Effects on an earth- and rockfill dam undergoing dam safety measures

J. Toromanović, H. Mattsson, S. Knutsson
Luleå University of Technology, Sweden, jasmina.toromanovic@ltu.se

J. Sipola
ÅF Hydropower Industry, Sweden

ABSTRACT
Over the lifetime of a dam several measures are usually taken in order to assure the stability and the performance of the dam. In this case a hydropower dam in Northern Sweden is in need of dam safety measures. The question arose, what consequences there might be when such measures are performed. In order to estimate these effects, simulations have been carried out in the finite element programme PLAXIS 2D. Thereby, the deformations and the stability of the dam for the planned work can be evaluated. The performed simulations are based upon previously conducted research at Luleå University of Technology, where soil parameters in the investigated dam were identified by a method of inverse analysis.

Three sections have been analysed: A, B and C. In section A increasing pore water pressure has been observed at the downstream side of the dam. Thereby it has been concluded that a new drainage system is needed; new trenches of large size are to be excavated. In section B new toe berms are planned, due to the requirement that the dam should be able to divert leakage without erosion occurring at the dam toe. This contains soil material that might degrade when stresses are increased, with intensified deformations as a consequence. In section C a new berm is to be constructed, before this can be conducted an excavation is performed at the toe of the dam.

The results have shown deformations of an acceptable magnitude and factors of safety that indicate conditions for the planned dam safety measures. Numerical values of deformations and factors of safety can be utilised as an attempt to establish alarm values for the stability of the dam. The finite element method is a useful tool for this kind of evaluation.

Keywords: dam, numerical modelling, PLAXIS, displacement, factor of safety

1 INTRODUCTION

In Sweden, a number of dams were constructed during the 1960s; some are today in need of measures in order to ensure the dam safety and dam performance. In this paper some dam safety measures are modelled for a hydropower dam by utilising the finite element software, PLAXIS, see Brinkgreve, Engin & Swolfs (2014). The effects of the measures at the dam body are evaluated by analyses of deformations and stability; the usability of the finite element method for this case is studied.

The finite element method (FEM) has been widely utilised for modelling within various disciplines, geotechnical engineering included. Numerical modelling considering geotechnical applications is described in numerous amounts of literature, for instance Potts & Zdravković (1999) and Muir Wood (1990). One of the advantages of performing finite element analyses, is the comprehensive applications when dealing with more complicated geotechnical problems.

By choosing a proper model for representing the soil behaviour during numerical modelling, the reality can be well described. This requires description of the elasto-plastic behaviour of the soil material. Theory of elasticity is often not sufficient, since soil behaves elasto-plastic. Thereby implementation of plasticity is usually required. Information regarding plasticity can be retrieved for example from Yu (2006).

However, no model is perfectly representing the behaviour of the soil material; the choice can be based upon the problem to be solved, the properties of the constitutive model and the available material data.

Problems are often faced when computational modelling is to be conducted for earth and rockfill dams. The reason is usually insufficient amount of reliable data for the material properties. Investigations by field testing are not easily performed, especially in the impervious parts, due to the probable negative effects on the dam performance as well as the dam safety. Therefore other methods, preferably non-destructive, have to be found.

Constitutive behaviour of the soil material within the dam structure can be determined by a method of inverse analysis. This is only possible if the dam is equipped with various instrumentations that are monitoring for instance pore pressures, deformations or seepage. Vahdati (2014) used an error function and a search algorithm combined with the finite element software PLAXIS to identify soil parameters of the dam in the study. Model parameters in the elasto-plastic constitutive models were calibrated until the simulated values for the deformations corresponded to the deformations from the inclinometer data.

2 CASE STUDY

With the aim to improve the dam safety and the performance of a hydropower dam in Northern Sweden, measures have been projected by the consulting company ÅF. The hydropower dam consists of earth- and rockfill, including a central impervious till core. Adjacent to the core, there are fine and coarse filters. As a supporting layer, rockfill is placed at each side of the filters. In some sections there are supporting berms on the downstream side of the dam. The dam body is partially founded on bedrock and partially on glacial till.

Various design solution for the dam safety measures of the dam were suggested by the consulting company ÅF. Thereafter the question arose of what effects these measures would exert on the dam structure. Three cross-sections have been chosen for the analyses; A, B and C in Figures 1-3.

