VYSOKÉ UČENÍ TECHNICKÉ V BRNĚ

BRNO UNIVERSITY OF TECHNOLOGY

FAKULTA STAVEBNÍ

FACULTY OF CIVIL ENGINEERING

ÚSTAV VODNÍCH STAVEB

INSTITUTE OF WATER STRUCTURES

NUMERICKÉ MODELOVÁNÍ UČINKŮ DO TĚSNĚNÍ SYPANÝCH HRÁZÍ A JEJICH PODLOŽÍ

Numerical modelling of resealing effects in earth dams

DIZERTAČNÍ PRÁCE – ZKRÁCENÁ VERZE

DOCTORAL THESIS – SHORT VERSION

AUTOR PRÁCE Ing. Somia Bredy

AUTHOR

VEDOUCÍ PRÁCE Doc. Ing. Jan Jandora, CSc.

SUPERVISOR

BRNO 2020

KLÍČOVÁ SLOVA

Hráz; Vodního dílo Karolinka; Těsnění; Těsnící stěna; Trysková injektáž; Stabilita svahu; Metoda konečných prvků; 3D Analýza průsaků; Analýza posunutí; Smrštění cementu; Dynamická analýza; Citlivostní analýza.

KEYWORDS

Earth dam; Karolinka dam; Sealing; Diaphragm wall; Jet grouting; Slope stability; Finite Element Method; 3D Seepage analysis; Displacement analysis; Cement Shrinkage; Dynamic Analysis; sensitivity analysis.

MÍSTO ULOŽENÍ ORIGINÁLU DISERTAČNÍ PRÁCE

Knihovnické informační centrum
Vysoké učení technické v Brně, Fakulta stavební
Veveří 95
602 00 Brno

Telefon: 541 147 171, 541 147 175
E-mail: library@fce.vutbr.cz

ISBN:

ISSN:

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1 INTRODUCTION

1.1 General remarks

One of the main causes of the earthen dam failure is seepage problem because of running water slowly through body dam and its foundations, which directly effects on slope stability, and this will be a big problem if it causes weakening, piping or sloughing, so this problem has to be controlled, or will lead to rapid failure of dam. Sealing is used in the recent years as an integral process and an extremely effective treatment technology to control seepage in earthen dam, fill voids, strengthening and mitigate the flow of groundwater. Methods of sealing in dams have been developed in order to determine the pump pressure, mixture properties and stop time for accurate sealing. It is very important to achieve the required sealing while avoiding ground movement or any damage in structure due to applied pressure.

1.2 Aim of work

The specific objectives of this study are as follows:

- Studying sealing technique effect with respect to the settlement behaviour, stress, strain, displacement, water flow, pore water pressure and stiffness coefficients in the Karolinka dam.
- Numerical modelling of diaphragm walls and jet grouting processes to determine the state of the dam body and foundation; before, during and after sealing by using finite element method FEM which is performed by Plaxis 3D program in case study Karolinka dam.
- Studying the effect of sealing equipment loads and rotational motion of drilling rod on the stability of the Karolinka dam.
- Numerical modelling of the early age autogenous shrinkage of cement.
- Investigation of the failure state for grouting system in the connection zone.
- Dynamic analysis of the drilling rod.
- Comparing between computed result by Plaxis and actual result of the Karolinka dam to evaluate this research concerning its accuracy and appropriateness for reality.

2 SEALING IN THE EARTHEN DAM

This technique has been used in remediation of the dams and their foundations (Bruce, 1990) as solver for many problem (seepage, settlement). When sealing is performed in a dam, caution must be taken in order not to cause damage to the core due to high pressures, and that is by the correct choice of the sealing method, pump pressures and appropriate procedure period. The main sealing methods used in the earthen dam are:

2.1 Diaphragm wall

Although it is not grouting technique, diaphragm walls are often the best choice when the dam suffers big damage. This technique is widely used in construction work. It is used in the earthen dam over the last 40 years, and suited for clay-rich environments (Bolton and Stewart,1994).

2.2 Jet grouting

Jet grouting is the most popular method for ground improvement. This technique is widely used over the world. Its application has been grown to a large variety of purposes as reducing structure displacements, increasing the bearing capacity and supporting open underground excavation. It is cutting and mixing the soil with grout material under high speed to form cylindrical columns (Fang et al, 1994). Jet grouting technique is used in earthen dam for reducing seepage through its body and foundation, without disturbing the nearby existing structures.

3 MATHEMATICAL MODELLING OF SEALING IN KAROLINKA DAM

3.1 Information about dam

The Karolinka earth fill dam was constructed between 1977 and 1984, on the Stanovnice river above the town of Karolinka in the region of Vsetínsko, to supply the cities of Vsetínsko and Vlársko, with pure and wholesome water, protect from floods, and generate hydroelectric energy. The first filling of reservoir of Karolinka dam was in year of 1986. Karolinka dam is earth-fill dam consists of vertical clay gravelly core surrounded on both sides by filters of gravel extracted from the valley of the Stanovnice water stream. The face zones are formed by gravel sand from the Novy Hrozenkov and the upstream face is reinforced with macadam filled with bitumen. (Fig 3. 1) (Pařílková et al, 2016).

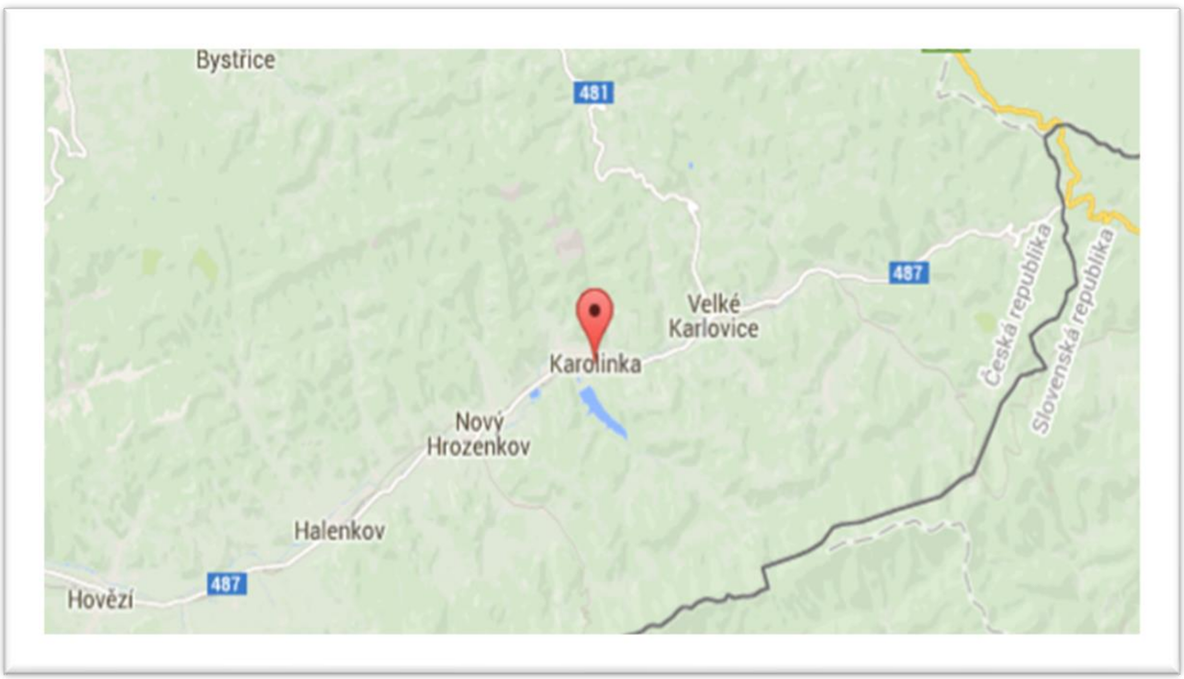


Fig. 3.1 Location of the Karolinka dam

3.2 Parameters of soil

Due to sensitivity analysis, some of parameters are assumed according to the specifications of the materials in dam.

Parameters	Core	Zone 2b	Zone 2a	Zone 3	Subsoil	Diaphragm Wall Jet pile	Mixture	Curtain	Drain	Bentonite
Hydrauli conductivity [*m/day*]	0.086	0.864	0.864	4.320	4.320	$0.864.10^{-3}$	$0.864.10^{-4}$	/	86.4	$0.864.10^{-5}$
Unsaturated unit weight [kN/m^3]	19	19	19	19	19	12.5	12.5	25	20	10.5
Saturated unit weight [kN/m^3]	21	21	21	21	21	12.5	12.5	25	21	10.5
Young modulus [kN/m^2]	20.10^3	70.10^3	70.10^3	70.10^3	70.10^3	25.10^3	500	40.10^6	100.10^3	400
Poisson´s ratio [-]	0.3	0.2	0.2	0.2	0.2	0.25	0.4	0.1	0.15	0.4
Cohesion [kN/m^2]	21	1	1	1	1	200	18	/	1	16
Friction angle [°]	/	33	33	33	33	/	/	/	37	/
Interface reduction [-]	0.7	1	1	1	1	1	1	1	1	1

Table.4.2 Material properties

3.4 Reliability analysis of Karolinka dam

3.4.1 Analysis of seepage problem

The calculation of the seepage has been simplified with numerical applications like FEM. The seepage analysis can be divided into:

A) Steady state flow analysis

The boundary conditions inside and outside the ground don't change with time. The storage function drops out and time dependent term disappears and only the coefficient of permeability is required.

B) Transient seepage analysis

The transient state condition is a variable of time and degree of saturation of the soil, different inflow and outflow with time.

3.4.2 Analysis of stability problém

A) Limit equilibrium (conventional slip circle analysis)

LM methods sum forces and moments related to an assumed slip surface passed through a soil mass. It assumes a slip surface and the soils along this surface providing shear resistance. Depending on the Mohr-Coulomb (MC) equation at the failure, the shear stress τ along the failure surface reaches the shear strength (Nash,1987).

B) Finite element method (shear strength reduction)

In FEM, failure occurs naturally through the zones where the applied shear stress exceeds the shear strength, thus no assumption about the shape or

3.4.3 Analysis of cement autogenous shrinkage problem

The shrinkage causes cracking in the element because of strains and stresses which decrease an element's ability to ban the flow of water and effect on its strength (Lura, 2003).
. In this study, the behaviour of the dam , foundation and the effects of the reconstructions have been analysed using FEM which is based on package Plaxis 3D.

3.4.4 Assumptions of material

- Homogeneous: The properties are not function of position.
- Continuum: There are no holes or voids.
- Isotropic and hydraulic conductivity are considered for each material.
- Elastic-Perfectly Plastic behaviour for the dam body and subsoil (Dawson et al, 1999).

- The strains are small.
- Mixture grouting is incompressible.
- Flow in the soil is ideal.

3.4.5 Constitutive model

The constitutive model used in this study is linear-elastic perfectly plastic with MC failure criterion. All expressions, formulas and input parameters of material and their models are described according to behaviour (Brinkgreve et al., 2017). MC failure criterion can be written as the equation for the line that represents the failure envelope (Labuz, Zang, 2012)

$$\tau = \sigma' \; tan\,\varphi' + \acute{c} \tag{3.1}$$

Where τ is shear stress, σ' is effective normal stress, φ' is effective angle of internal friction and $\acute{c}$ is effective cohesion. As a result, the failure criterion can be expressed in terms of the relationship between the principal stresses (Trigonometric Functions):

$$\sigma'_1 = \sigma'_3 \tan^2\left(\frac{\varphi}{2} + 45\right) + 2\acute{c}\tan\left(\frac{\varphi}{2} + 45\right) \tag{3.2}$$

Where σ'_1, σ'_3 are major and minor effective principal stress respectively. MC model is a reliable model and its parameters are well known and can be obtained from different soil tests.

3.4.6 Initial conditions

The initial conditions in general comprise the initial groundwater conditions, the initial geometry configuration and the initial effective stress state.

