifically formulated, PVC geomembrane, 2.5 to 4 mm thick depending on the projects, which provides watertightness, and of a backing geotextile, mass per unit area 500 to 700 g/m^2, heat-bonded during manufacturing to the geomembrane. The geotextile provides anti-puncture protection, enhances thermal stability and stability on inclined slopes, and provides some drainage capability. SIBELON® CNT geocomposites are produced with a proprietary formulation under ISO 9001 certification. Large thickness is required to increase service life and survive mechanical loads during construction and operation.

2.3 FACE ANCHORAGE

The face anchorage system maintains the waterproofing liner stable of the dam face, resisting uplift by backpressures, uplift by wind, and variations of water level and waves. The face anchorage system is different depending on the methodology of dam construction and on the type of base layer on which the geocomposite is laid.

If the upstream base layer is made by extruded porous concrete curbs, the anchorage system consists of PVC anchor strips embedded in the extruded porous concrete curbs. This system is described in chapter 3.

If the upstream base layer is not made with curbs, depending on site-specific construction method and on the type of uppermost layer, the anchorage system can be of two types: PVC anchor strips embedded in trenches excavated in the base layer at site-specific spacing, as described in chapter 4.1, or deep anchors that are driven into the embankment after it has been completed, according to site-specific patterns, as described in chapter 4.2.

All the anchorage systems described in the paper are Carpi patents.

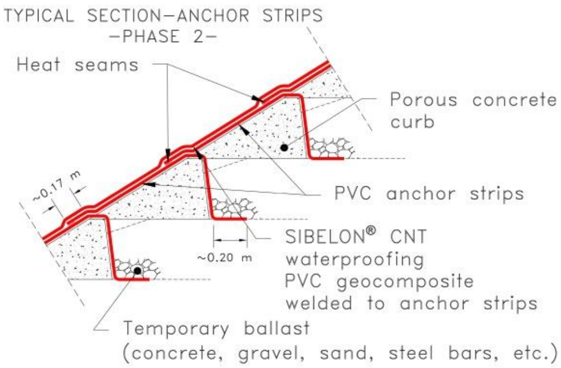
2.4 PERIMETER SEALING

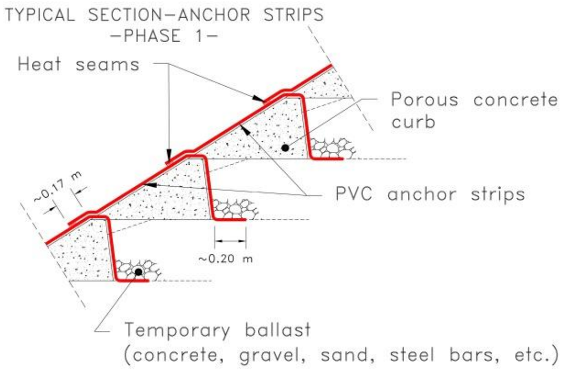
At all peripheries, the SIBELON® CNT geocomposite is anchored by perimeter seals to avoid water infiltrating behind the liner. If made on concrete, the perimeter seal is of the tie-down type, which has been tested to water pressures exceeding 800 meters. If made on less cohesive material such as clay, the seal is of the embedded type, generally made laying the geocomposite in a trench then filled with impervious material. The case histories that follow will describe how the seal was designed in function of the underlying substrate.

3. FACE ANCHORAGE BY PVC STRIPS EMBEDDED IN CONCRETE CURBS

This system applies to new construction of embankment dams where the dam body is raised placing the fill against extruded porous concrete curbs that form a draining finishing layer at the upstream face of the dam. A PVC anchor strip fixed to one curb overlaps the PVC anchor strip fixed to the preceding curb, and the overlapping strips are heat-welded to form continuous anchor lines (Figure 1a). The distance between the anchor lines is designed in function of the design uplift loads. The waterproofing geocomposite is then deployed over the PVC anchor lines and watertight heat-welded to them (Figure 1b).

The advantage of adopting extruded porous concrete curbs is that the curbs allow constructing in a short time a regular slope not erodible by rainwater, which can be steep and has constant characteristics. The curbs also provide a stable and regular base layer for the waterproofing geocomposite, and allow using a simple face anchorage method that is constructed by the waterproofing contractor concurrent with raising of the fill by the main contractor. In such a way, when the fill is completed also the face anchorage system for the waterproofing geocomposite is completed.





Figures 1a and 1b. Face anchorage with PVC anchor strips embedded in curbs

An outstanding recent project adopting this technology is Nam Ou VI rockfill dam in the Lao PDR, which is part of the Nam Ou VI Hydropower Project with a total installed capacity of 180 MW. The scheme includes an 88 m high rockfill dam, designed as a GFRD and the highest of its type in the country. The only water barrier is an exposed SIBELON® CNT geocomposite. The dam body consists of slate rockfill for the whole cross-section except for the bottom drainage facility. The upstream slope inclination is 1.6H/1V, the net upstream face surface is 38,000 m^2.

The original design envisaged a polyethylene geomembrane sandwiched between two anti-puncture geotextiles, placed on a 3 m wide granular base layer, and covered by precast concrete elements. A study carried out by the designers of the dam showed that the net expected settlement and horizontal displacement due to the embankment load and the reservoir hydrostatic head would be in the order of 100 mm vertical and 80 mm horizontal. Such expected deformations could exceed the resistance of a concrete cover, possibly causing damage to the polyethylene geomembrane. The initial solution was modified, eliminating the concrete cover and leaving the geomembrane exposed. To ensure long-term successful performance in an exposed position, instead of a polyethylene geomembrane, a SIBELON® CNT geocomposite was selected, consisting of a 3.5 mm thick PVC geomembrane, heat-bonded during fabrication to a nonwoven, needle-punched, continuous filament polypropylene geotextile, 700 g/m^2. The final design substituted the granular base layer with an extruded porous concrete curbs facing, on which the SIBELON® CNT geocomposite was placed in exposed position, and anchored to the dam body by the above described system using SIBELON® PVC anchor strips embedded in the curbs, forming parallel anchorage lines at 6 m spacing, as shown in Fig. 2.

Figure 2. Nam Ou VI GFRD: The SIBELON® PVC anchor strips placed on the extruded curbs

The SIBELON® CNT geocomposite was then deployed over the face of the dam and anchored by heat-seaming it to the anchor strips. Adjoining SIBELON® CNT geocomposite sheets were watertight heat-seamed, forming one watertight facing.

Figure 3. Nam Ou VI GFRD: Seaming of SIBELON® CNT geocomposite to anchor strips and of adjoining geocomposite sheets

A double mechanical seal of the so-called tie-down type confines the geocomposite at the peripheral plinth. Watertightness against water in pressure is attained by compressing the geocomposite unto the concrete subgrade with a flat stainless steel batten strip secured by stainless steel anchor rods embedded in chemical phials at regular spacing. Regularising resin, rubber gaskets, stainless steel batten strips and splice plates are used to achieve even adequate compression necessary for watertightness. The design of the peripheral sealing system is such as to grant the capability of following the differential deformations between the dam body and the concrete plinth.

The dam was constructed in three separate stages, two for the dam body and one for the parapet wall. Construction of Stage 1, embedding the face anchorage system, started on July 23rd 2014 and was completed on November 2nd 2014, immediately followed by installation of the geomembrane system, completed in 24 days for about 13,700 m^2 of upstream facing, a fraction of what a concrete facing would have required. Construction of Stage 2 started on December 12th 2014 and was completed on April 14th 2015, immediately followed by installation of the geomembrane system, completed in 28 days, on about 23,000 m^2. The settlements that occurred during construction of Stage 2 of the dam body were sustained by the deformable geocomposite.

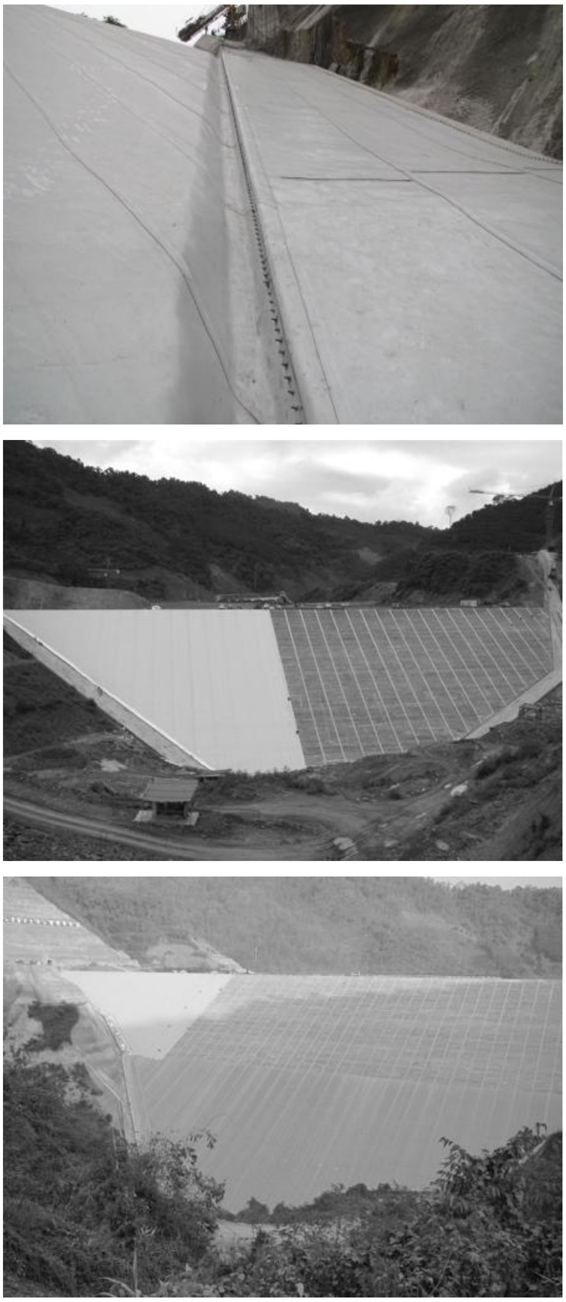


Figure 4. Nam Ou VI GFRD: Watertight perimeter seals at plinth, and Stage 1 and Stage 2 waterproofing

Impoundment of the reservoir started after completion of Stage 2, in the afternoon of June 17th 2015, less than eleven months after construction of the dam body had started. Lining of the parapet wall started on April4th 2016 and was completed on April 26th 2016. In 2017 the owner reports [3] "*After one year's operation, especially after Oct 2016, the dam has come to the longterm operation at normal storage level or high water level. The parameters from the monitoring devices show the dam deformation tends to be stable and dam performs well. … construction is speeded up greatly by the application of geocomposite*".

The same system was installed at the upper part of Runcu GFRD in Romania (2015) and in three mining projects, at Sar Cheshmeh tailings dam raising in Iran (2008), at Ambarau hardfill dam in DR of Congo (2017), and at an ongoing tailings dam project in South America where the geomembrane system is reaching a height of 118 meters, with an expected total height of the dam of 230 m.

Figure 5. Geocomposites on extruded curbs: at left Runcu GFRD in Romania, at right a tailings dam in South America

4. FACE ANCHORAGE IN GRANULAR SUBGRADES

When the base layer for the waterproofing geocomposite is not made with curbs but with granular materials (e.g. roller-compacted soil, or gravel stabilized with lean concrete and compacted with a vibratory plate, or cemented material), the anchorage can be made by PVC anchor strips embedded in trenches, or by deep anchors driven into the granular layers.