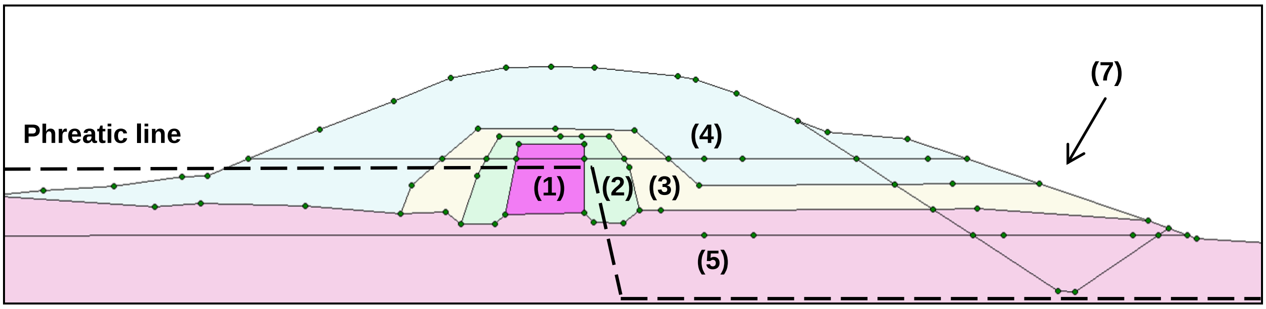


Figure 1 Section A, geometrical model of dam body. The zones are: (1) core, (2) fine filter, (3) coarse filter, (4) rockfill, (5) foundation consisting of till and (7) new trench.

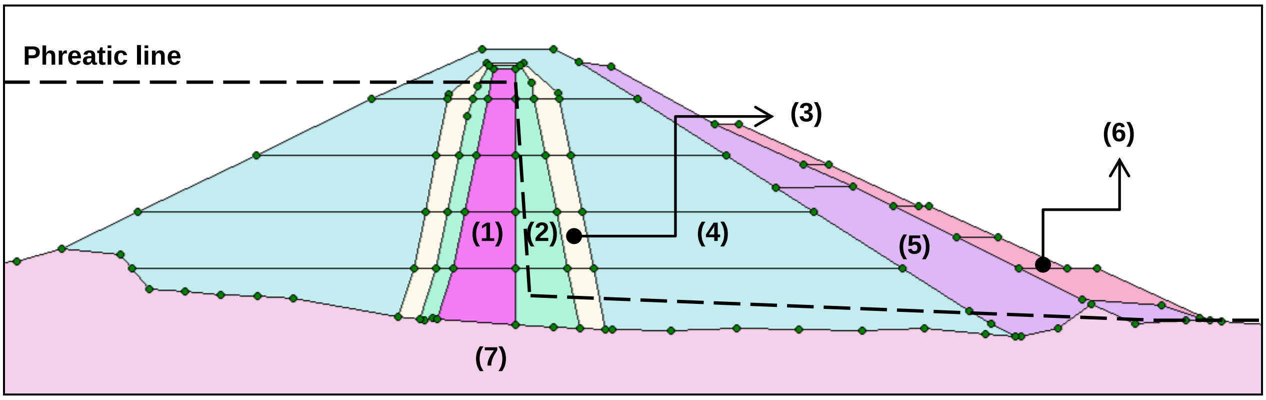


Figure 2 Section B, geometrical model of dam body. The zones are: (1) core, (2) fine filter, (3) coarse filter, (4) rockfill, (5) berm, (6) new berm and (7) foundation consisting of rock.

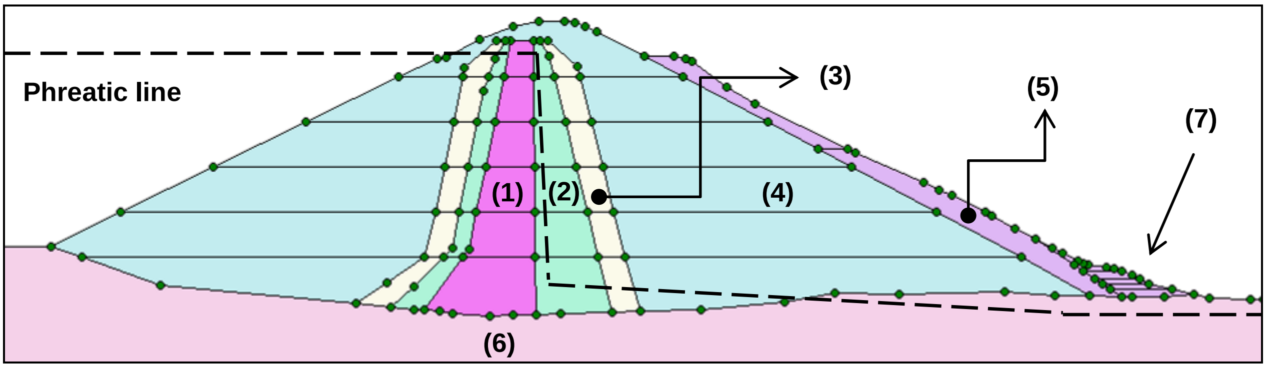


Figure 3 Section C, geometrical model of dam body. The zones are: (1) core, (2) fine filter, (3) coarse filter, (4) rockfill, (5) berm, (6) foundation consisting of rock and (7) planned excavation.