3.4.7 Boundary conditions

Boundary conditions are required at the boundaries of solution domain to define the limits and conditions in the cross-section that is being analysed. Setting up the boundary conditions in the model is a major step because the result is dependent on the chosen boundary conditions in the model.

3.5 Numerical solution in Plaxis 3D

Creating the model in the program Plaxis can be summarized in four phases (Fig. 3.2).

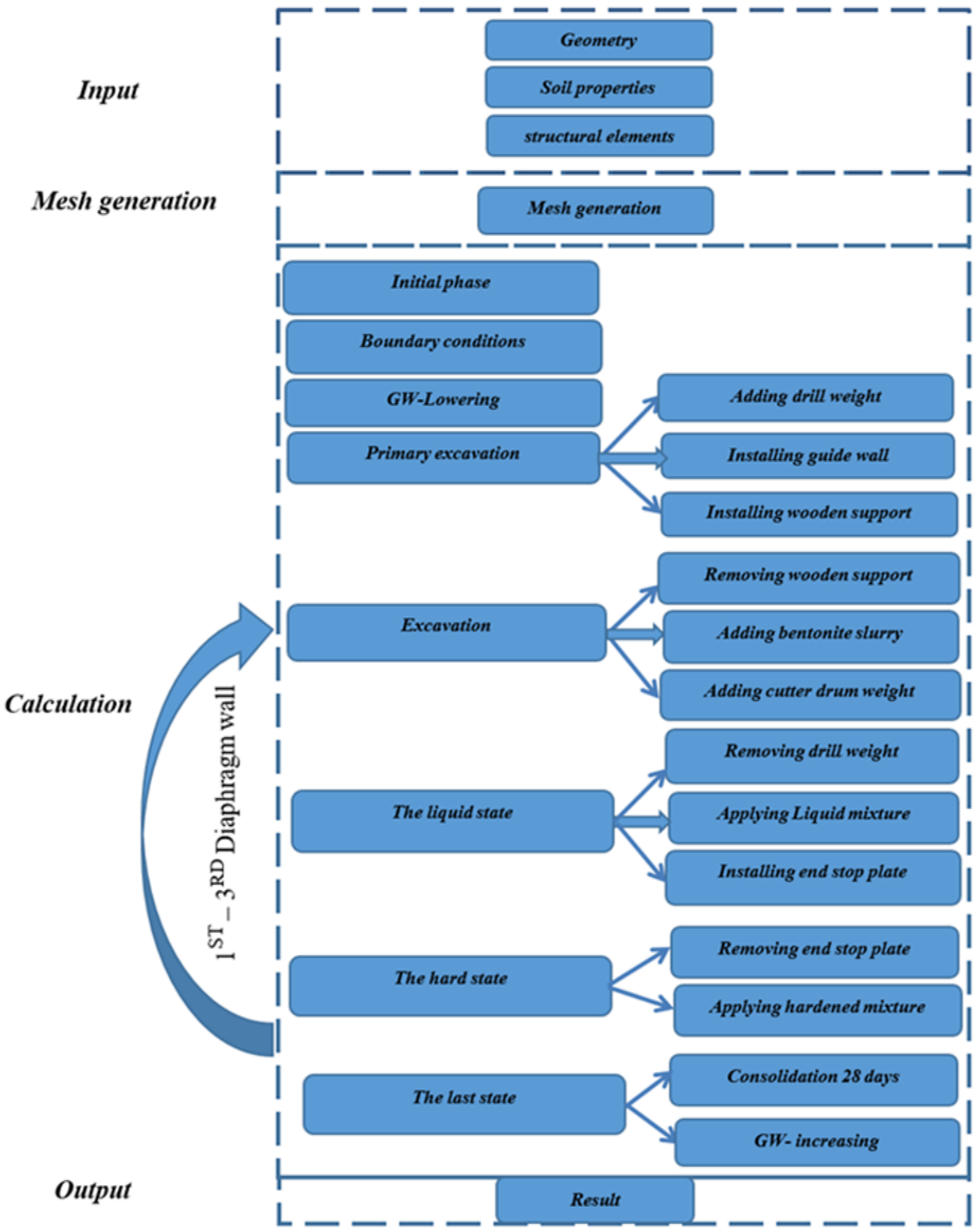


Fig. 3.2 The calculation steps in Plaxis

4 THE PRACTICAL PART

4.1 Numerical modelling of diaphragm wall

- The total length of the diaphragm walls is 301.75 m with total area of 4777m^2
- The depth of the diaphragm walls ranges from 10.50 m to 19.30 m.
- The width of the diaphragm wall is 0.60 m.
- The length of the diaphragm wall is 3.60 m.

Figure (4.1) shows the cross section of the Karolinka dam.

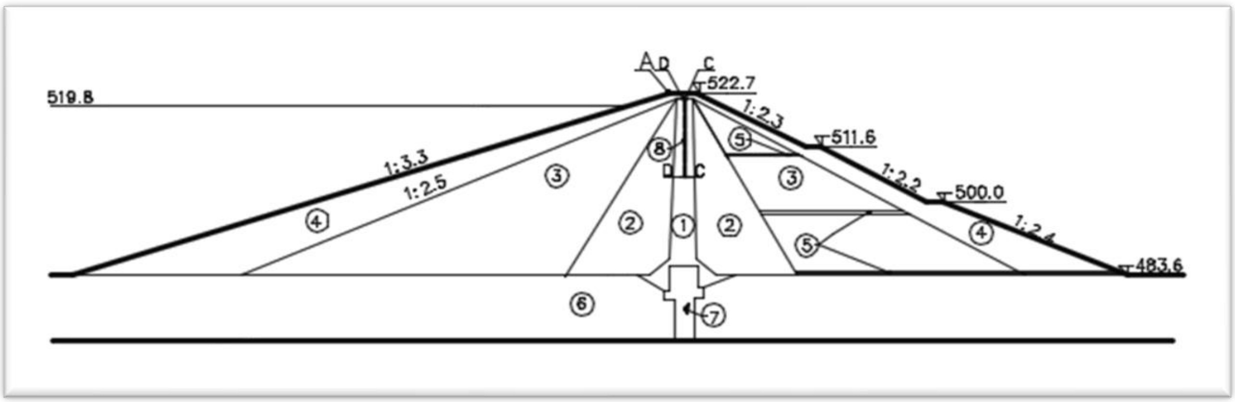


Fig. 4.1 Cross section A-A of the Karolinka dam

Legend

1. Core Clay gravelly, 2. Zone 2B Gravel with fine –grained soil, 3. Zone 2A Gravel with loam, 4. Zone 3 Gravel with fine-grained soil, 5. Gravel Drain, 6. Gravel with loam, 7. Curtain Grouting, 8. Diaphragm wall.

- ***Boundary conditions***

The Figure (4.2) shows the boundary condition of case study. The prescribed displacement at borders $\Gamma_{2,\ 3,\ 4}$ assumed to be zero :

$$U\big|\Gamma_{2,\ 3,\ 4} = 0 \tag{4.1}$$

The value of water head at borders $\Gamma_{1,5,6,7}$ assumed to be:

$$h|\Gamma_1 = H_1(t) \tag{4.2}$$

$$h\big|\Gamma_{5,\ 6} = H_2(t) \tag{4.3}$$

$$h|\Gamma_7 = Z(x, y, t) \tag{4.4}$$

Where $H_1(t), H_2(t)$ are known piezometric heads in borders Γ_1 , $\Gamma_{5,\ 6}$ respectively and $Z(x, y, t)$ is the free surface water in studied boundary.

The Neumann boundary condition for flow:

$$\left(k_{ij}\ \frac{\partial h}{\partial x_i}\right) n_i \big| \Gamma_{2,\ 4} = q_n \tag{4.5}$$

$$\left(k_{ij}\ \frac{\partial h}{\partial x_i}\right) n_i | \Gamma_3 = 0 \tag{4.6}$$

$$\left(k_{ij}\ \frac{\partial h}{\partial x_i}\right) n_i | \Gamma_7 = 0 \tag{4.7}$$

Where n_i is normal vector in directions x, y, z , q_n is specified seepage in the studied boundary, and h is hydraulic head.

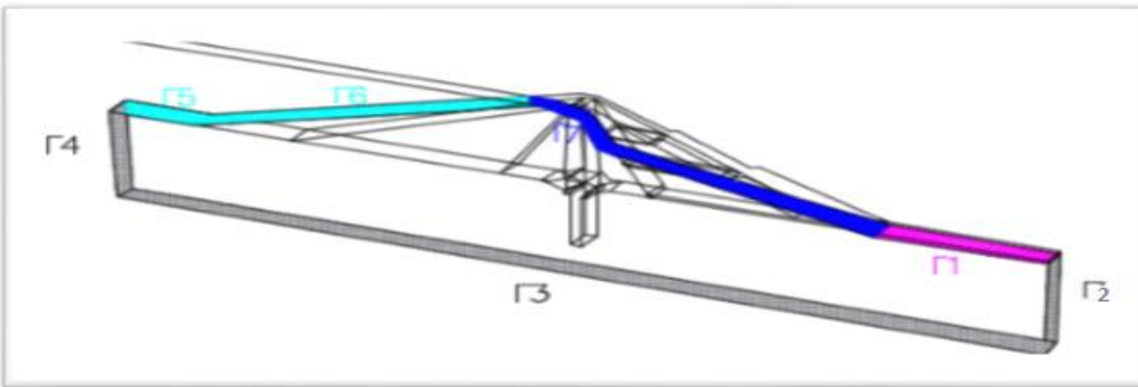


Fig. 4.2 Boundary conditions of the case study

- ***Initial conditions***
  - ***Initial displacements***

The initial value of the displacements equals zero.

- ***Initial ground water surface***

$$h_{p,0} = H_0 \tag{4.8}$$

Where $h_{p,0}$is initial piezometric head, and H_0 is specified piezometric head.

- ***Initial Stresses***

Plaxis allows calculation of the initial stress state to be carried out automatically using the coefficient of earth pressure K'_0

$$\sigma'_h = \sigma'_v . K'_0 \tag{4.9}$$

Where σ'_V is the vertical effective stress, σ'_h is the horizontal effective stress, and K'_0 is the coefficient for lateral earth pressure (Brinkgreve et al., 2017).

4. 1.1 Numerical solution

The numerical technique used in this study is the FEM which is performed by the program Plaxis.

- ***Mesh generation and boundary conditions***

In this modelling, 10-node tetrahedral elements for soil elements were used Fig (4.3). The well-refined mesh is generated with extra refinement to specific clusters.