4.1 FACE ANCHORAGE BY PVC STRIPS EMBEDDED IN TRENCHES

This method can be adopted with different types of base layers, provided they are stable and with a regular surface. The trenches, of site-specific dimensions and spacing, in function of the foreseen loads, are formed or excavated in the base layer, and PVC anchor strips are placed inside the trenches, to form parallel PVC anchor lines, like the system with the extruded curbs. Configurations are site specific depending on the construction procedures adopted by the main contractor. The trenches are generally backfilled with porous concrete, or concrete, or granular material, to provide the right amount of ballast needed to keep the anchor strips in place against wind uplift. The waterproofing geocomposite is then deployed over the anchor lines and watertight heat-seamed to them, as for the extruded porous concrete curbs method.

At Murdhari 36 m high rockfill dam in Albania the design was modified from a dam with an asphalt core to a GFRD when the project had already started. The design was modified with the objective of making construction of the dam safer, faster, easier to build, and less expensive. Since construction of the fill was already ongoing, there was no possibility of adopting the curbs, so a system similar to the curbs was conceived: the base layer was made with porous concrete slabs, leaving a trench between adjacent slabs for placing the PVC anchor strips. Starting from the bottom and proceeding upwards, the trench was gradually filled by superimposed porous concrete blocks: after the first bottom block was completed, a PVC anchor strip was placed on it and permanently anchored by another porous concrete block placed on top, and so on up to the crest. In practice, instead of placing porous concrete horizontally by a curb extruder, porous concrete is constructed within the vertical trench, in superimposed small blocks embedding the PVC anchor lines. The waterproofing geocomposite was then deployed and heat-seamed to the anchor strips. Waterproofing works started on September 16th 2013 and were completed on October 19th 2013, a total of 6,770 m^2 of geocomposite installed in about one month.

At Bulga coal mine dam in Australia, a 460 m long and 17 m high zoned earthfill embankment, the base layer was granular material, where trenches were excavated to insert the PVC anchor strips and the backfilled. A drainage geonet was placed between the base layer and the geocomposite.

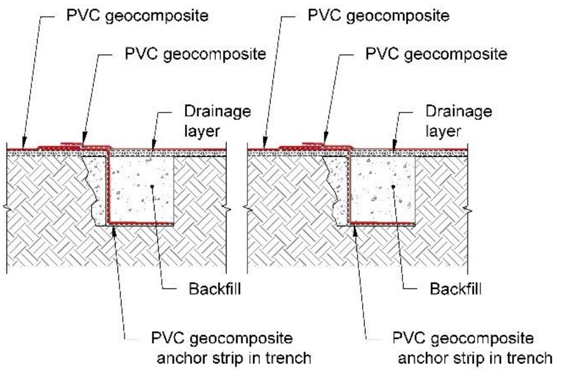


Figure 6. Geocomposite anchored by PVC anchor strips embedded in trenches at Bulga GFED (2016)

At the upstream toe, the PVC geocomposite extends into a trench providing perimeter anchorage and a positive cut-off for foundation seepage minimisation.

The same system was installed at the embankment forming the Kohrang head pond in Iran (2004), in the lower part of Runcu GFRD in Romania (2015), and at some of the slopes and invert of the 18 Water Saving Basins of the Panama Canal Expansion (2014-2015).

Figure 7. Murdhari GFRD (left) and 9 of the 18 Water Saving Basins of the Panama Canal Expansion

4.2 FACE ANCHORAGE BY DEEP ANCHORS

Deep anchors are adopted in base layers of granular material or cemented material, where they are driven into the embankment after it has been completed, placed in regular patterns, at spacing calculated in function of the loads acting at the dam face. Depending on the characteristics of the base layer into which they are driven, deep anchors can be of the grouted type, or of the Duckbill type.

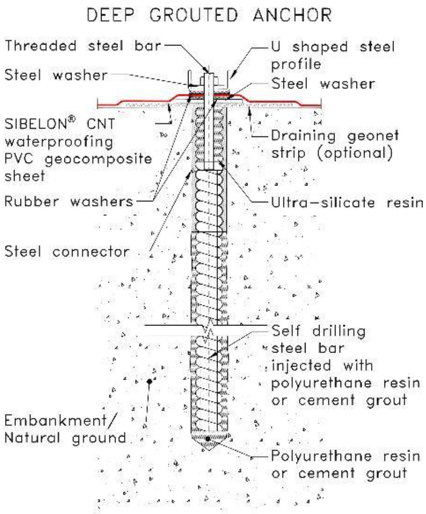


Figure 8. Deep grouted anchors used at the Panama Canal Water Saving Basins and at Ambarau dam

Regardless of the type, deep anchors are equipped with stainless steel anchor rods that stick out of the dam face. The waterproofing geocomposite is punched over the anchor rod, and covered by a watertight patented end piece. The end piece consists of a stainless steel disk, of an anti-puncture geotextile disk, and of a geomembrane cover patch, watertight heat-seamed to the geocomposite.

Deep grouted anchors have been used at some locations of the 18 Water Saving Basins of the Panama Canal Expansion and of Ambarau hardfill dam, and at Filiatrinos (Greece 2015) hardfill dam. Duckbill anchors have been adopted for the rehabilitation of Vaité earthfill dam in French Polynesia (2011).

5. UNDERWATER INSTALLATION

In the last few years, a totally innovative underwater waterproofing technology, SIBELONMAT®, has been developed to provide/restore watertightness in embankment dams and canals without impacting on operation. The system consists of two watertight geomembranes connected to form a mattress.

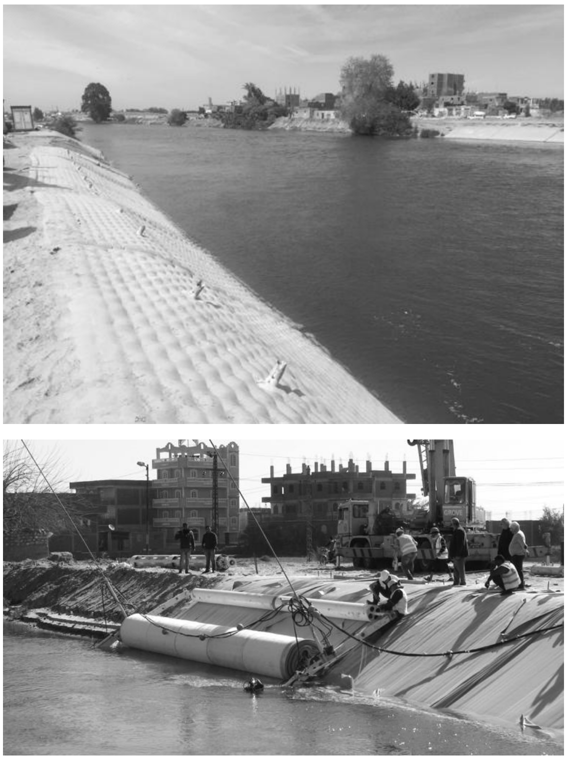


Figure 9. SIBELONMAT® installed underwater at Ismailia canal, Egypt 2016

The connection of the geomembranes is designed to allow injecting between the two a ballasting filling material such as inexpensive cement grout. The lower geomembrane provides watertightness, the grout provides the ballast to anchor the mattress, and the upper geomembrane provides containment of the grout, protects the ballast during operation, and in canals improves hydraulic efficiency. SIBELONMAT® is prefabricated in 10 m wide mattresses having custom-made length to minimise junctions and facilitate placement. Adjoining mattresses are joined by watertight heavy-duty zippers pre-attached to each mattress during fabrication. Installation can be performed totally underwater and without stopping operation or reducing water speed.

This new technology has already been successfully installed on two canals with no reduction of water speed. SIBELONMAT® can be considered also for new construction of embankment dams, to provide an impermeable upstream facing or an impermeable blanket even on very aggressive irregular subgrade.

REFERENCE

[1] Giroud, J.P., and Soderman, K.L.: „Comparison of Geomembranes Subjected to Differential Settlement“, Geosynthetics International, Special Issue on Design of Geomembrane Applications, 1995 Vol. 2, No. 6, pp. 953-969

[2] ICOLD - International Commission on Large Dams, Bulletin 135: „Geomembrane Sealing Systems for Dams - Design Principles and Review of Experience“, Paris, 2010

[3] TANG C., and Zhang, G.: „Design and construction of composite geomembrane face rockfill dam of Nam Ou 6 hydropower station“, IV International Symposium on rockfill dams – SIBE, Belo Horizonte, Brazil, 2017.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NUMERICAL ANALYSIS ON "ST. PETKA" DAM

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

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

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

Keywords: arch dam, safety, stresses, displacements

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 [Tanchev, 2005; Novak et all, 2007]. In Macedonia, up to now are constructed 27 large dams. Different types of dams are represented, having in consideration the various geological, topographical and hydrological conditions, among which 19 are embankment dams, 6 concrete arch dams and 1 concrete multiarch dam. The stored water is used for meeting the demands for water supply of population and industry, irrigation, production of electricity, flood and erosion control, provision of minimum accepted flows, recreation and tourism. The total stored water volume is about 2.4×10^9 m^3. The assessment of the stability and the behaviour of the dam during construction, at full reservoir and during the service period is of vital meaning for this type of structures. In 2012 was completed “Saint Petka” arch dam, in nearby 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•10^{6} m^{3}. The main purpose of the dam is electric power production. The “St. Petka” dam was commissioned in August, 2012.

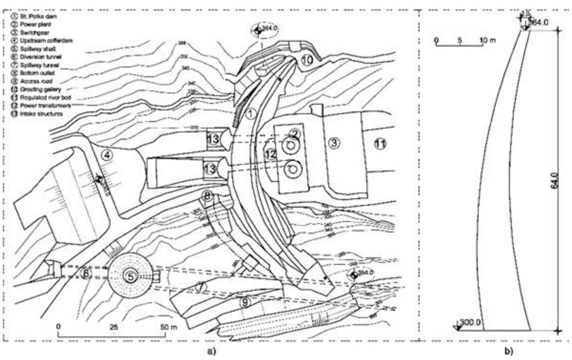

Figure 1. “St. Petka” dam. a) layout and b) typical cross-section

3. DAM MODELLING

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

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

3.1 nput parameters for the materials

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

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

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

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

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

Table 2. Concrete parameters

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

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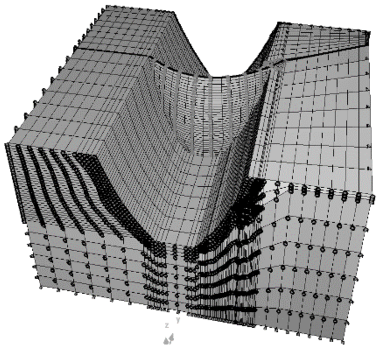
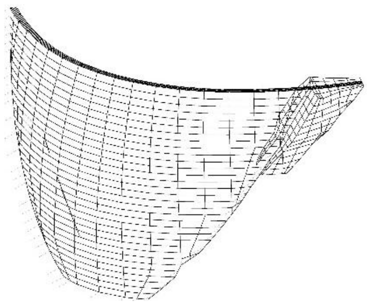
3.2 MODEL GEOMETRY

The numerical model is composed of the dam body and the rock foundation. The rock foundation boundaries are adopted according to literature [ICOLD, 1987]. More detail, the numerical model is composed of dam body, limited by the dam site banks, and rock foundation, with length upstream and downstream of the dam central cantilever in interval (1÷2)H, where H denotes dam height apropos a length of 100 m is adopted, while the rock foundation under the dam is adopted at depth of 65.0 m (Fig. 2a). By such parameters is defined the non-deformable boundary condition (displacements in X, Y and Z direction are fixed at the lowest section).



a)



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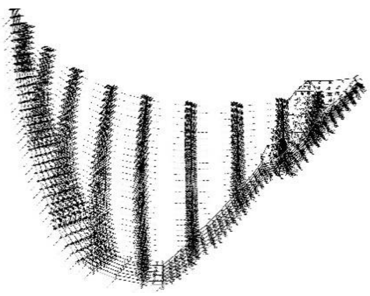

c)

Figure 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

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).