2.1 Section A

In measuring gauges, at the downstream side of the dam, a trend of increasing pore water pressure has been observed. Thereby, the drainage system has been deemed as not properly functioning. A new trench is therefore planned at the dam toe, see Figure 1. In the chosen section the depth of the planned trench is the largest compared to the height of the dam. Thus the most extensive effects of the planned work are expected in this section.

2.2 Section B

According to the Swedish dam safety guidelines, RIDAS by Svensk energi (2002), a dam should be able to divert leakage without erosion occurring at the toe. For the dam studied a new toe berms is required to attain this guideline. The planned berm is shown in Figure 2. The highest section of the dam is chosen for the analysis, due to the largest deformations being expected there.

During the construction of the already existing berm, denoted as (5) in Figure 2, abnormally large deformations occurred at the downstream side of the dam; this was noticed by the dam owner and numerically by Vahdati (2014). A possible explanation of this behaviour is degradation of the rockfill material, due to the added external load. The degradation results in a more fine grained soil mass, which in turn can cause increasing deformations.

Two cases are to be analysed when the new berm is added, one including the degradation of the rockfill material and other without the degradation.

2.3 Section C

In this section, see Figure 3, a new toe berm is needed based upon the same RIDAS requirement as for section B. Due to the insufficient space at the downstream side of the dam, a retaining wall is to be constructed. Before the retaining wall can be constructed, an excavation is performed. The excavation has a limited extent in the transversal direction.

Effects of the excavation are analysed. Influences of the new berm on the dam body are not inspected in this study.

3 CONSTITUTIVE MODELS AND MATERIAL PARAMETERS

The constitutive models Mohr Coulomb and Hardening soil have been utilised during the case study. Both models are regarded as suitable for their respective application area; though the Hardening soil model is in general considered more versatile. The applicability of the models is explained more thoroughly by Brinkgreve et al (2014).

The values for the unit weight, friction angle and permeability have been provided by the dam owner. Values for Poisson’s ratio and the cohesion are based on advices from Bowles (1988). The dilatancy angle has been assigned according to an empirical relation from Brinkgreve et al. (2014). The shear moduli and the reference secant stiffness of the core and rockfill was optimised by Vahdati (2014). Material parameter values from the constitutive model Mohr Coulomb are presented in Table 1. For the constitutive model Hardening soil, values are found in Table 2.

Table 1 Material parameter values for the constitutive model Mohr Coulomb, from Vahdati (2014) and ÅF.

Zone	γ_u kN/m^3	γ_s kN/m^3	E MPa	E_{inc} MPa	v' -	c' kPa	φ °	k_x/k_y m/s
Core	21	23	48.6	1414.8	0.35	20	38	3.0E-7
Fine filter	21	23	106.4	266.0	0.33	0	32	9.0E-5
Coarse filter	21	23	106.4	266.0	0.33	0	34	5.0E-4
Rockfill	19	21	26.6	159.6	0.33	7	30	1.0E-2
Foundation (rock)	21	23	1400	-	0.30	0	45	1.0E-8
Foundation (till)	21	23	20	-	0.30	0	36	1.0E-8

Note: γ_u is the unit weight above the phreatic level, γ_s the unit weight under the phreatic level, E is the Young's modulus, E_{inc} is the Young's modulus increment, v' the effective Poisson's ratio, c' the effective cohesion, φ the friction angle and k_x as well as k_y are the hydraulic conductivity in the horizontal and vertical direction respectively. E and φ for the foundation till are based on field data, provided by ÅF. γ_u, γ_s, v', c' and k_x/ k_y for the foundation till are chosen accordingly with the rock foundation parameter values.

Table 2 Material parameter values for the constitutive model Hardening soil, from Vahdati (2014) and ÅF.