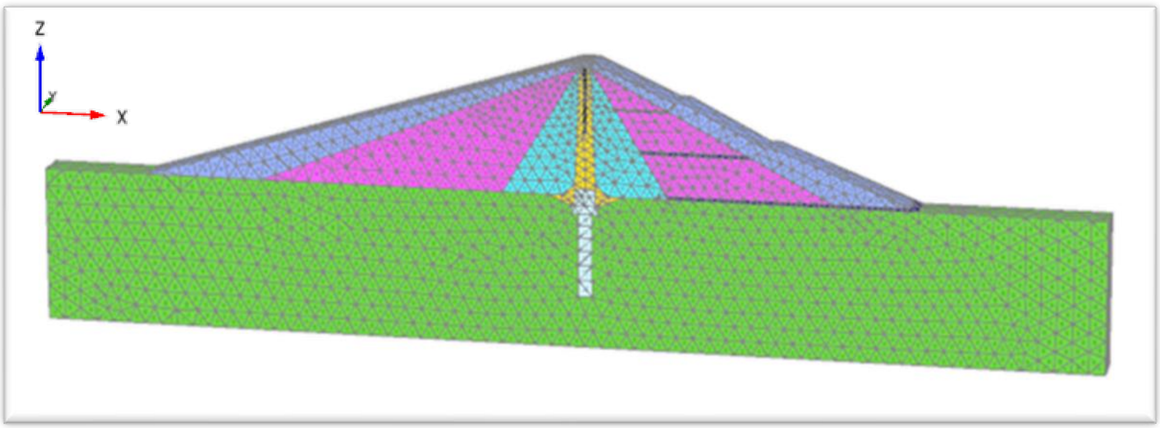


Fig. 4.3 Generated mesh

- ***Wall diaphragmm procedure***

Figure (4.4) shows the construction steps can be summed up as following:

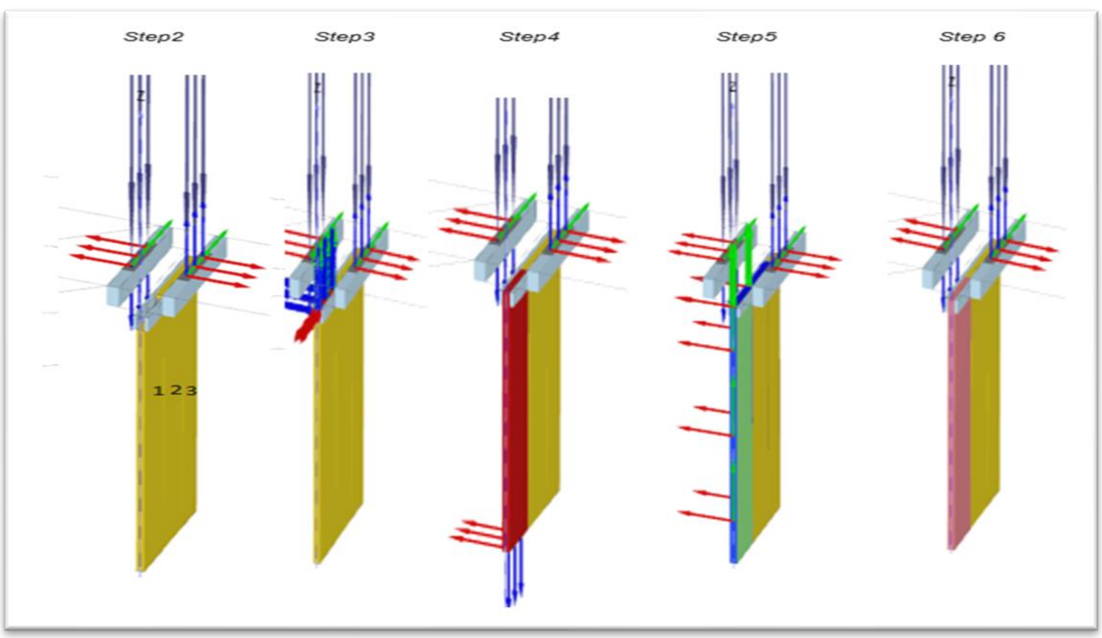


Fig. 4.4 Diaphragm wall construction sequence

4.2 Numerical modelling of piles

Additional sealing has been conducted at both end of the dam (2 × 25 m long) by using jet pile with a diameter of 1 m and overlap of 0.2 m, from a cement- bentonite mixture.

- ***Boundary conditions***

Depending on Dirichlet and Neumann boundary conditions, the boundary conditions of case study were defined at the border area. Figure (4.5) shows the boundary condition of Karolika dam (mentioned in paragraph (4.1))

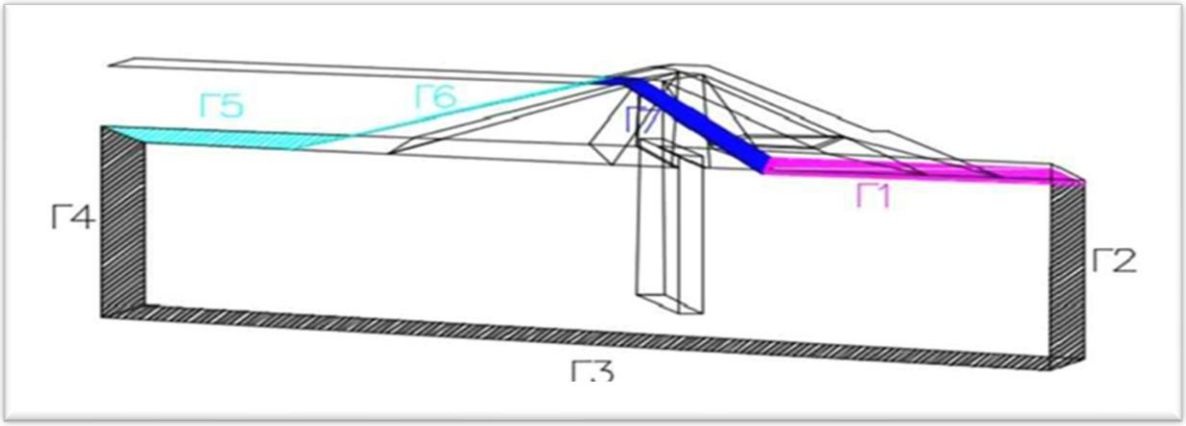


Fig. 4.5 Boundary conditions of case study

4.2.1 Numerical solution

The numerical technique used in this study is FEM that was performed by the program Plaxis. Fig (4.6) shows the cross-section B-B of the dam.

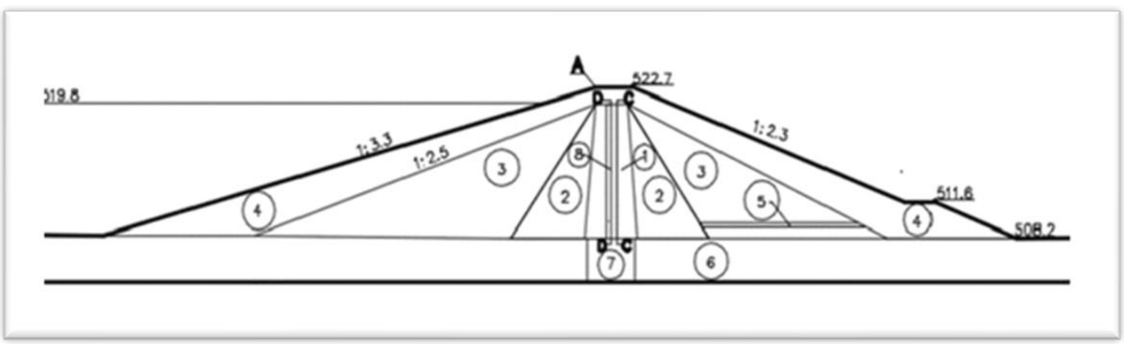


Fig. 4.6 The Cross section B-B of the Karolinka dam

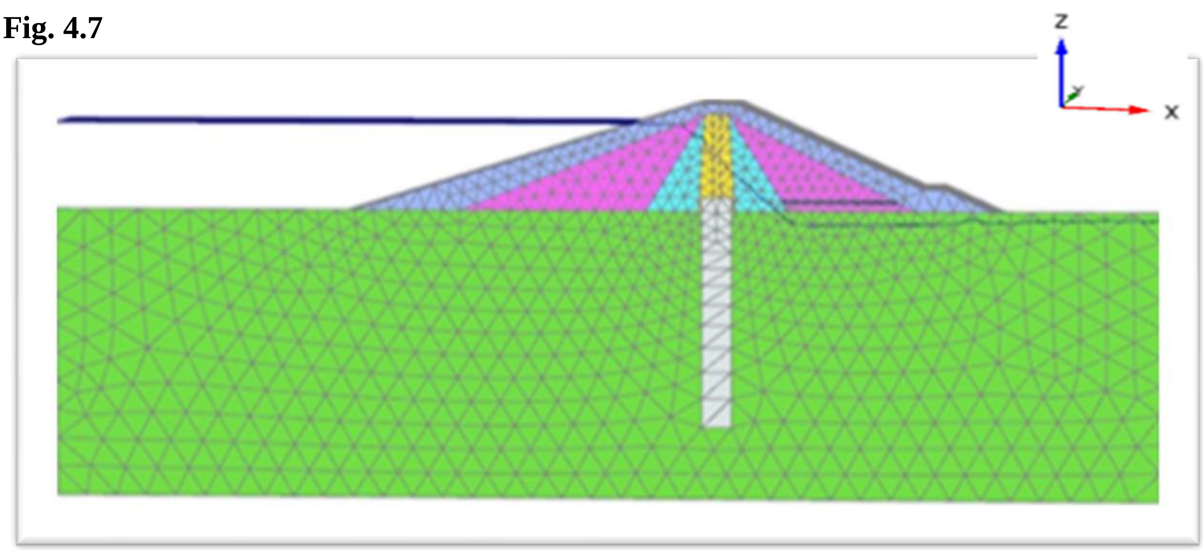
Lenged

1. Core Clay gravelly, 2. Zone 2B Gravel with fine –grained soil, 3. Zone 2A Gravel with loam, 4. Zone 3 Gravel with fine-grained soil, 5. Gravel Drain, 6. Gravel with loam, 7. Curtain Grouting, 8. Pile.

- ***Mesh generation and boundary conditions***

In this modelling, 10-node tetrahedral elements for soil elements were used Fig (4. 7). A sufficient and well-refined mesh was generated.

Fig. 4.7



Generated mesh

- ***Jet grouting procedure***

Figure (4.8) shows the construction steps can be summed up as following:

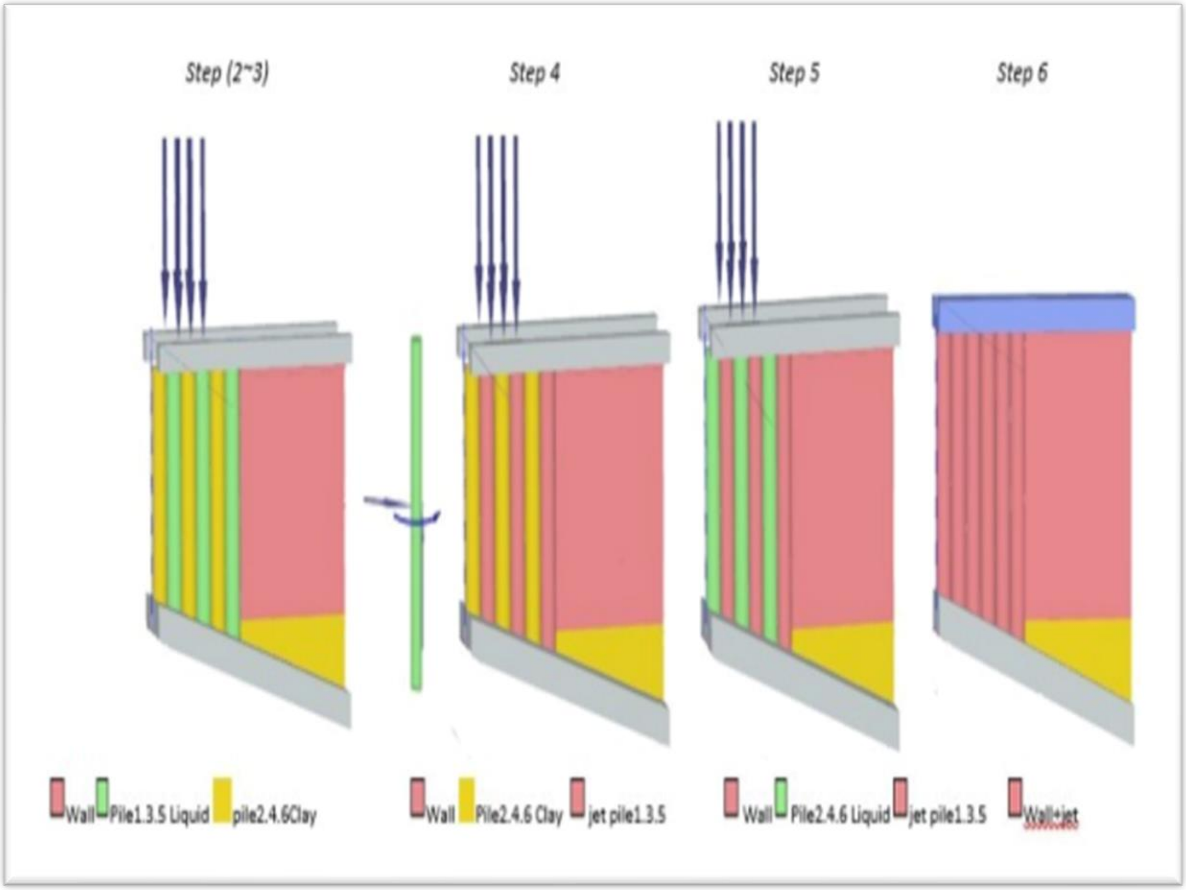
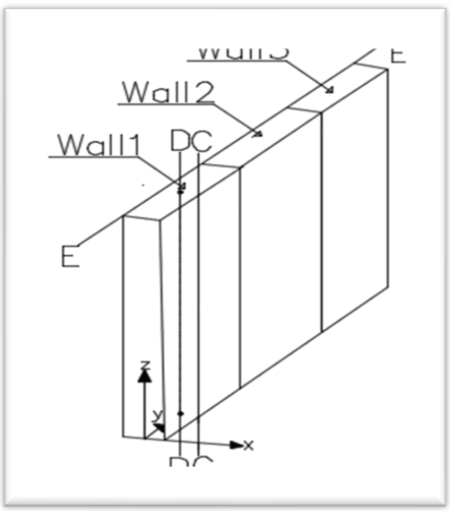