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).

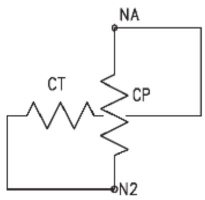

Figure 3. Interface element of type “spring”

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

On the current level of development in geotechnics, several approaches of shear strength along contact zones testing are known, but there are still cases when it is very usual to adopt or assume them, and very often this problem is not even treated. Along with this, it is very difficult to conclude how close is the prognosis of the parameters to the actual conditions which are expected in the phase of exploitation of the structures. The theory and methodologies for determination of shear strength along discontinuities are also developed [Goodman 1974]. Furthermore, there are some developed methodologies to define shear strength along interfaces concrete-rock mass in a phases of design for concrete dams [Anđelković, 2001; Jovanovski M. et all, 2002; Barton and Kjaernsli, 1981; Tanchev, 1989; Tanchev 2014]. The investigation and adoption of input data for the specified stiffness properties can be done by specially arranged laboratory direct shear test by applying the Hoek's apparatus [Mitovski, 2015]. Namely, the values for the lateral constant were adopted on base of laboratory and „in situ" testing on the shear strength parameters by Hoek apparatus at the contact concrete/rock foundation and concrete/grout mixture in the dam joints (Fig. 5). The output results from one sample from the testing are displayed on Fig. 5. The adopted values are $C_p=16\times10^6$ kN/m^3 and $C_t=1\times10^6$ kN/m^3 for contact dam-foundation and $C_t=5\times10^6$ kN/m^3 for contact concrete/grout mixture.

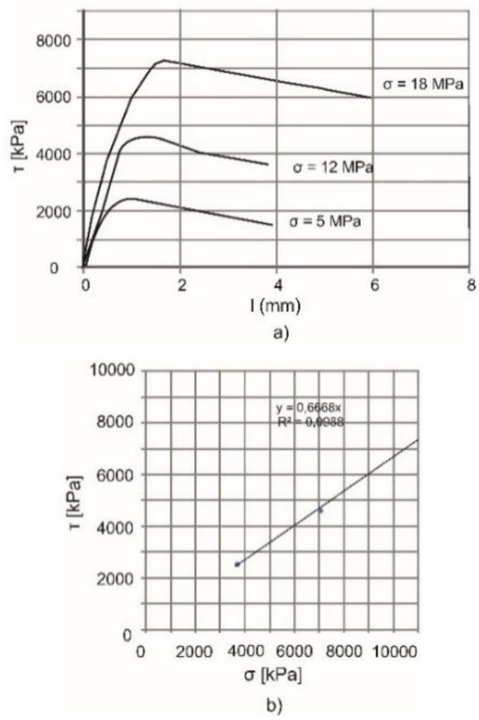

Figure 5. Output results for testing sample 378 I/A, contact concrete-grout mixture at constant normal load of 6, 12 and 18 MPa. a) Dependence shear load-displacement, b) dependence shear/normal load τ=σ(s)

3.4 DAM LOADING

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

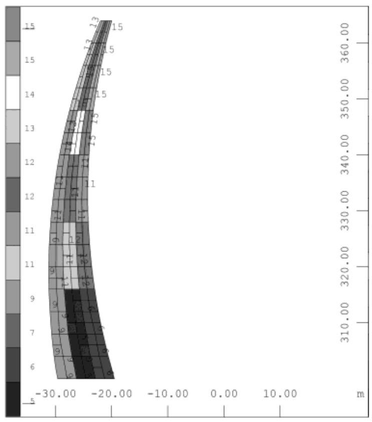

Figure 6. Temperature loading, applied on the dam

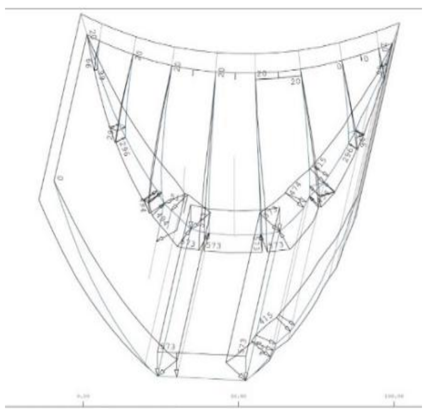

Figure 7. Water loading, applied on the dam

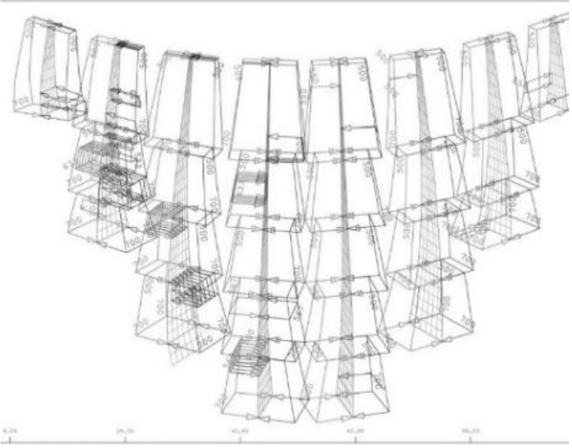

Figure 8. Grouting load, applied along surface of both sides of the dam joint

4. OUTPUT RESULTS

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

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

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

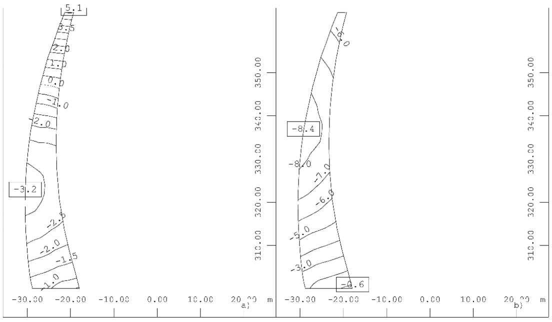

Figure 9. Horizontal displacements in the dam central cantilever [mm]. a) Isolines of displacements after dam construction; b) Isolines of displacements upon grouting stage

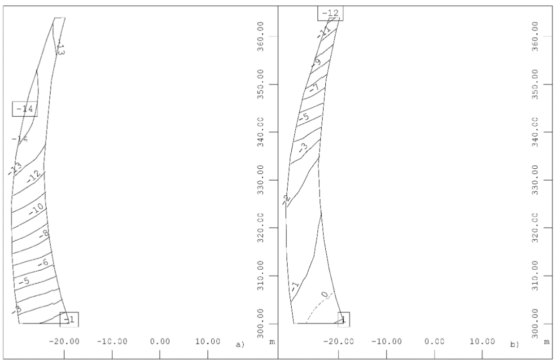

Figure 10. Horizontal displacements in the dam central cantilever [mm]. a) Isolines of displacements before reservoir filling; b) Isolines of displacements at full reservoir

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

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

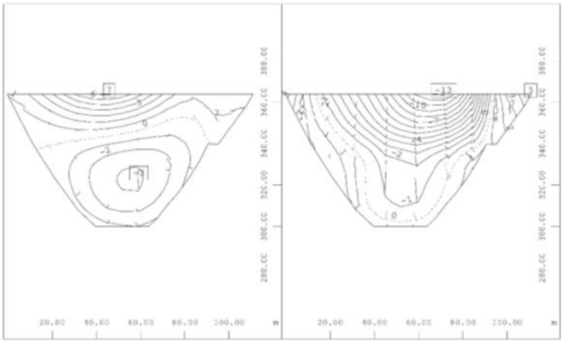

Figure 11. Horizontal displacements in longitudinal section in dam axis [mm]. a) Isolines of displacements after construction; b) Isolines of displacements at full reservoir

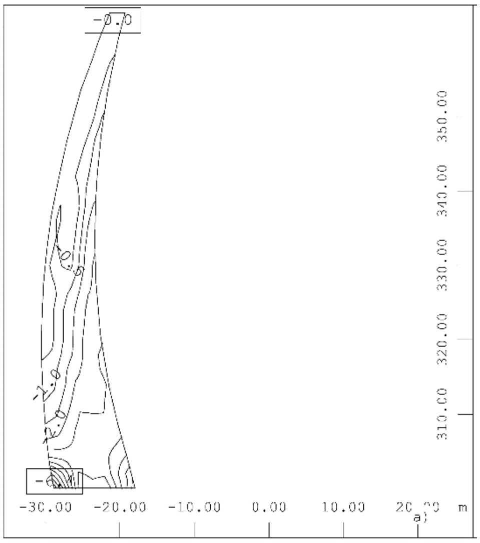

Figure 12. Isolines of principal stresses σ3 in the central dam cantilever after construction

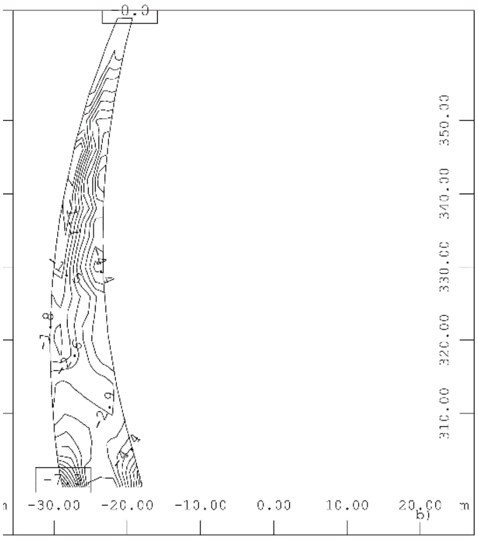

Figure 13. Isolines of principal stresses σ_3 in the central dam cantilever at full reservoir

5. CONCLUSIONS

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

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

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

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

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

REFERENCE

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

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

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

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

[5] Goodman R., "Methods of Geological Engineering in Discontinuous Rocks" West Publishing Company, New York, 1976

[6] Hoek E., Practical Rock Engineering, 2000

[7] Hudson J., Harrison J."Engineering Rock Mechanics - An Introduction to the Principles", Pergamon Press, Elsevier Science Ltd., 1997

[8] ICOLD Bulletin no. 145, The physical properties of hardened conventional concrete in dams, January, 2009

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

[10] Mitovski S., (2015) PhD thesis, Static analysis of concrete dams by modelling of the structural joints, Ss Cyril and Methodius University, Civil Engineering Faculty – Skopje

[11] Novak P., Moffat A. I. B., Nalluri C., Narayanan R. "Hydraulic structures", Taylor & Francis Group, London, 2007.

[12] Synthesis Elaborate on performed investigation and testing for concrete arch dam at dam site "St. Petka" on river Treska, Civil Engineering Institute "Macedonia", 2004.