Fig. 4.8 Pile construction sequence



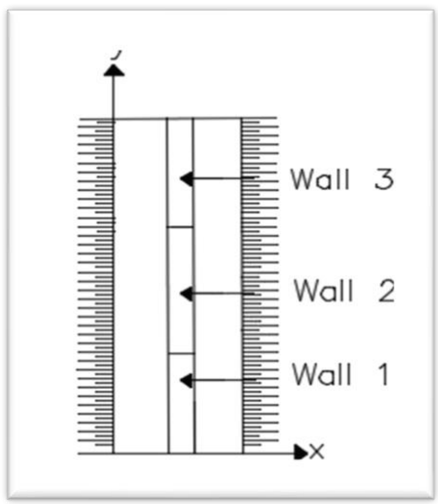


Fig. 4.9 Cross -Section lines

Fig. 4.10 Top view of the dam crest

5 RESULTS AND DISCUSSION

5.1 Diaphgram wall

5.1.1 Ground water head

Figures(5.1) and (5.2), show the variations of ground water head during decrease and increase water respectively. This result was concluded depending on FCFD analysis which analyses the development of deformation and pore water pressure as a result of time-dependent hydraulic boundary condition . In other words, it takes into account the permeabilities, the change of pore water pressure, and time.

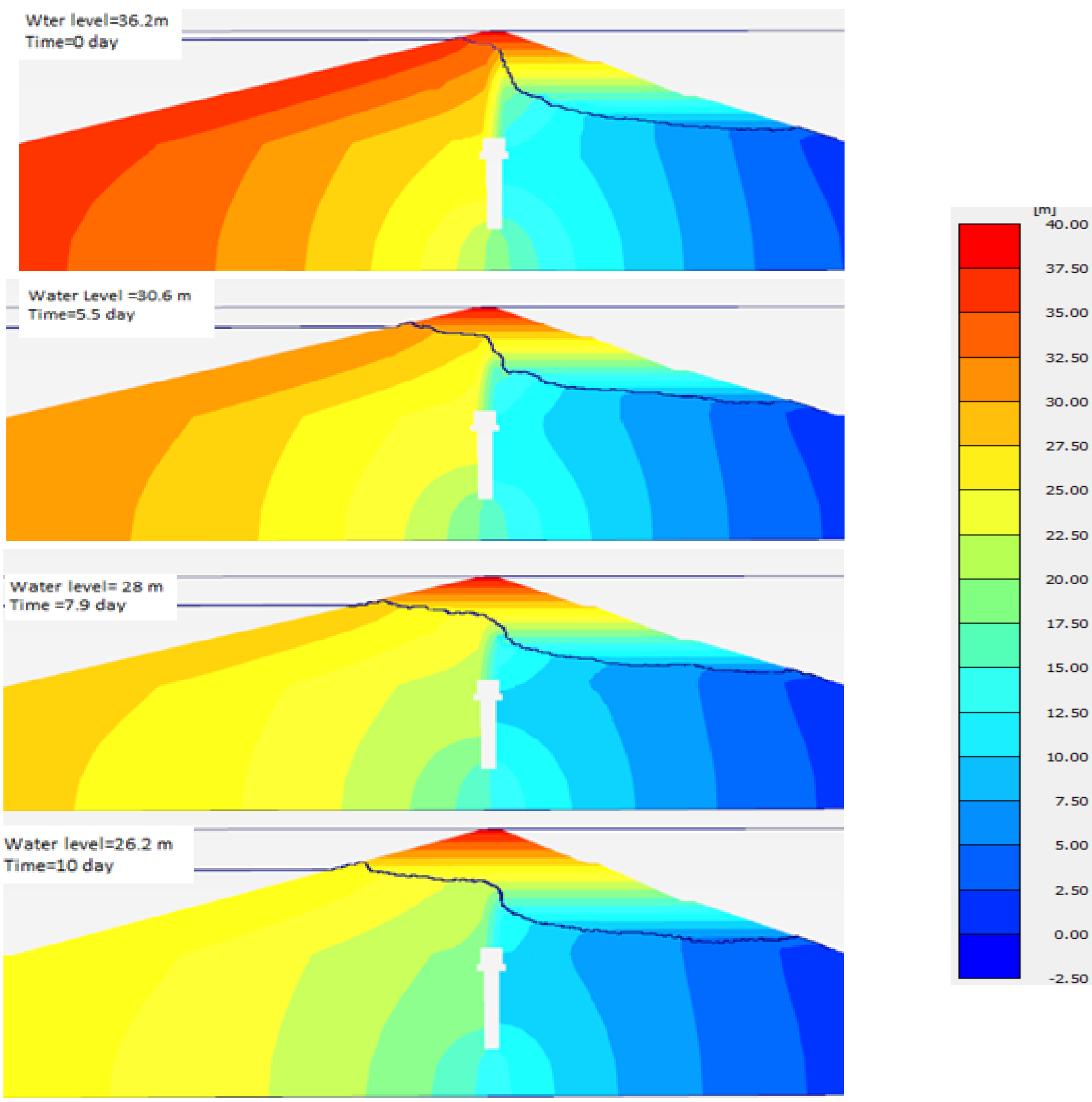


Fig. 5.1 Variations of ground water head (decrease WL)

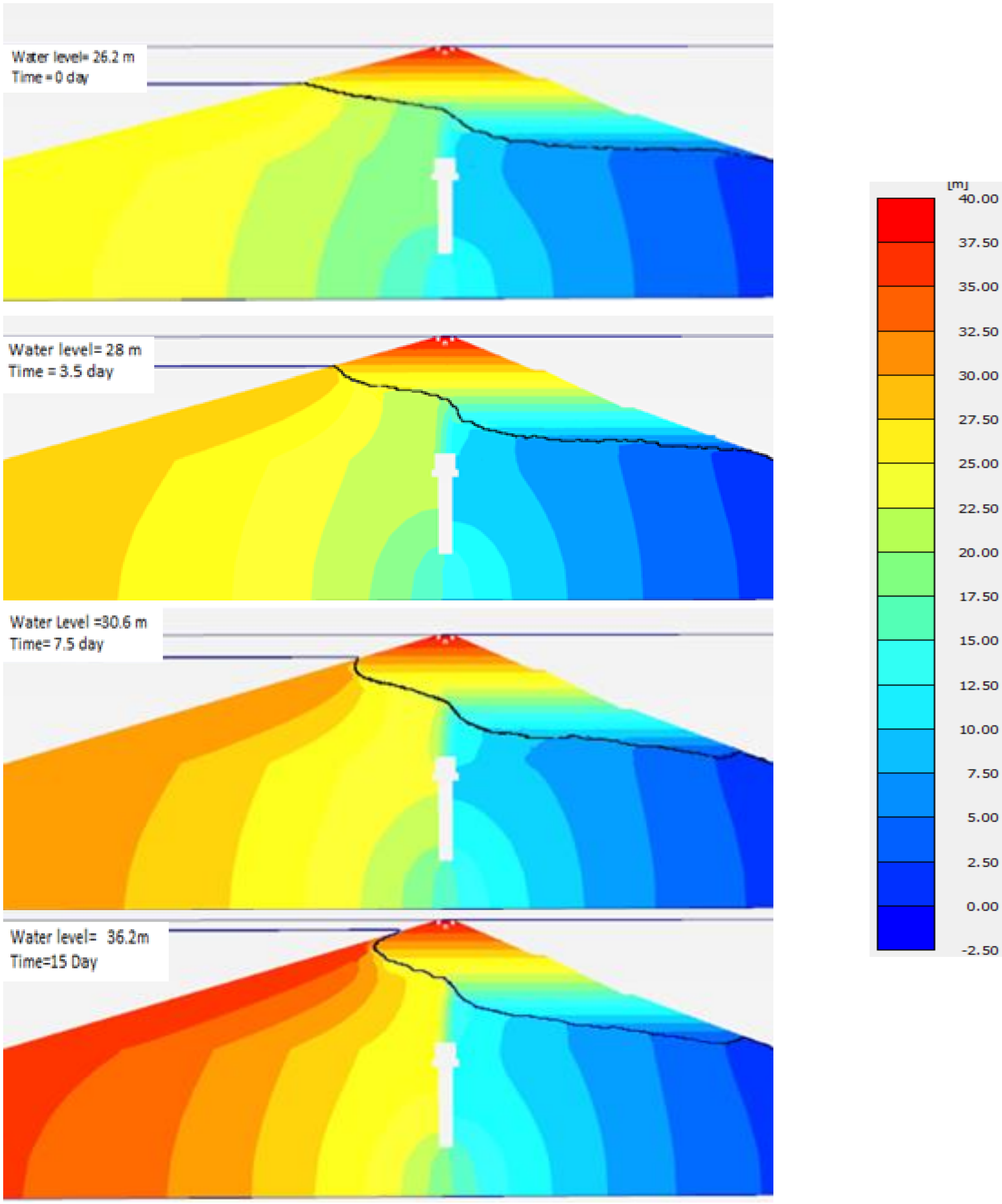


Fig. 5.2 Variation of ground water head (increase WL)

5.1.2 The total displacement

Displacement results are expressed in Figures (5.3), and (5.4) which show the horizontal displacement with respect to the time (drawdown-fill) respectively at crest point A (see Figure

4.9). The maximum value of the horizontal displacement reached 32 mm during decreasing WL and 23.5 mm during increasing WL.

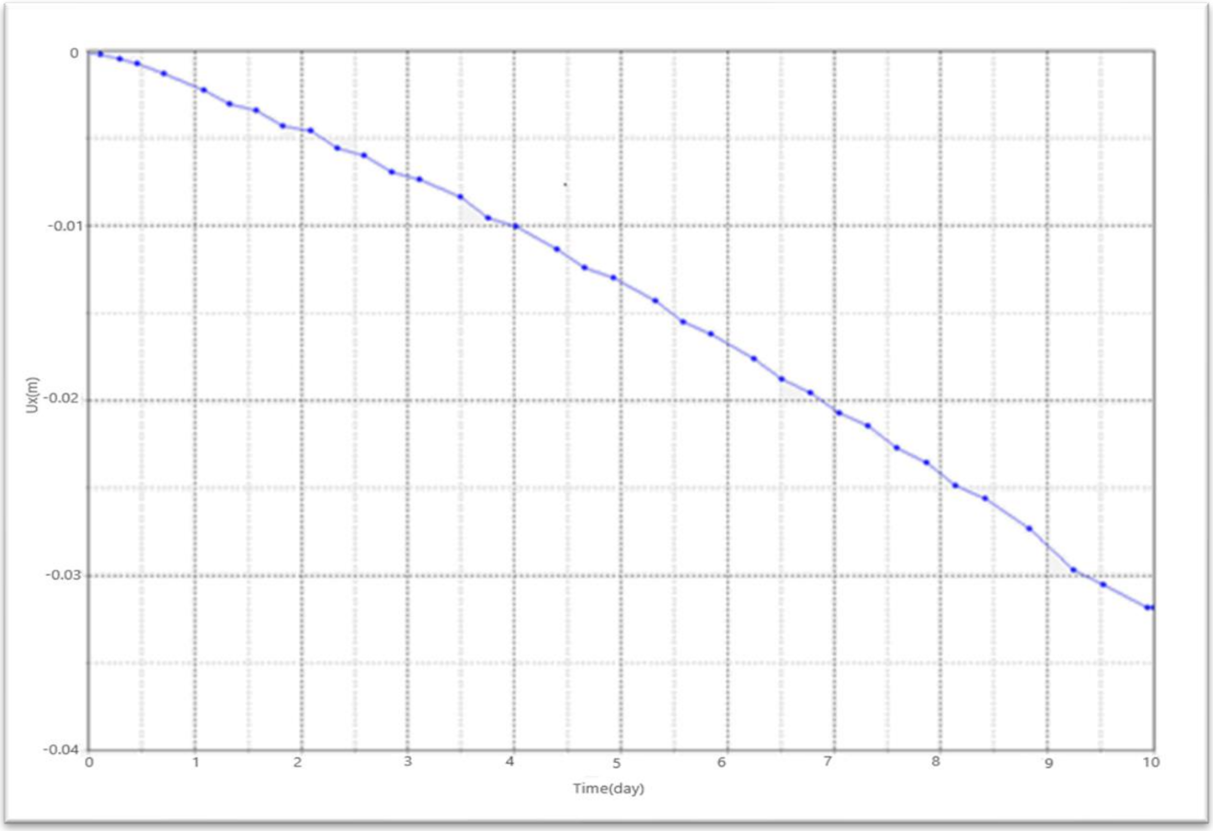


Fig. 5.3 Horizontal displacement-time (decrease water) history at point A (-2.5, 0, 39)

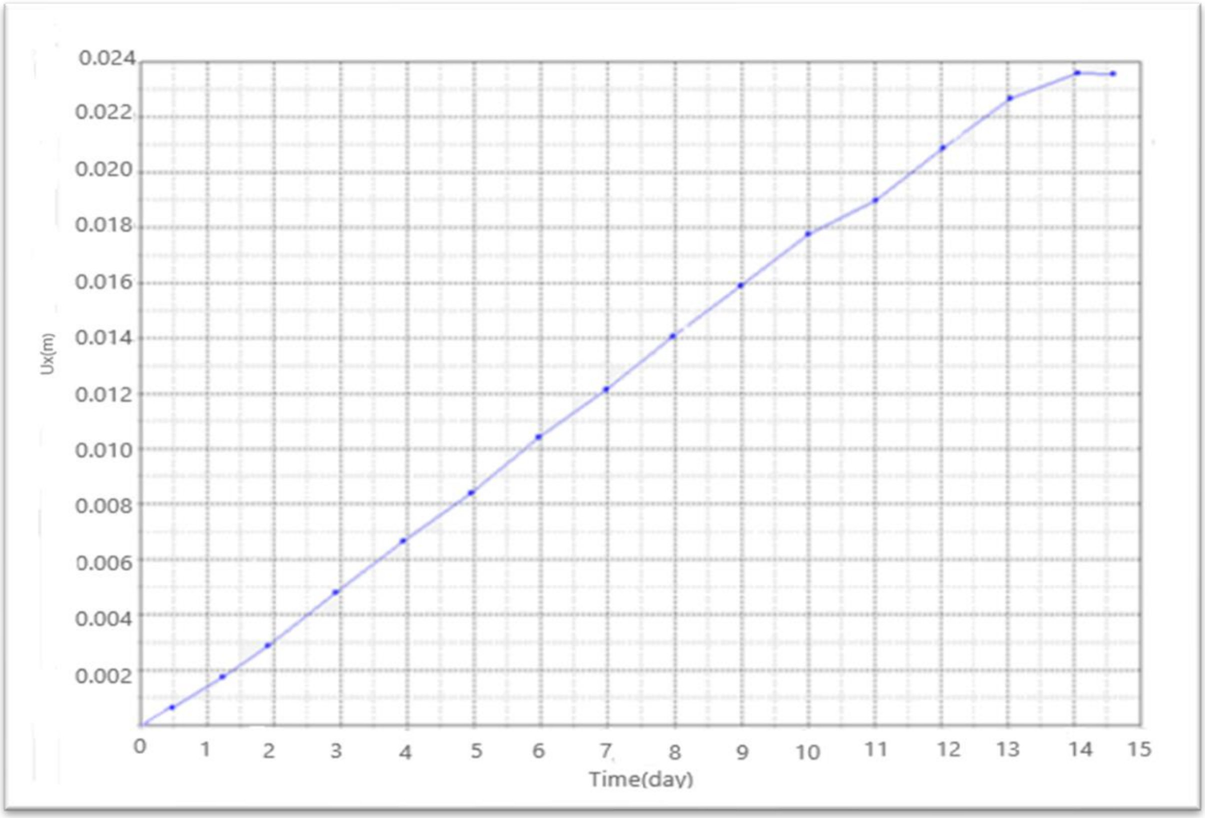


Fig. 5.4 Horizontal displacement-time (increase water) history at point A (-2.5, 0, 39)

5.1.3 Safety factor

Figures (5.5) and (5.6) depict that the most critical surface in the initial state is deep with a large radius. Also it is less deep with smaller radius in the last state. It is found to be near the upper part of the core and berm before reconstructions so any remedial steps applied to lower the seepage at the clay will have essential improvement in FS. The value of SF increases in this analysis, it goes from 1.48 to 1.56.