[13] Tanchev L., (2014) Dams and appurtenant hydraulic structures, Second edition, A.A. Balkema Publ., CRC press, Taylor & Francis Group plc, London, UK

[14] Tanchev Lj. (1989) Static analysis of rockfill dams. Studentski zbor, Skopje(in Macedonian)

[15] Jovanovski M., Gapkovski N., Anđelković Vl., Petrović Lj.: Some possibilities for determination of bedrock-concrete interface shearing strength in Hoek's box. Proceedings from the First symposium of Macedonian Association for Geotechnics, Ohrid, 2002, pp.78-86 (in Macedonian)

UPGRADING OF SEISMIC OSCULATION SYSTEMS ON “PIVA” DAM

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Strong earthquakes always draw attention to the behavior of engineering structures of major importance, such as large dams, in which the assessment of seismic risk is a very difficult task, mainly because of the lack of real records from the impact of strong earthquakes on this type of engineering structures and lack of data on their response to such type of excitement.

Modern, digital systems for seismic oscultation enable continuous and reliable monitoring of the seismic activity and recording the occurred earthquakes. The "Piva" dam is one of the highest concrete arch dams in the world and as early as its putting into operation in 1976, instruments for seismic oscultation were installed - 6 analogue SMA-1 accelerographs, supplied with a starter which is automatically triggered at a predetermined acceleration level. The results from the measurements are recorded on a film.

In 2013, given the importance of the construction for the Montenegrin economy, the seismic oscultation network was upgraded with the installation of equipment for digital recording of earthquakes manufactured by Kinemetrics - USA, in order to provide high quality data on the seismic activity and behavior of the dam during seismic actions.

Keywords: seismic monitoring, high dam, network, earthquake, damage

1. INTRODUCTION

The data on the ground motion during earthquakes in the close region in which the dam is located as well as the knowledge of the structural behavior are the basis for the evaluation of the seismic risk associated with the dam. This holds not only for definition of the earthquake parameters and design criteria but also for all the remaining dynamic tests. Without these data, all investigations and analyses would be based on assumptions only. The best way of obtaining high quality and reliable data is installation of a network with as many instruments as possible for recording of soil displacements, i.e., structural response to the seismic effects.

Lately, seismic monitoring has been of a great importance, not without reason, since the data obtained during the osculation contribute to the increase of the safety of the dam when exposed to seismic effects.

According to their genesis and effect on engineering structures, earthquakes represent very complicated phenomena of a big hazardous potential so that, within a relatively short time of action, they can cause extensive damage and even failure of structures. Considering the history and intensity of seismicity of Montenegro, there is a permanent threat related to earthquake occurrence so that preventive actions should be started as early as possible and should be permanently applied and upgraded. Many dams worldwide are constructed in areas characterized by high seismic activity. During their operation, there have occurred strong earthquakes causing smaller or heavier damages, while in some cases, even failure of dams (1958 Habgen, Montana, USA; 1964 Eklutna, Alaska, USA; 1971 Van Norman, California, USA; 1991 Shigh Kang, Taiwan). Here, one should also mention the possibility of occurrence of induced earthquakes due to the effects of the water reservoirs on the surrounding rock area. These can also have a significant magnitude (M = 6.4, Koyna, India) and can cause considerable damage.

2. MAIN TECHNICAL DATA ON HYDROELECTRIC POWER PLANT „PIVA“

HPP „Piva“ dam (i.e., „Mratinje“ dam) is among extremely high dams and among the 25 highest arch dams in the world (Fig.1). The stress and deformation state of the dam is dominantly affected by the reservoir level (hydrostatic pressure) and the temperature of the dam body. The rock massif at the dam location is composed of massive limestone originating from the Middle Triassic that was exposed to intensive tectonic motions. The structure of the damage in the form of cracks caused by these disturbances stretches in the direction that is approximately perpendicular to the river course, which is favourable from the aspect of safety against sliding. The cracks divide the rock mass into large blocks, but without crushed zones in their surrounding and are partially filed with clay and calcite. The size of the crack openings is reduced with depth so that they lose the characteristics of mechanical discontinuities. The mechanical characteristics of the rock massif at the sides of the dam are variable. The deformation modulus is within the range of 15000 N/mm^2 to 7000 N/mm^2 going from the bottom of the canyon toward the dam crest, with the exception of one part in the last fourth of the height on the right side where it drops to 2500 N/mm^2, wherefore in that zone, a massive abutment is constructed. Through the system of horizontal concrete piles, this abutment rests on deeper parts of the rock massif of approximately the same geotechnical characteristics as those at the corresponding heights of the left side.

Figure 1. Dam "Piva"

HPP „Piva“ dam represents concrete, asymmetric, arch dam with a double curvature (Fig. 1). Construction of this dam, created an artificial water reservoir used for production of electricity in the accompanying hydroelectric power plant „Piva“. The topographic, geological, geotechnical and geophysical documentation show that the dam profile is asymmetrical and has a pronounced „V“ shape meaning that HPP „Piva“ dam is a surface bearer with pronounced arch effect. The arch is asymmetrical and of variable thickness and curvature. The extrados and intrados of the arch are defined by the fragments of the circle, while the central line, i.e., the arch axis represents an ellipse, which is a combination of the circular curvatures drawn from three centres. The foundation joints of the dam represent parabolic curves. Such defined shape of the horizontal cross-section – arch has enabled the most favorable fitting of the dam into the profile considering the stretching of the main tectonic lines on which the entire stability of the structure depends. At the dam foundation, from the galleries, consolidation, connection and contact injection as well as sealing injection of the triple and single grouted apron was performed down to the depth of up to 150 m'. Dam body consists from 18 units (consoles) and there are there are 5 inspection galleries at levels: 482, 522, 562, 602 and 642 mnm. The main technical data on the dam a represented in Table 1.

Table 1 Technical data for "Piva" Dam

Year of construction	1976
Construction hight	220.00m
Length of arch axis at the crest	268.56 m
Length of arch axis at the bottom, at level 478	63.05 m
Thickness of the cross-section at the crest crown	4.51 m
Thickness at the crest abutment	6.46 m
Maximum thickness at the crown cross-section, at level 522	22.90 m
Maximum thickness at the bottom, at the supporting cross-section	45.00 m
Central angle of the arch axis at the crest	84.78º
Central angle of the arch axis at level 478	55.50º
Radius of the central arch axis at the dam crest	168.49 m
Radius of the central arch axis at level 478	54.52 m
Dam crest level	678.00 mnm
Maximum backwater level	677.50 mnm
Normal backwater level	675.25 mnm

Results of the study of the seismicity of the HPP “Piva” location done in 2008 (Civil Engineering Faculty, Podgorica, Energoprojekt-hidroinzenering A.D. Belgrade), showed that HPP “Piva” dam and its reservoir are situated in the seismically active area of the inner Dinarides that is generally characterized by moderate seismic hazard. According to the performed analyses of the seismic hazard elements, as it is usually done – for the so called design earthquake (with a return period of 200 years for engineering structures of the type of high dams) in the area in which HPP “Piva” dam is situated, there is a probability of 70% for occurrence of an earthquake that will cause at the bedrock of the dam location a maximum horizontal ground acceleration of 9.5%g, i.e., an earthquake with intensity of VII degrees according to the MSK-64 scale (or MCS). For the case of the so called maximum earthquake (with a return period of 1000 years) at this location, under the same conditions, it is realistic to expect maximum effects of local or distant earthquakes expressed by maximum horizontal ground acceleration of 17.0% g, i.e., earthquakes with intensity of VII-VIII degrees according to MSK-64. For the return period of 475 years (EUROCODE 8), also with a probability of 70%, one should expect maximum accelerations of up to 13.0% g at the dam foundation level.

3. SEISMIC MONITORING

Seismic monitoring of "Piva" dam started in 1976, when an seismic monitoring network comprised of 6 analog SMA-1 accelerographs was established. The instruments were located in such manner that enabled obtaining strong motion records on free field (bedrock) at the bottom of the dam and other 5 were located on the predefined points on the dam at elevations 510, 562, 602, 642and on the dam’s crest (Fig. 2). In addition, in Pluzine (20 km away from the dam), at the electrical company's Headquaters building and the Hotel were installed two more SMA-1 instruments (both at ground level). These 8 instruments formed sort of local strong motion network. Later on two more instruments (digital QDR) were added, placed approximately at the middle of the dams body, so that the dam was fully instrumented along the height of its central segment. Well distributed instruments on the dam body and the surrounding soil enables obtaining of high quality data on the seismic activity and structural response, which represents a good basis for the understanding of the behaviour of the dam exposed to seismic action and making appropriate decisions for the case of a possible strong earthquake to which the dam could be exposed.

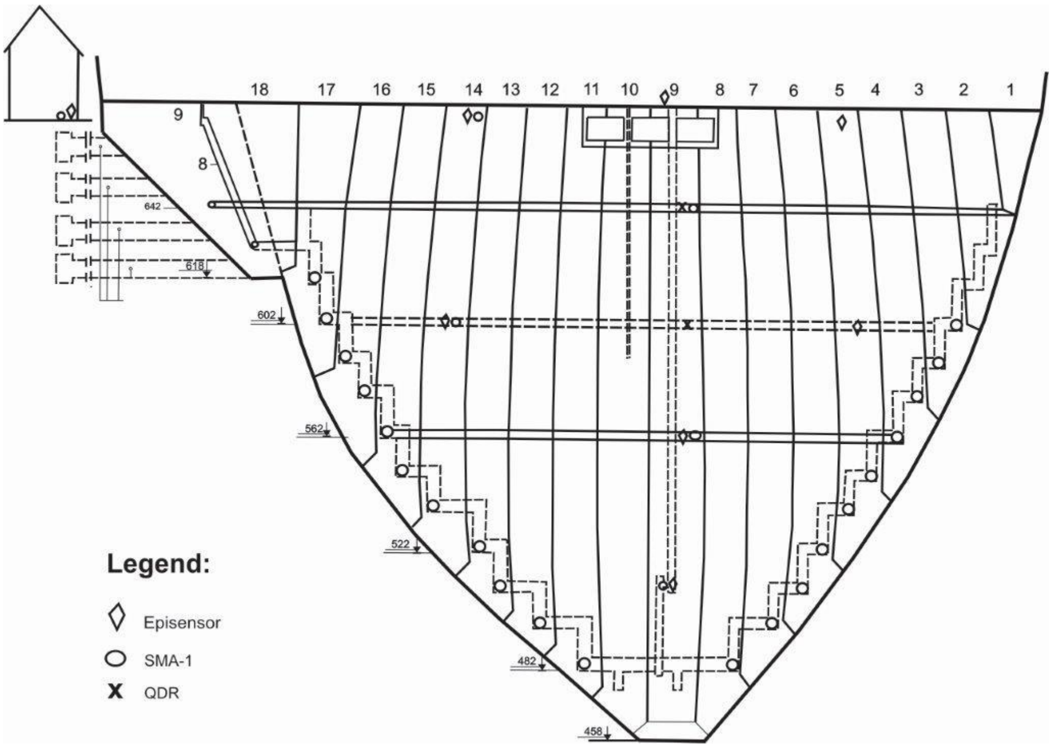

Figure 2. Location of the seismic monitoring instruments

The modern, digital system for seismic osculation enables continuous and reliable monitoring of the seismic activity and recording of occurred earthquakes. In 2013, a modern digital seismic monitoring network was installed (Fig. 3). Eight digital episensors are installed, tree on the crest, four on different locations on the dam's body and one on bedrock in the occultation building. Instruments are placed so that the full height of the central console is instrumented. Central unit is providing recording in real time.