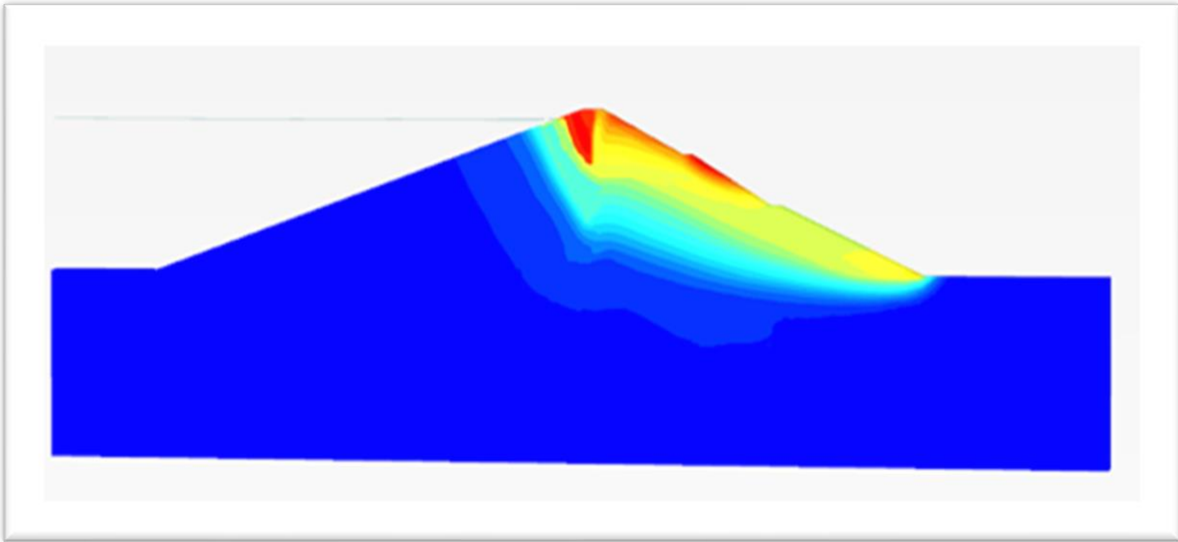


Fig. 5.5 Slip surface at failure (Initial state), **FS =1.48**

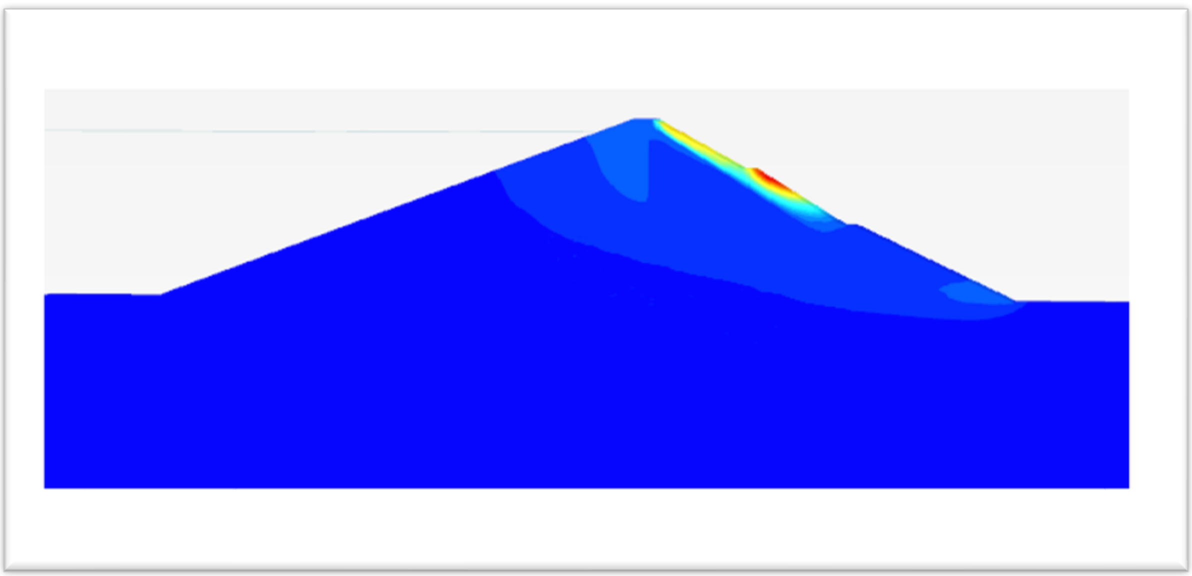


Fig. 5.6 Slip surface at failure (Last state), **FS =1.56**

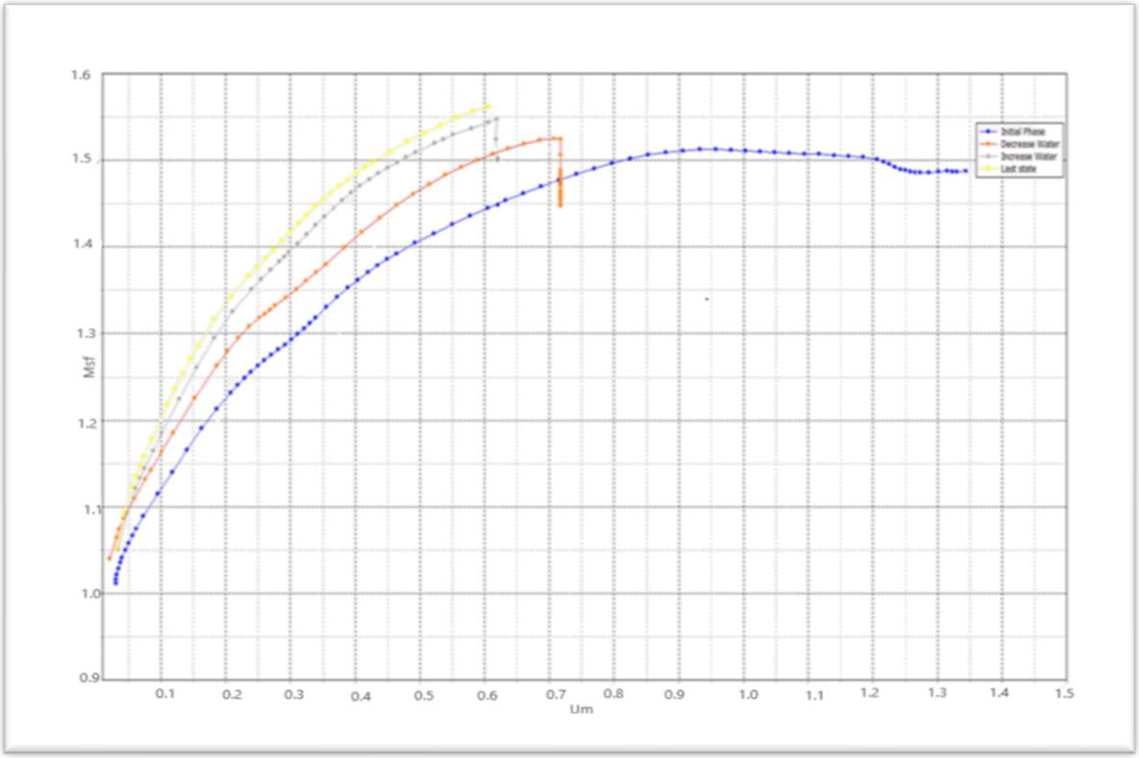
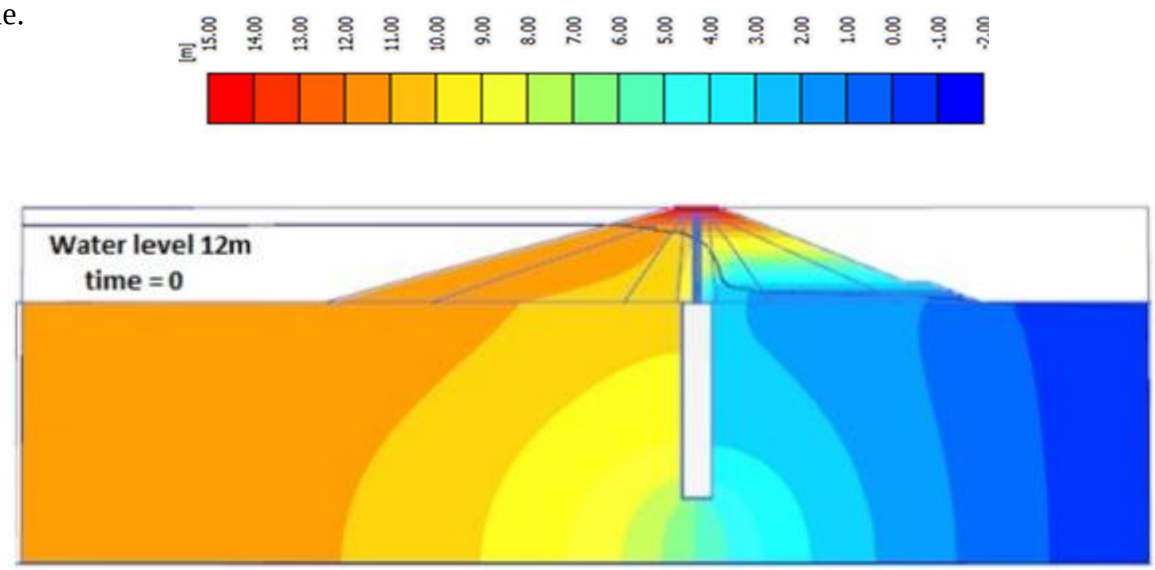


Fig. 5.7 Evaluation of safety factor
Initial state Decrease WL Increase WL Last state

5.2 Jet grouting

5.2.1 Ground water head

Figures (5.8) (5.9), show the variation of ground water head during decrease and increase water respectively, taking into consideration the influence of pore water pressure variations with the time.



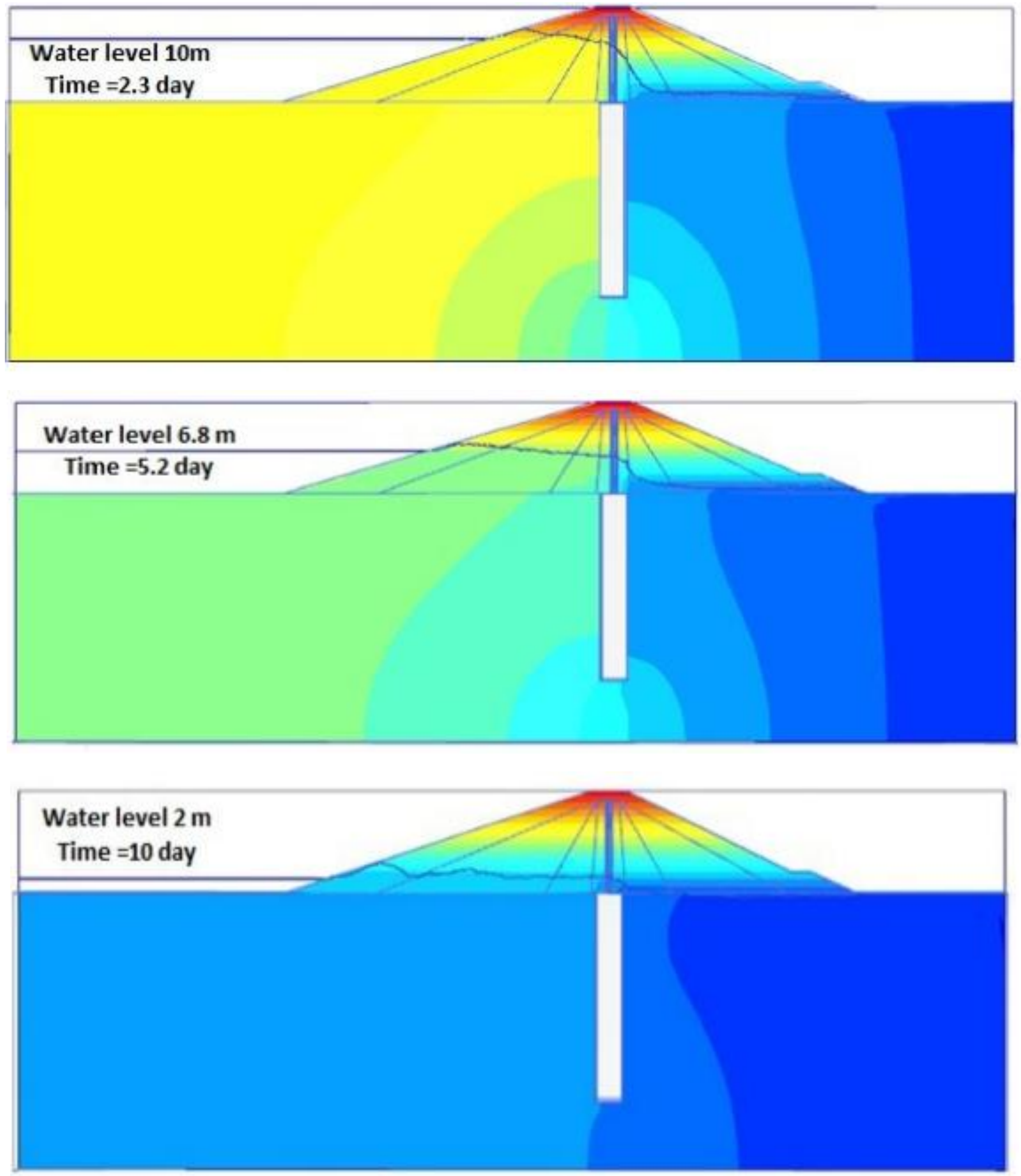
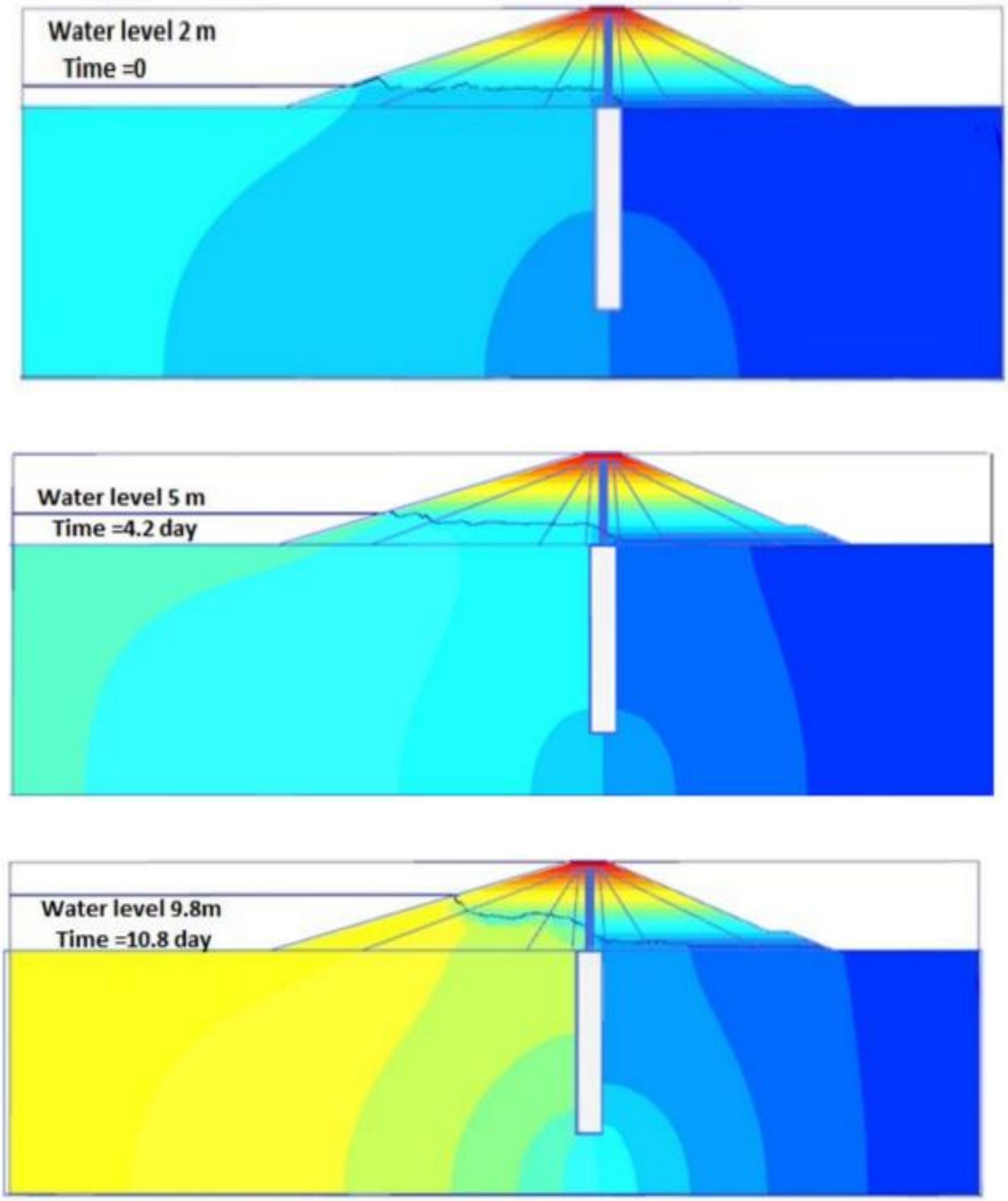


Fig. 5.8 Variation of ground water head (decrease WL)


Water level 2 m
Time =0
Water level 5 m
Time =4.2 day
Water level 9.8m
Time =10.8 day

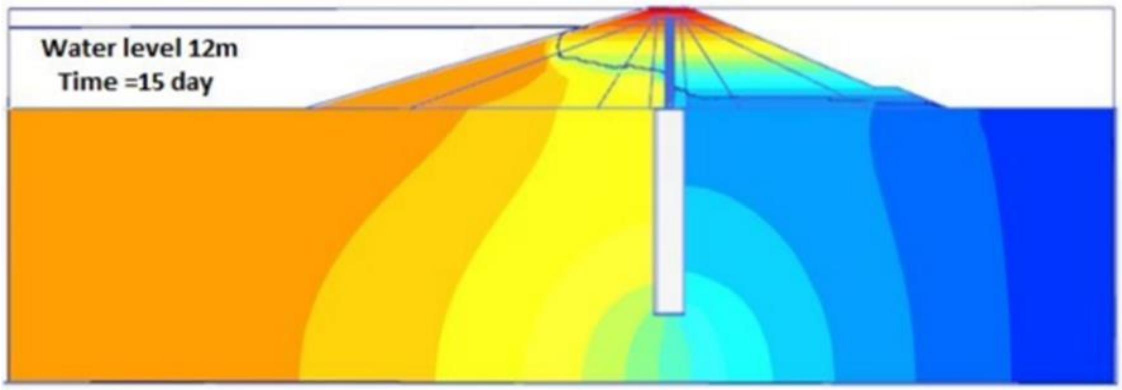


Fig 5.9 Variation of ground water head (increase WL)

5.2.2 The total displacement

Figure (5.10), shows the horizontal displacement distribution with depth at line cross section C-C Figure (4.9). It is clear that the maximum value of horizontal displacement reached 17.5 mm during decrease WL in reservoir, and 10.9 mm during increase the water.