There were no significant earthquakes in the period since the new seismic monitoring network is in operation, but network recorded some machine activity (Fig. 4). In the period of next two years, beside the new network, the analog SMA-1 accelerographs will be left in place, as a backup.

Figure 3. Installation of the seismic monitoring network

Recorded at bedrock (Osculation building) *Recorded on dam's crest (block 14)*

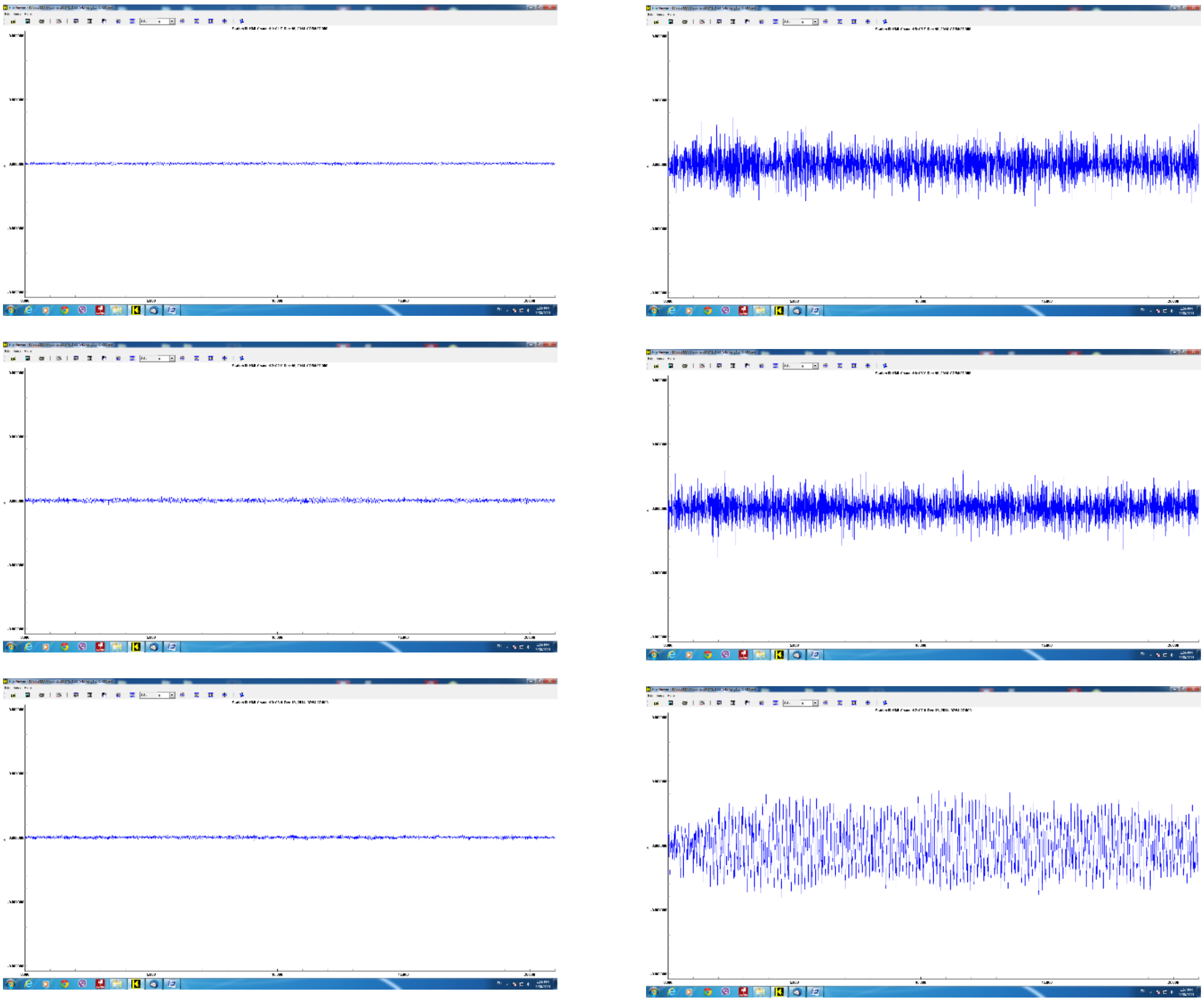

Figure 4. Three-component registrations obtained on two different instruments (bedrock and dam's crest)

4. CONCLUSIONS

The new seismic monitoring network is expected to enable obtaining quality digital records of the response of the dam in different points along it height and width thus creating a data base for reliable analysis of the behaviour of this type of structure exposed to strong seismic excitation. The data on basic seismic parameters (time histories of ground acceleration, velocity and displacement will help in decision making process in the immediate aftermath of eventual extreme event regarding the dam.


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NUMERICAL ANALYSIS OF DISPLACEMENTS IN THE POST-EXPLOITATION PERIOD OF TAILINGS DAMS WITH A COMBINED CONSTRUCTION METHOD

The similarities between the tailings dams and the embankment dams for water storage have contributed a great number of procedures and techniques in the design, construction and maintenance of the conventional dams, to be applied to tailings dams. However, the numerous reports of collapses of the tailings dams in the last three decades, all over the World, indicate that the structural, (static and dynamic), filtration, hydrological and hydraulic safety were not controlled with the same rigor and carefulness - as for the embankment dams. This fact, in part, results from the long-term construction of the tailings dams, where the building material is sand obtained by separating the waste material from the floatation process during the exploitation of the mine. This paper presents the results from the static analysis of the hydro tailings Topolnica, on the river Topolnica, in the east part of Republic of Macedonia. It is a tailings dam with a combination of downstream (in the first phase) and upstream (in the second phase) methods of construction, with a total height from the crest to the downstream toe of the dam of 141.2 m.

Keywords: tailings dams, combined construction method,

1. INTRODUCTION

Tailings dams are complex engineering structures, composed of an initial dam, sand dam, waste lagoon, drainage system, outlet pipe for discharge of clear water, and structures for protection in case of inflow (external) water [Petkovski L., Gocevski B., Mitovski S., 2014.06], [Petkovski L., Peltechki D., Mitovski S., 2014.09].

The tailings, on one hand, due to the numerous structures of which are composed, should be checked on great number of safety cases at static loading, similar as for conventional fill dams [Petkovski L., Tanchev L., Mitovski S., 2007. 06], and on other hand, due to the enormous volume of the waste lagoon, they are fill structures with highest potential hazard for the surrounding [Petkovski L., Mitovski S., 2015.06]. Due to the great importance of the tailings dams, one of the ICOLD's Technical Committees is exactly for tailings dams and deposit lakes - ICOLD Committee on Tailings dams and Waste Lagoons), that has published 1o Bulletins, [ICOLD, 1982, Bulletin 45] - [ICOLD, 2011, Bulletin 139].

Due to the long construction period, the approach for conventional dams (for creation of water reservoirs) for confirmation of proper accomplishment of the hydraulic structures – with full supervision of the construction and control of the first reservoir filling, as well and the assessment of the dam's proper behaviour with construction parameters throughout comparison with monitoring data, at most cases is not applied fully in case of tailings dams. Unfortunately, such main difference between the conventional and tailings dams is amplified in case of technical solutions with combined construction method [Petkovski L., 2015.09] and heightening [Petkovski L., Mitovski S., 2012.10] thus providing increase of the deposit space of the tailings dams. The purpose of the research is to analyse the settlements in tailings dams body upon service period of the waste lagoon to plan the dam crest heightening and to estimate limit value for the displacements, in order to compare with measured values within monitoring process, so the proper conclusion can be drawn out for the regular behavior of the dam in the future period. In the below text, the paper will be illustrated with data from the research of the displacements in the post service period of the waste lagoon, generated with various scenarios for pore pressure dissipation within the tailings, for tailings dam Topolnica of mine Buchim, Republic of Macedonia, with combined construction method.

2. BASIC PARAMETERS OF TAILINGS DAM TOPOLNICA

Tailings dam Topolnica of mine Buchim, Radovish, commissioned in 1979, is created by deposition of the flotation pulp. By the method of pulp hydro-cycling, from the sand is created the downstream sand dam, and the spillway from the hydro-cyclones (sometimes and non-cycled tailings) is released in the upstream waste lagoon. In such way in the waste lagoon is done mechanical deposition of the finest particles and chemical purification of the used reagents, present in the tailings. In the past period in tailings dam Topolnica (fig. 1) is deposited tailings volume over 130 million m^3 and water is stored in volume of approximately 9 million m^3.

Figure 1. Crest and waste lagoon of tailings dam Buchim, March, 2016

The tailings dam is characterized with stage construction and combined construction method, by downstream progressing in first stage and upstream progressing at heightening from second stage, in two phases. The construction started with the initial dam, fig. 2, with foundation elevation 518.5 masl and crest elevation 558.5 masl. The construction of the sand dam in first stage, up to elevation 610 masl (I stage), fig. 3, was constructed in inclined layers, by progressing in downstream direction from the initial dam. Afterwards, the construction of the sand dam to elevation 630 masl (II stage, phase 1), due to the vicinity of village Topolnica to the downstream toe of the dam, was constructed by filling in upstream direction, fig. 4. At terminal stage is adopted sand dam crest at 654.0 masl (II stage, phase 2), by progression in upstream direction, fig. 5.

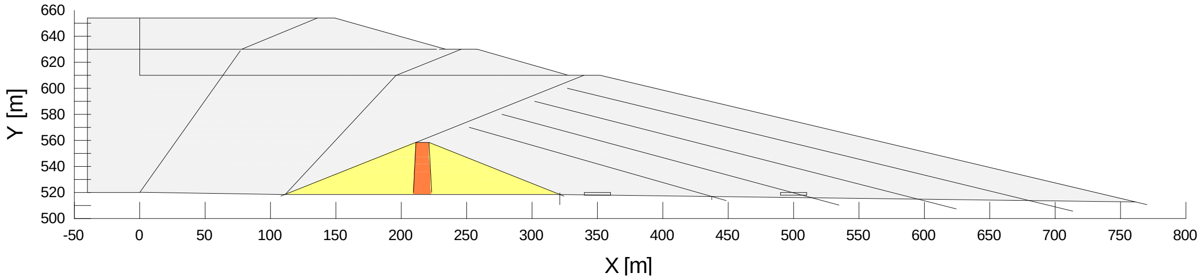

Figure 2. Construction of initial dam to elevation 558.5 masl

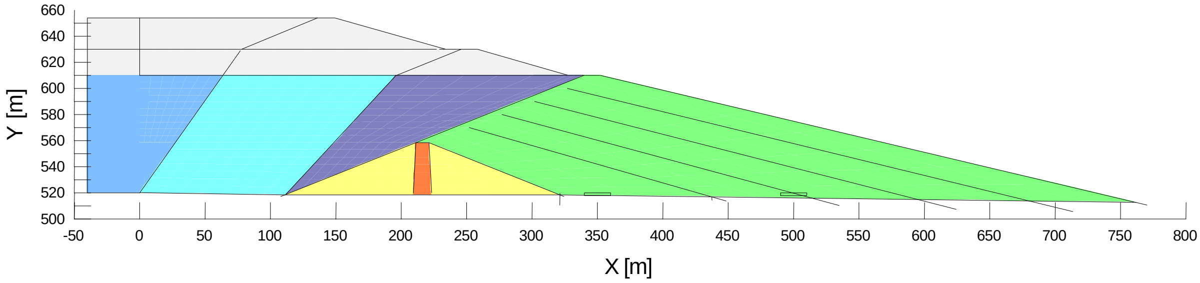

Figure 3. Tailings dam construction to elevation 610 masl (I stage)