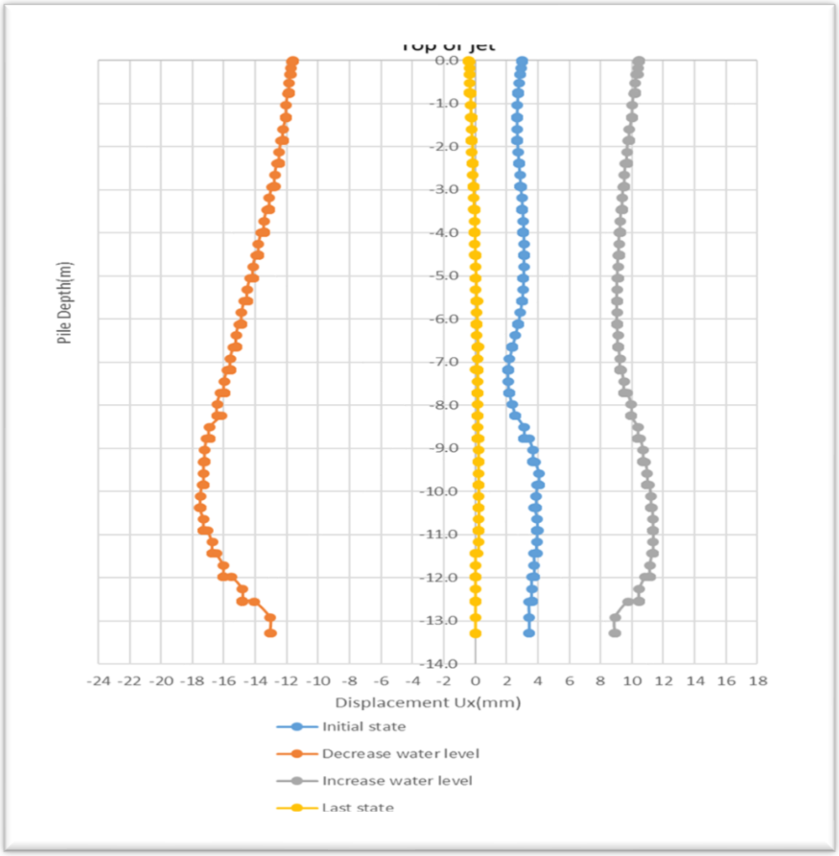


Fig 5.10 The horizontal displacement along the line cross section

C-C (construction pile 1, 3,5)

5.2. 3 Safety factor

The failure surfaces generated from the analysis are given in Fig (5.11) and (5.12). The failure is shallow, flatter with a small radius in both stages (initial state. last state) and the most critical surface in both stages is at the top of the dam. with the little difference in its shape can be ignored. Figure (5.13) shows evaluation of safety factor. SF even goes a little bit as up as 1.62 for the last state.

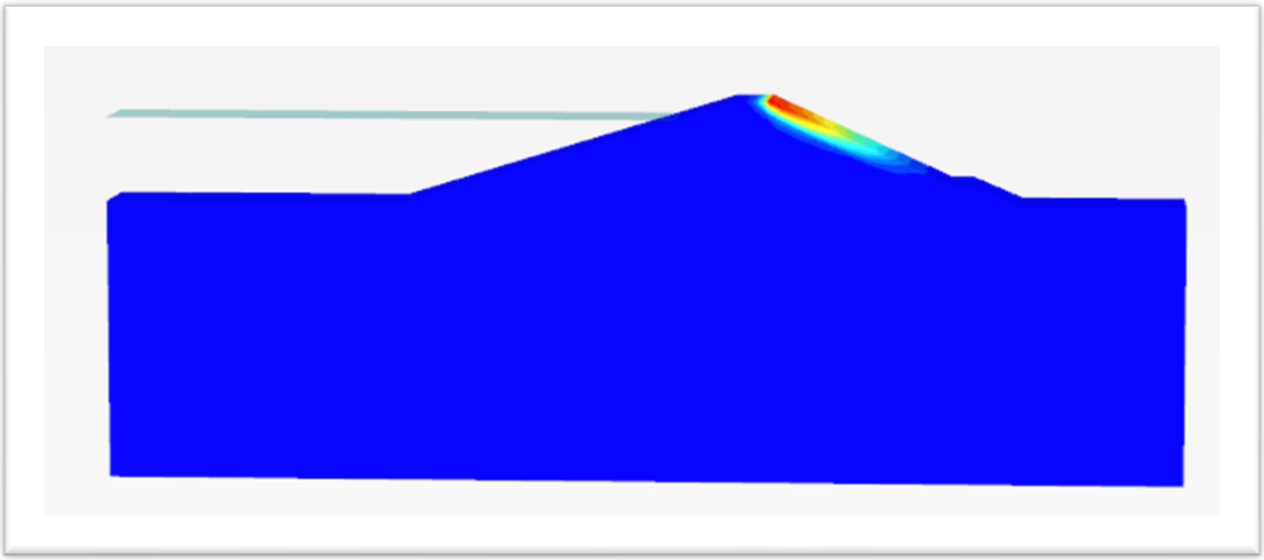


Fig. 5.11 Slip surface at failure (Initial state), **FS =1.60**

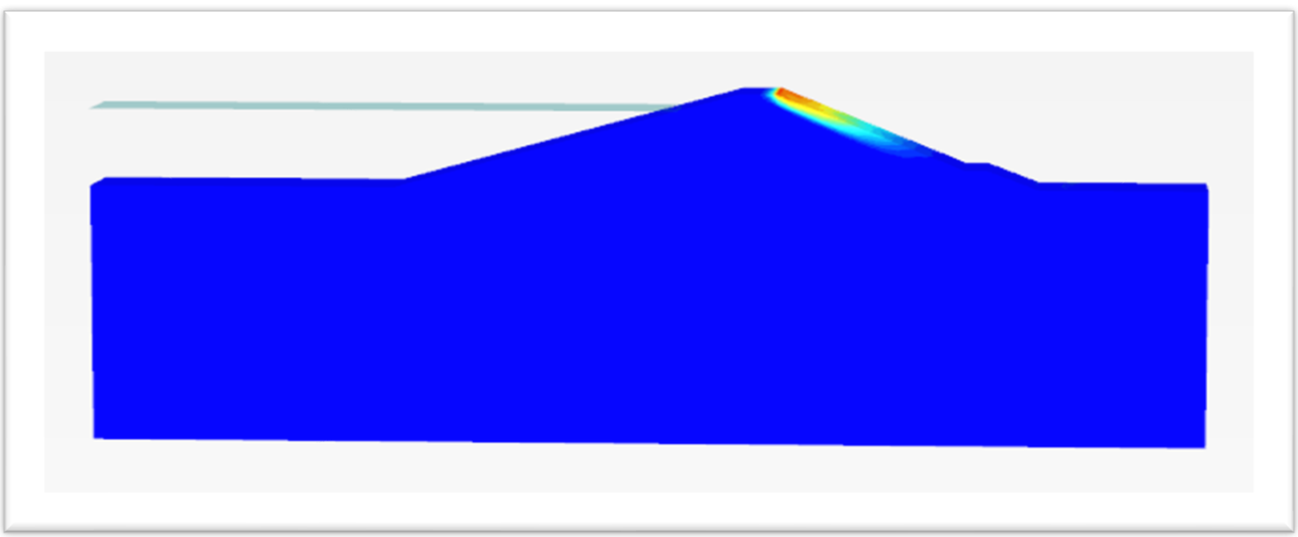


Fig. 5.12 Slip surface at failure (Last state), **FS =1.62**

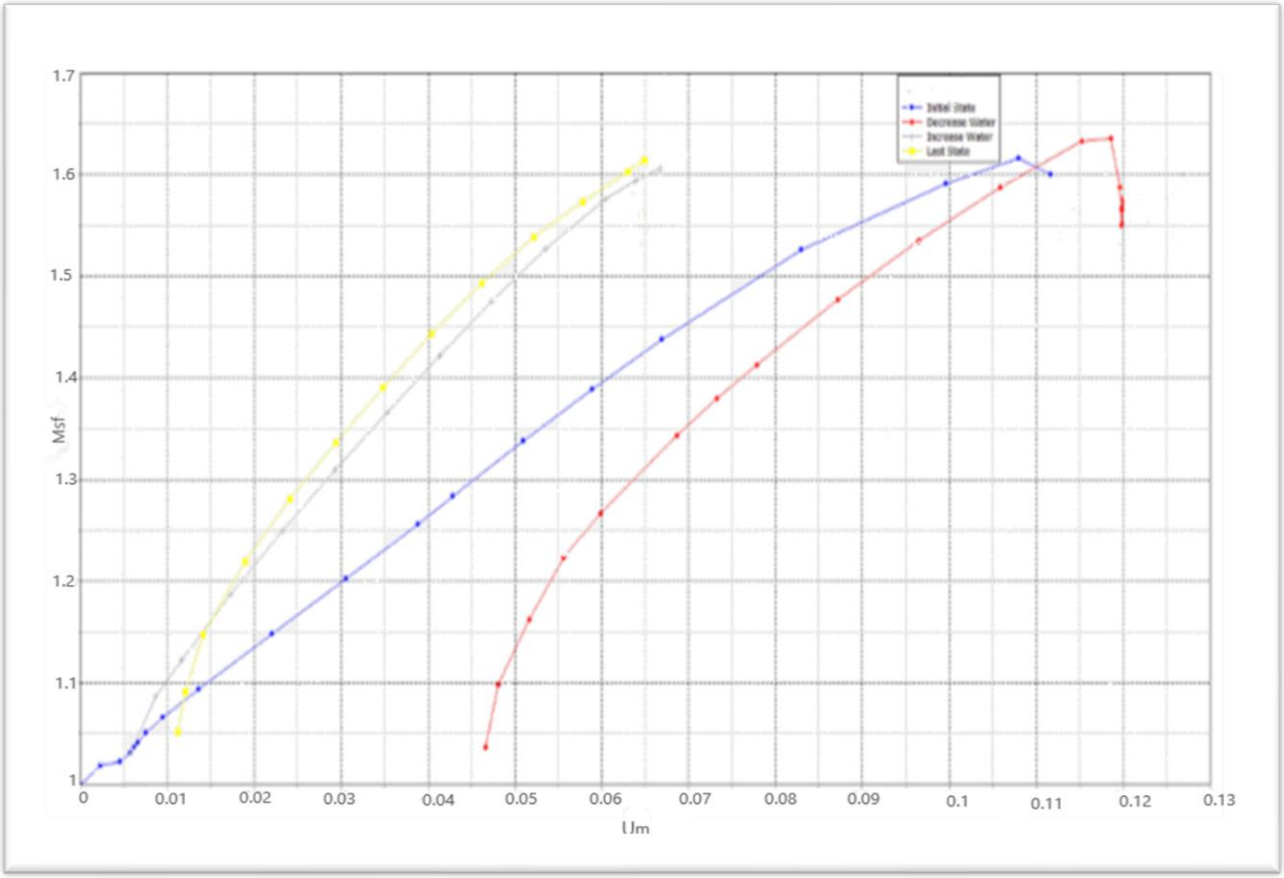


Fig. 5.13 Evaluation of safety factor

Initial state Decrease WL Increase WL Last stat

5.2.4 Stress state

Figure (5.14), shows the status of principal stresses for all points considered in the connection zone at line cross section C-C Figure (4.9) MC failure envelope is drawn for core (clay). According to this figure, the minor principal stress is compressive in all connection zone, so no probability of hydraulic fracture occurrence in the connection zone, and the failure of the core does not occur for the connection system.

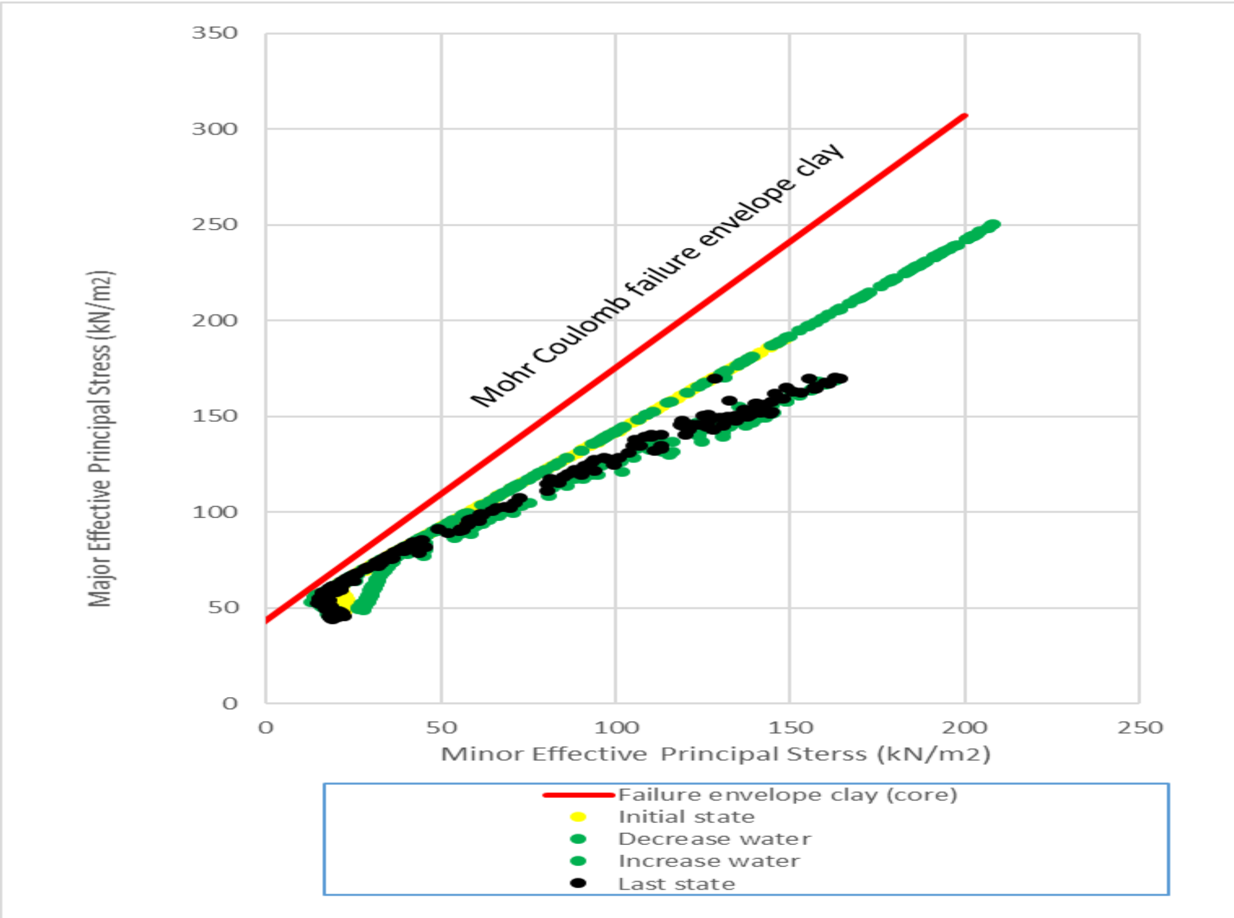


Fig 5.14 The investigation of the failure for grouting system in the core

6 CONCLUSIONS

This study reports the ability of the sealing to reduce water flow and increase stability of the dam. It investigates the potential effect of the sealing on the studied dam. This work shows the valuation of safety factor during reconstruction stage of Karolinka dam, and minimize the efforts when verifying the safety of slopes in site, in addition to performing designs of excavating works. It presents some charts to facilitate the site work.

It is clear in our case study that satisfying changes of the soil state before and after sealing can be noticed. It is quite difficult to model the case study without specified material properties, so some reasonable values should be input. In this study, a numerical investigation is conducted using Plaxis software which is based on the (FEM), and the results are compared to the experimental and analytical data performed by Vodni dila -TBD company. The main findings in this study can be summarized as follows:

Diaphragm wall:

1. The prediction of this study for the vertical displacement at crest dam (diaphragm wall installing) is (8.2mm), which is comparable to measured value (12.2 mm) in the field measurement of displacement. The total displacement due to an impact of loading drill is (13mm), which is comparable to calculated value (8mm). Also the horizontal displacement is (23.4 mm) which is comparable to (35.1mm) of the horizontal displacement in the field measurement and analytical data performed by Vodni Dila-TBD company (Hodák, 2014).
1. The horizontal displacement at the point A (-2.5,0,39) during decreasing water reaches its height value (32 mm), and during increasing water is (23.5 mm), due to the influence of water load and pore water pressure variations with the time.
2. The most critical surface in both cases (initial state, last state) is near the upper part of the core and berm so any remedial step is applied to lower the seepage at the clay will have essential improvement in SF.
3. The value of safety factor before reconstruction stages is (1.48), which is compared to the calculated value depended on: **1-** the shape of failure surface, 2- the data taken from measuring well, **3-** Bishop method, equals (1.498) (Bednárová and Gramblièková, 2006).
4. The results of safety factor consider the cross-section positions in two cases: **1-** in the middle (diaphragm wall case), **2-** in the end of dam (jet grouting case). The results show that the value of safety factor in the middle of dam -where the highest height- equals (1.48). On the other hand, the highest value of the safety factor in the end of dam -where the lowest height-equals (1.6). As a result, the height of dam has a clear impact on the value and shape of the failure surface and the safety factor.

5. The value of safety factor before reconstruction stage is (1.48) and after reconstruction is 1,56 and this result is compatible with the traditionally required value of 1.5 (ČSN 75 2410). As a result, diaphragm wall is an effective technology to improve dam stability (Fathani and Legono, 2011).
6. It is noted that the variation of WL (decrease- increase) affects SF because of water movement in the soil pores, thus reducing the effective stress, soil strength and stability.
7. The applied element method is a trusty tool for the installing process of diaphragm wall in the earthen dam. The numerical evaluation using FEM analysis was successfully carried out to investigate the effects of installing process on the surrounding soil.
8. The diaphragm wall is analysed by the presented 3D analysis, taking into consideration the influence of pore water pressure variations with the time.
9. Good matching between the measured and numerical results has been obtained using Plaxis.

Jet grouting:

1. The horizontal displacement in the connection zone (cross section C-C) reaches its height value during decreasing water is (17.9 mm), and during increasing water is (11mm), taking into consideration the influence of pore water pressure variations with the time.
2. The failure is shallow, flatter with a small radius in both stages (initial state. last state) and the most critical surface in both stages is at the top of the dam. The safety factor even goes a little bit as up as 1.62 for the last state, with the little difference in its shape can be ignored.
3. Regarding the investigation of failure state, the minor principal stress is compressive in all connection zone, so there is no probability of hydraulic fracture occurrence in the connection zone, and the failure of the core does not occur for the connection system.
4. It is very important to choose the appropriate period for decreasing and increasing water level in the reservoir. In uncontrolled drawdown, water load disappears so, there is no supporting pressure to dam stability. Also, the generated tensile-downward forces lead to a decrease in shear strength of the upstream slope. On the other hand, the unplanned filling the reservoir creates excess pore pressure which may put the dam at risk in some critical conditions. As for the case studied and depending on some recommendations (ČSN 75 2310) the level of water was decreased by one meter per day.
5. The process of jet grouting in the case study was modelled by using Plaxis 3D analysis, taking into consideration the influence of pore water pressure variations with the time.
6. The measured and numerical results seem to be close.

7 REFERENCES

BEDNÁROVÁ, E., GRAMBLIČKOVÁ, D. 2006. VD Karolinka – těsnící clona. Dokumentace pro vydání stavebního povolení, Hydroconsulting s.r.o. Bratislava.

BOLTON M. D., STEWART D. I., 1994. The effect on propped diaphragm walls of rising groundwater in stiff clay .Journal of Geotechniqu.Vol 44 (1): pp. 111-127.

BRINKGREVE, R. B. J., ENGIN, E., SWOLFS, W. M. 2017. PLAXIS 3D Anniversary edition. [online] https://www.plaxis.com/support/manuals/plaxis-3d- manuals.

ČSN 75 2410. 2011. Malé vodní nádrže. Small water reservoirs. Prague

ČSN 75 2310. 2006. Sypané hráze. 2006. Embankment Dam. Prague.

DAWSON, E. M., ROTH, W. H., DRESCHER, A. 1999. Slope stability analysis by strength reduction. Geotechnique. Vol. 49(6): pp. 835–840.

FANG, S.Y., LIAO, J. J., SZE, C, S. 1994. An empirical strength criterion for jet grouted soilcrete. Engineering Geology. Vol. 37(3): pp. 285-293.

LURA, P .2003. Autogenous Deformation and Internal Curing of Concrete. DUP Science Netherlands. ISBN: 90-407-2404-0.

NASH, D. 1987. Comprehensive Review of Limit Equilibrium Methods of Stability Analysis Journal of Slope Stability. Chapter 2. pp. 11-75. Wiley. New York.

PAŘÍLKOVÁ, J., ZACHOVAL, Z., PAŘÍLEK, L. 2016. The earth-fill dam of the Karolinka water reservoir monitored by the EIS method. Eureka 2016. 4th Conference and Working Session within the Framework of the International EUREKA, Project No.: E!7614, Lednice, Czech Republic 13–14.10.2016, pp.185–209.

PUBLICATIONS OF THE AUTHOR

BREDY, S. 2014. Přehrady v Syrii. Časopis Vodní hospodářství. Vol 4: pp. 34-37.

BREDY, S., JANDORA, J. 2019. Three-Dimensions Modelling of a Jet Pile Construction in the Karolinka Dam. Acta Journal. Vol 67(3): pp. 621-636.

BREDY, S., JANDORA, J. 2019. Numerical Modelling of a Diaphragm wall Process in Karolinka Dam. International Journal of Sciences: Basic and Applied Research .Vol 48(7): pp. 93-109.

BREDY, S., JANDORA, J. 2020. Effect of Dam Height on The Stability of Earth Dam (Case Study: Karolinka Dam). Journal of Engineering. Vol 26(3): pp. 117-126.

ABSTRACT

This study focuses on the most important sealing technologies; Diaphragm walls and Jet grouting as a major most popular reliable option when it comes to engineering constructions rehabilitation. Those two methods have been used in Karolinka dam for reducing seepage through its body. Diaphragm walls were used along the dam, and jet grouting was used at both ends of the dam. The study deals with the possibility of numerical modelling of these two technologies. It is included how to carry out, interaction with adjacent soil, cement shrinkage, slope stability, changing of pore water pressure with the time, dynamic analysis of drilling rod and the effects of both technologies on slope stability, seepage and settlement of dam. This modelling was conducted with the finite element method based on software Plaxis 3D.