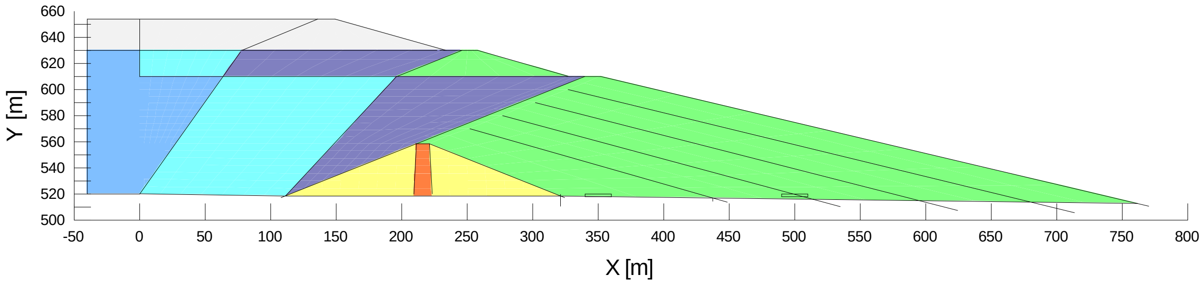

Figure 4. Construction of tailings dam to elevation 630 masl, (II stage, phase 1)

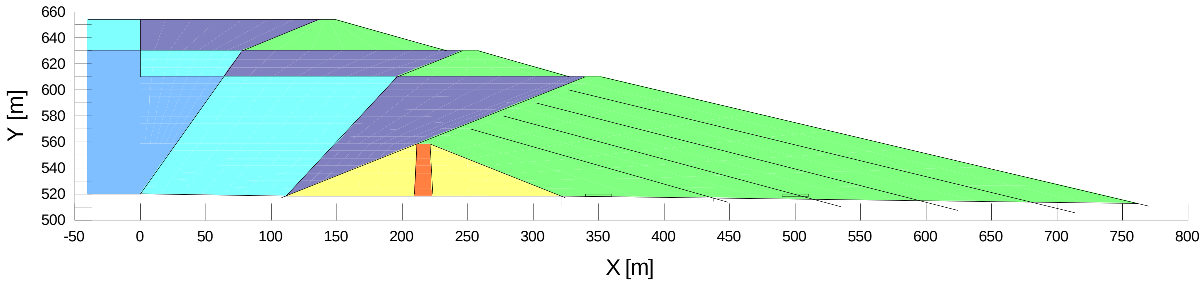

Figure 5. Construction of tailings dam to elevation 654 masl, (II stage, phase 2)

The overall dimensions of the representative cross section for structural (static and dynamic) analysis are: length 801.4 m and height 141.2 m. The tailings dam Topolnica, with height of dam no. 2-2 above the foundation of initial dam of H_0 =654.0-518.5=135.5 m, is one of the highest tailings dams in Europe. The final height of the tailings dam no. 2-2, from crest to downstream toe, is H_2 = 654.0 - 512.8 = 141.2 m, by what Topolnica tailings dam is highest dam in R. Macedonia. Namely, the highest conventional dam (for water reservoir), dam Kozjak, according to as built data from 2001, has height from dam crest to core foundation of 472.2 – 341.8 = 130.4 m. The enormous dimensions of the sand dam, heterogeneous composition of the geo-medium and combined construction method, downstream in stage I and upstream in stage II, obviously shows that dam Topolnica is one of the most complex and most important fill structures in R. Macedonia.

Regarding the geomechanical parameters of the local materials, from which the tailings dam is constructed certain approximations are foreseen, thus contributing to simplification of the numerical experiment, and in same time does not decrease the safety analysis. The simplification of the material parameters is provided by the following approximations: (1) the waste lagoon, possessing highly non-specified and heterogeneous composition, by finer grain size fractions in the upstream and coarser grain size particles in the downstream part of the sand dam, is represented with 3 different materials; (2) the filter transition zones in the initial dam are neglected, for which is estimated that they have small dimensions, compared to the geostatic medium from interest in the analysis. In such a way is prepared idealized cross section for structural analysis, and the heterogeneous composition of the tailings dam is modelled with number of segments by 6 different materials, (fig. 6). The discretization of the tailings dam for static analysis (fig. 7) is done in order to model the stage construction, by development and dissipation of the consolidation pore pressure.

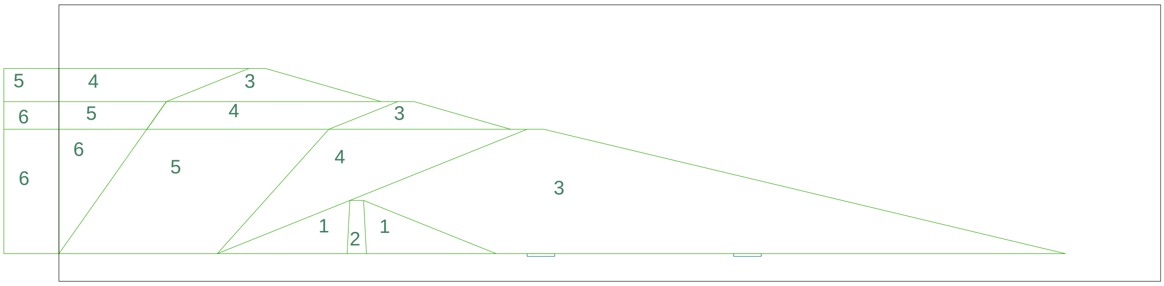

Figure 6. Segments by 6 different materials. 1 – gravel in initial dam body, 2 – clay in initial dam core, 3 – sand in tailings dam, 4 – sand silt in beach, 5 – sand silt between the beach and lagoon and 6 – silt in waste lagoon

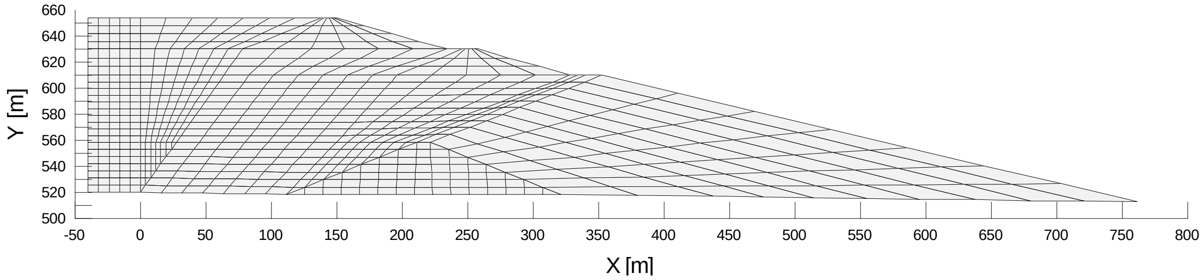

Figure 7. Discretization of the mediums for static analysis by FEM (N=725, E=687)

3. MODELING OF TAILINGS DAM CONSTRUCTION STAGE

The initial state of stresses and development of pore pressure for analysis of the displacements in the post-service stage of tailings dam Topolnica is state of dam construction. The realistic construction dynamics of the sand dam is approximated in 48 time steps for the tailings dam, with different duration. Such 48 time stages are divided on 24 (for initial and sand dams) and 24 for the waste lagoon, whereas the realistic time for tailings dam construction is simulated in days (fig. 8) for the needs of the mathematical model. Realistic progress of the tailings dam is simulated by the model, apropos filling of the waste lagoon is by appropriate time delay upon sand dam construction.

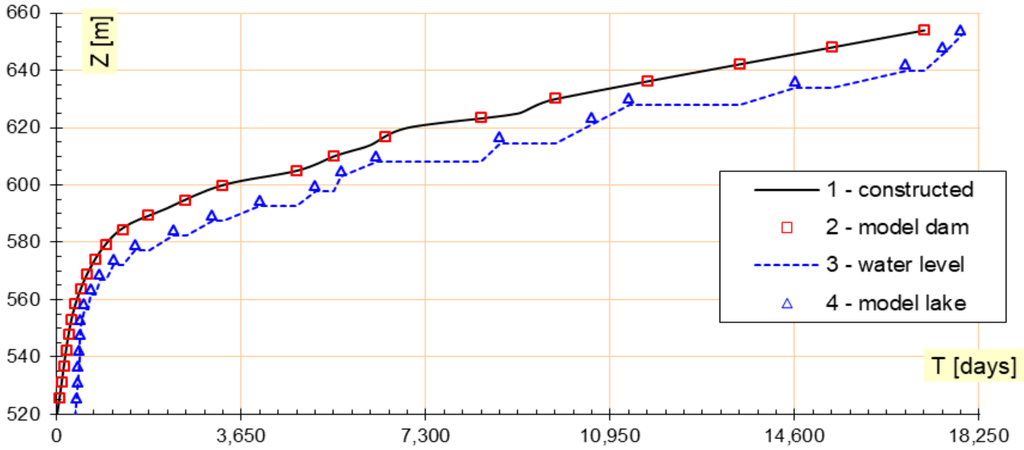

Figure 8. Construction chart, constructed (1 - dam, 3 - lagoon) and modeled (2 - dam, 4 – lagoon) state, in days

The upstream water saturation of the tailings due to the existing water inflow from river Topolnica in the tailings dam during progressing of the waste lagoon is adopted to be 2.0 m lower than the deposited tailings. Such upstream non-steady hydraulic boundary condition is necessary for the analysis of the effective stresses for the alternative with upstream water saturation of the tailings during construction, apropos for the service period of the structure. In the present consolidation analysis, by analysing the effective stresses in drained conditions in realistic time domain [Geo-Slope SIGMA/W, 2012] is adopted water filling function in the waste lagoon, as variable upstream boundary condition for analysis of the non-steady seepage [Geo-Slope SEEP/W, 2012]. In such complex and coupled analysis (by parallel mechanical and hydraulic response), in the same time are simulated: (a) stage construction, (b) development and dissipation of consolidation pore pressure, (c) change of the upstream hydrostatic pressure and (d) heterogeneous medium by irregular geometry. In the analysis, that simulates the tailings behaviour most realistically, the material parameters have influence and also the time component, apropos the realistic construction dynamics. In continuation, for construction of dam no. 2-2 (for crest elevation 654.0 masl) are displayed: iso-lines of vertical displacements (fig. 9), horizontal displacements (fig. 10) and maximal normal total stresses (fig. 11). Throughout comparison of the partial displacements in zone of the inclinometer J4 (installed from elevation 625 masl to 570 masl), for initial state for construction of dam no. 2-2 up to elevation 630 masl) and final state for filling of the tailings till elevation 654 masl, it has been verified the regular behaviour of the embankment structure and also are verified the material parameters for the local materials adopted for the static analysis.

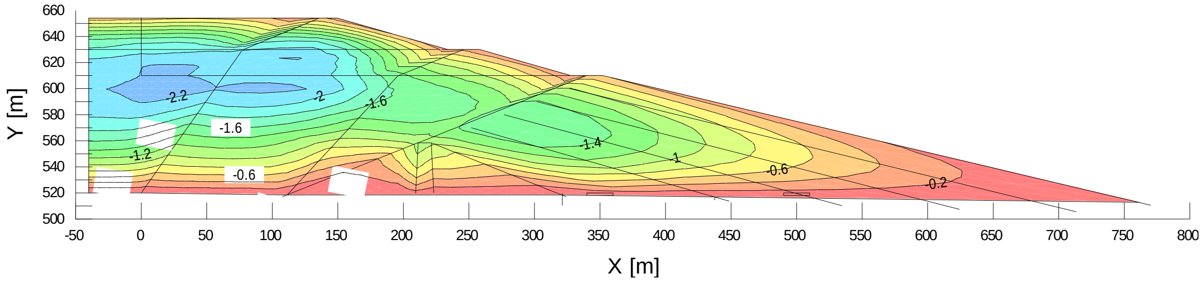

Figure 9. Vertical displacement in tailings II stage - phase 2 (increment 48), by linear-elastic model, by analysis of the effective stresses in drained conditions with upstream water saturation, K_{II2} = 654.0 masl, Y=(-2.347÷ 0.000)m

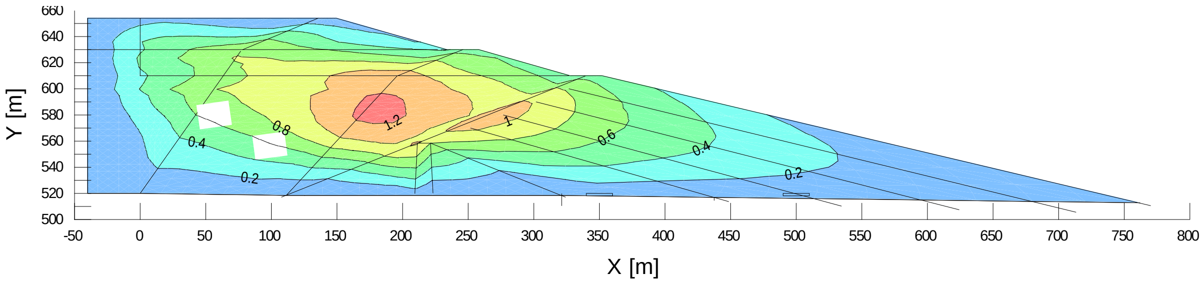

Figure 10. Horizontal displacement in tailings II stage - phase 2 (increment 48), by linear-elastic model, by analysis of the effective stresses in drained conditions with upstream water saturation, K_{II2} = 654.0 masl, X = (-0.000 ÷ 1.249) m

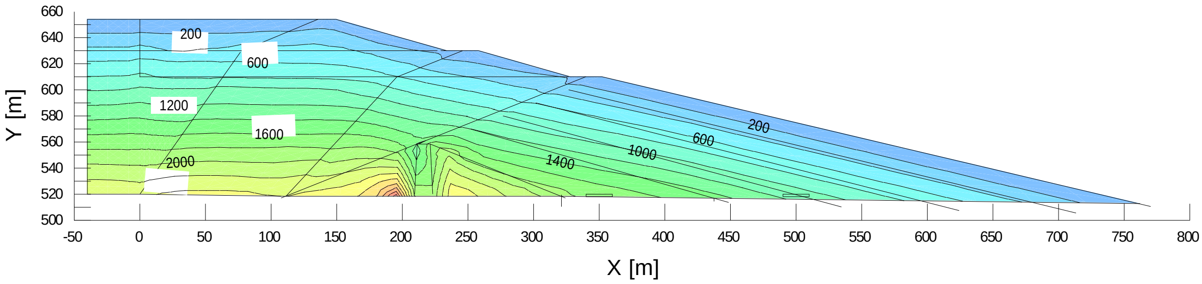

Figure 11. Maximal normal total stresses in tailings II stage - phase 2 (increment 48), by linear-elastic model, by analysis of the effective stresses in drained conditions with upstream water saturation, K_{II2} = 654.0 masl, σ_1 = (25.19 ÷ 3503.14) kN/m^2

4. MODELING OF POST- EXPLOITATION STATE

Within the research of the behavior of the tailings dam, of special interest for the finding of the structure safety is the assessment of the maximal values of the horizontal and vertical displacement of the sand dam in the post-service stage. Such data are important, for comparison with monitoring data (the monitoring process will take place during service period) as well for making valid conclusions for the proper behavior of the tailings dam and as for adoption of required heightening above crest of dam no. 2-2 at elevation 654.0 masl, that will include the residual settlements after dam construction. In the research of the tailings dam response by elastic-plastic model at transformation of the consolidation pore pressure, generated in the final time step in drained conditions, during construction (fig. 12), in the future post-service period are analyzed three scenarios: (1) full dissipation of the consolidation pressure, (2) transformation of the consolidation pressure in steady pore pressure for seepage with constant level of upper water at 630.0 masl (fig. 13) and (3) transformation of the consolidation pressure in steady pore pressure for seepage with constant level of upper water at 652.0 masl (fig. 14).

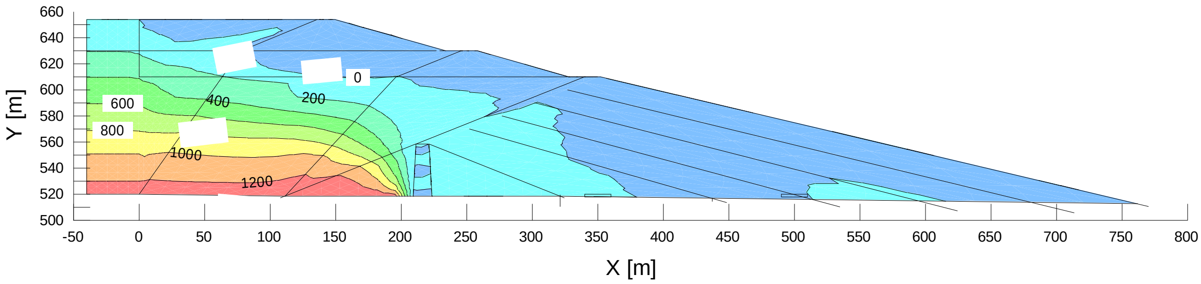

Figure 12. Pore pressure in tailings dam II stage - phase 2 (increment 48), by linear-elastic model, analysis by effective stresses in drained conditions with upstream water saturation according to the realistic construction dynamics up to crest elevation K_{II2} = 654.0 masl, P_w = (-157.03 ÷ 1309.24) kN/m^2

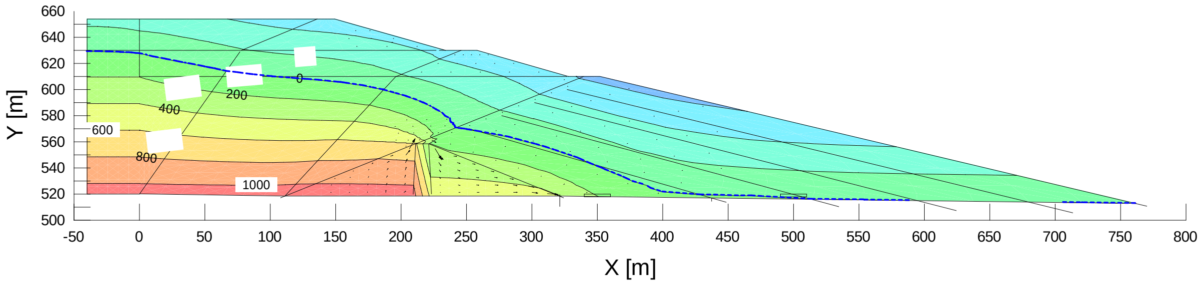

Figure 13. Pore pressure in tailings dam II stage - phase 2 for steady seepage, with upstream water saturation at 630.0 masl, P_w < 1093.48 kN/m^2

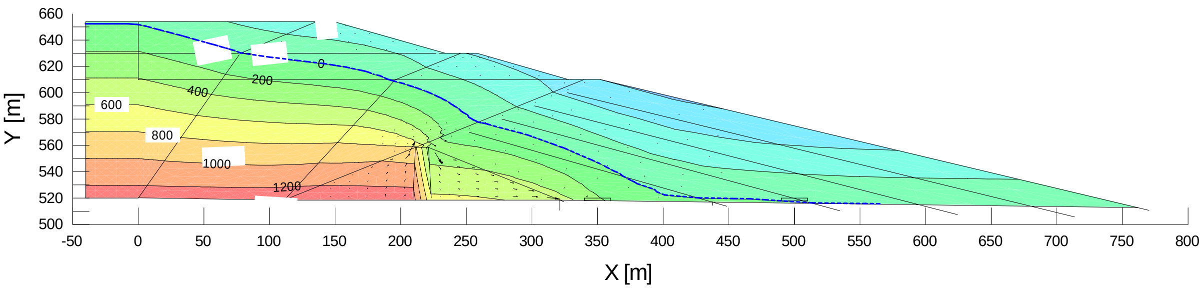

Figure 14. Pore pressure in tailings dam II stage - phase 2 for steady seepage, with upstream water saturation at 652.0 masl, P_w < 1309.23 kN/m^2

RESULTS FROM ANALYSIS OF THE POST-SERVICE STATE

The first scenario, by full dissipation of the consolidation pressure (fig. 15 and 16) undoubtedly is fictive state, because in the post-service period, the existing inflow of water from river Topolnica in the waste lagoon will create steady seepage regime through the tailings dam, and most probably will not allow full dissipation of the pore pressure. Therefore, the value of such analysis should be treated as maximal theoretical values. If during displacements monitoring for the sand dam are recorded values higher that the specified in the analysis, that is certain clue that there are some anomalies in the regular behavior of the tailings dam.

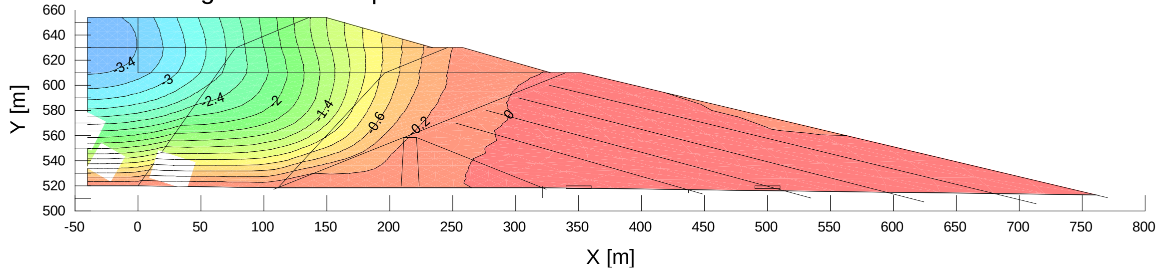

Figure 15. Isolines of vertical displacements ΔY = (-3.583) - (+0.023) m, by full dissipation of the consolidation pressure

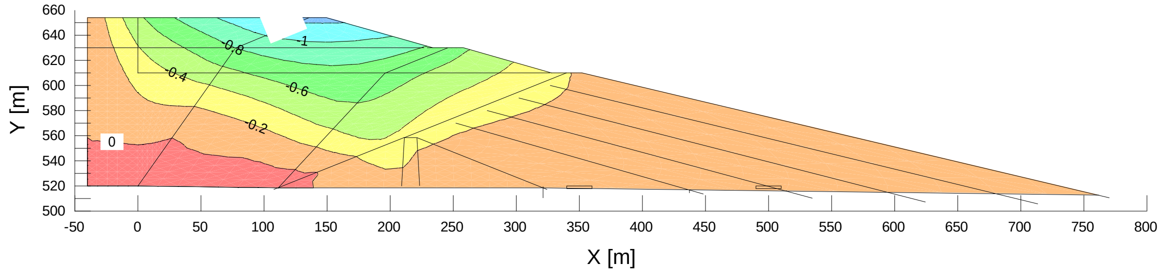

Figure 16. Isolines of horizontal displacements ΔX = (-1.269) - (+0.057) m, by full dissipation of the consolidation pressure

For the second scenario by transformation of the consolidation pressure in steady regime at elevation 630.0 masl (fig. 17 and 18), for which we are on opinion that is it most probable for the future period, in the crest of dam no. 2-2 at elevation 654.0 masl the expected settlements are approximately 60.0 cm.

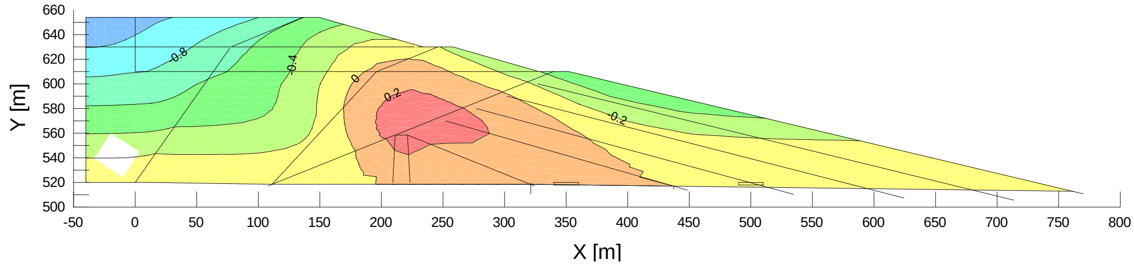

Figure 17. Isolines of vertical displacements ΔY = (-1.176) - (+0.281) m, by transformation of the consolidation pressure, in steady seepage pressure at upper water elevation 630.0 masl

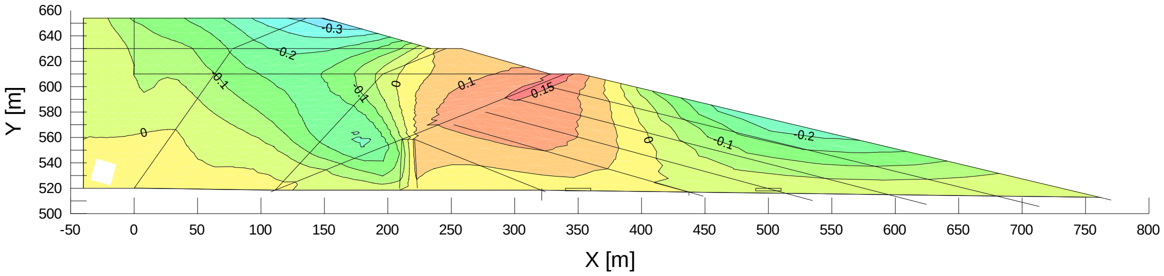

Figure 18. Isolines of horizontal displacements ΔX = (-0.363) - (+0.177) m, by transformation of the consolidation pressure, in steady seepage pressure at upper water elevation 630.0 masl

For the third scenario by transformation of the consolidation pressure in steady pressure at elevation 652.0 masl (fig. 19 and 20), for what we think that is practically impossible and it serves as theoretical knowledge as possible boundary state, in crest of dam no. 2-2 at elevation 654.0 masl, due to the material swelling from the upstream water saturation, there is occurrence of rising of approximately 20.0 cm. But, such value obtained by elastic response of the fill structure according to the numerical experiment, in the realistic structure it is not possible due to the deformations caused by creep of the material, that cannot be modeled yet in satisfactory way.

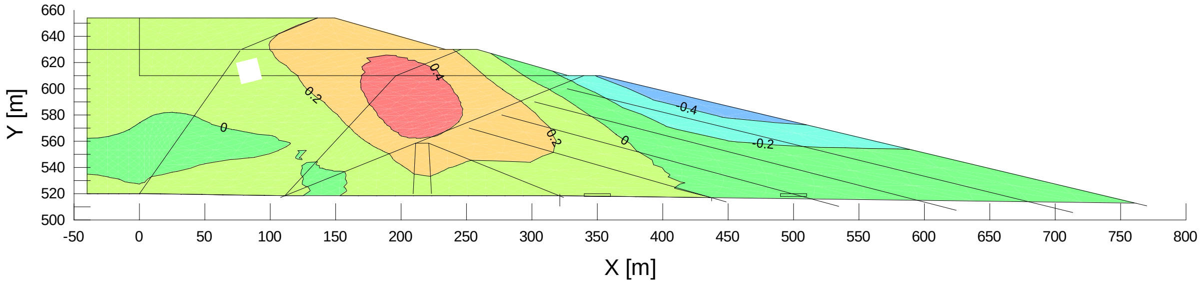

Figure 19. Isolines of vertical displacements ΔY = (-0.553) - (+0.497) m, by transformation of the consolidation pressure, in steady seepage pressure for upper water at 652.0 masl

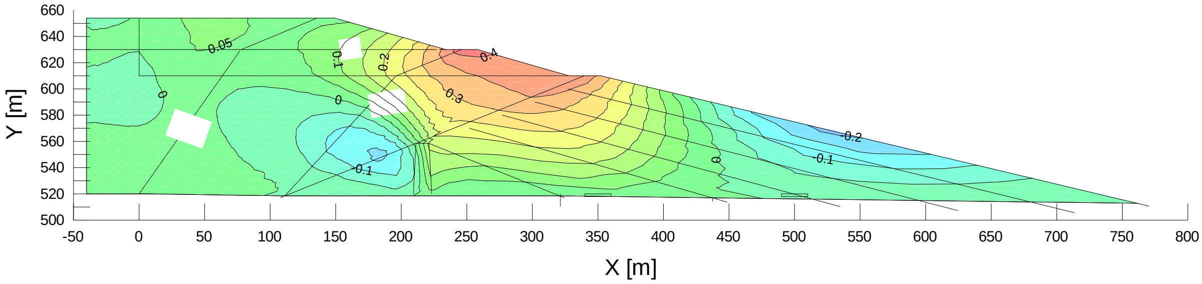

Figure 20. Isolines of horizontal displacements ΔX = (-0.218) - (+0.431) m, by transformation of the consolidation pressure, in steady seepage pressure for upper water at 652.0 masl

CONCLUSION

For assessment of the maximal value of the horizontal and vertical displacement in the sand dam post-service stage, three scenarios are analyzed for transformation of the consolidation pore pressure generated in the final time step in drained conditions during construction.

The first scenario is by full dissipation of the consolidation pressure apropos full dry out of the waste lagoon that is practically impossible due to the natural inflow of river Topolnica. This is the most pessimistic scenario and the values for crest settlements of dam no. 2-2 at elevation 654.0 masl, obtained approximately 140-160 cm, and should be treated as maximal theoretical values. If during monitoring of the displacements of the sand dam are registered higher values then the specified that is sure clue that there are some anomalies in the regular behavior of the dam apropos occurrence of not-permitted seepage, followed by process of mechanical suffusion that can cause collapse of the fill structure.

The second scenario is by transformation of the consolidation pressure in steady pore pressure for seepage at constant level of upper water at 630 masl. For such scenario, for which we think that is most probable in the future period, the crest settlements in the sand dam can be expected approximately of 50-60 cm. The third scenario is by transformation of the consolidation pressure in steady pore pressure for seepage at constant level of upper water at 652 masl. For such scenario, due to the material swelling from the upstream water saturation, by the mathematical model (due to the elastic response of the materials) a rising of the material would be occurred for approximately 20 cm.

Having in consideration the consolidation settlements in the post-service period, according to scenario no. 2 (for which we think that most probably will occur in future), as well and material creep deformations, that can not be modelled yet on satisfactory way, we recommend heightening in crest of dam no. 2-2 at elevation 654.0 masl to be 1.0 m. Such value of 1.0 m will be for the mediate part with greatest height of the dam, and for remaining parts from the mediate zone towards the valley banks, such heightening is linearly decreased to value of 0.0 m.

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REFERENCES

[1] Geo-Slope SIGMA/W, 2012. "User's Guide for finite element stress/deformation analysis", GEO-SLOPE International Ltd., Calgary, Alberta, Canada

[2] Geo-Slope SEEP/W, 2012. "User's Guide for finite element seepage analysis", GEO-SLOPE International Ltd., Calgary, Alberta, Canada

[3] ICOLD, 1982, Bulletin 45, Manual on tailings dams and dumps. ICOLD, 1989, Bulletin 74: Tailings Dam Safety Guidelines. ICOLD, 1994, Bulletin 97: Tailings Dam - Design of Drainage

[4] ICOLD, 1995, Bulletin 98: Tailings Dams and Seismicity - Review and Recommendations. ICOLD, 1995, Bulletin 101: Tailings Dams, Transport, Placement and Decantation - Review and Recommendations. ICOLD, 1996, Bulletin 103: Tailings Dams And Environment - Review and Recommendations. ICOLD, 1996, Bulletin 104: Monitoring of Tailings Dams - Review and Recommendations

[5] ICOLD, 1996, Bulletin 106: A Guide to Tailings Dams and Impoundments - Design, Construction, Use and Rehabilitation

[6] ICOLD, 2001, Bulletin 121: Tailings Dams Risk of Dangerous Occurrences - Lessons Learnt From Practical Experiences

[7] ICOLD, 2011, Bulletin 139, Improving tailings dam safety - Critical aspects of management, design, operation and closure

[8] Petkovski L., 2015.09 "Modified Conventional Dam in Phase Construction – an Alternative to Tailings Dam", UDK: 627.821, Original scientific paper, Watereconomy, ISSN 0350-0519, Vol. 47 (2015), No. 273-275, p. 41-50, Beograd, Srbia;

[9] Petkovski L., Gocevski B., Mitovski S., 2014.06 "Comparative Analysis on Most Favourable Method for Tailings Dam Construction", paper, IV Symposium of MAG, June 25-28, Struga, R.Macedonia, Proceedings 293-300;

[10] Petkovski L., Mitovski S., 2012.10 "Structural safety of tailings dams at increasing of the deposit space", Conference, Talinigs dams in Republic of Macedonia, MACOLD, 30-31.10.2012, Shtip, R.Macedonia, Proceedings, 89-100

[11] Petkovski L., Mitovski S., 2015.06 "Creating of tailings space by stage construction of rockfill dam", 25. Congress on Large Dams, ICOLD, 13–19 June 2015, Stavanger, Norway, CD Proceedings Q.98-R.4, p. 53-65

[12] Petkovski L., Peltechki D., Mitovski S., 2014.09 “Contribution to the Methodology on Choice of Most Favourable Site and Optimal Dam Type for Creation of Tailings Dam”, paper, 10th Conference on water economy and hydrotechnics, 18-19 September 2014, Struga, R. Macedonia, Proceedings, p.227-236, ISBN 978-608-65373-3-3;

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