Zone	γ_u kN/m^3	γ_s kN/m^3	$E_{50}^{ref} = E_{oed}^{ref}$ MPa	E_{ur}^{ref} MPa	m -	c' kPa	φ °	k_x/k_y m/s
Core	21	23	70	210	1	20	38	3.0E-7
Fine filter	21	23	50	150	0.5	0	32	9.0E-5
Coarse filter	21	23	50	150	0.5	0	34	5.0E-4
Rockfill	19	21	10	30	0.5	7	30	1.0E-2
Berm	21	23	10	30	0.5	7	30	5.0E-2
New berm	21	23	25	75	0.5	7	42	1.0E-2
Foundation (rock)	21	23	3000	9000	0.5	0	45	1.0E-8

Note: γ_u, γ_s, c', φ and k_x/ k_y are defined in the note under Table 1. E_{50}^{ref} is the reference secant stiffness for primary loading in a drained triaxial test, E_{oed}^{ref} is the reference tangent stiffness for primary oedometer loading, E_{ur}^{ref} is the reference stiffness for unloading/reloading in a drained triaxial test, m is the power for stress-level dependency of stiffness. The material parameter values for the new berm are based upon values for the previous berm, as well as information from ÅF.

In section B the potential degradation of the rockfill material (4), in Figure 2, can be considered by reducing the moduli with respect to the increased load. The decrease of the values for the moduli values were observed by Vahdati (2014), during the simulation of the construction of the berm (5), in Figure 2. The berm itself (5) is also consisting of the same material. Therefore the moduli values are linearly reduced with respect to increased load for both the rockfill (4) and the berm (5). The values for the reduced moduli are found in Table 3; the original ones are shown in Table 2.

Table 3 Reduced material parameter values due to potential degradation.

Zone	Reduced $E_{50}^{ref} = E_{oed}^{ref}$ MPa	Reduced E_{ur}^{ref} MPa
Rockfill	6.9	20.7
Berm	5.9	11.7

4 NUMERICAL MODELLING

The numerical modelling is based upon the research of Vahdati (2014), both considering the values of the material parameters, Table 1 and Table 2, as well as the established finite element model.

Plane strain conditions are assumed, since the dam is a long structure. All cross-sections, A, B and C, have been somewhat modified when importing the data to PLAXIS; some lines have been slightly smoothened out. A number of adjacent geometry points have been removed. However, these modifications have no significant effect on the results.

The size of the geometry model is chosen for all sections in such a way that the extent is sufficient for the accuracy of the computations. Standard fixities in PLAXIS are chosen for generation of boundary conditions. The outer vertical boundary lines are fixed in the horizontal direction. The horizontal bottom boundary line is fixed in both horizontal and vertical directions. The choice of finite element mesh is an important factor for the numerical solution strategy. A denser mesh usually produces more accurate results, but the computation time increases. A suitable mesh for the computation accuracy was chosen by refinement until the results did not vary significantly. Thereby a sufficient accuracy is obtained, for a minimum computation time.

The dam is built up in several steps, in order to create a proper initial stress field. The horizontal filters are omitted in the geometry models. This modification of the geometry is considered to have no practical influence in this study.

4.1 Section A

The foundation in section A consists of till. No parameter values were available for the till for the Hardening soil model. Therefore, the analyses of section A were conducted with the more simple Mohr Coulomb model for which the parameter values for the till were available.

The modelling of the excavation of the drainage trench is performed in five steps, as shown in Figure 4.

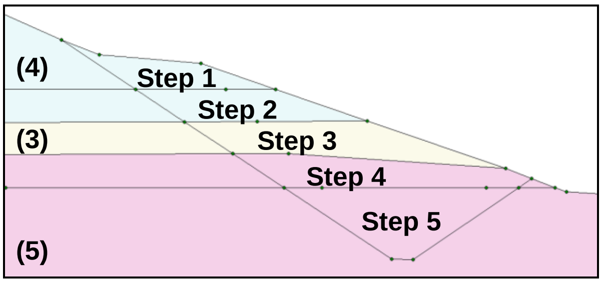


Figure 4 Excavation steps for the new drainage trench, material zones denoted according to Figure 1.

The inclination of the excavation walls is set to 1V:1.5H, based on a design suggestion by ÅF.

The phreatic line, see Figure 1, is assigned upstream to the retention water level, +440.5 metres above sea level (m.a.s.l), since the planned excavation is assumed to be performed under normal conditions. At the downstream side of the dam the water level is assigned to +432.5 m.a.s.l. based upon stand pipe data from the site. The levels of the phreatic line within the dam body have been assumed as in Figure 1. Simulations have shown that it is not necessary with more accurate levels between the known ones at the upstream and downstream side.

Since the dam was constructed during the 1960s, all excess pore pressures are assumed to have dissipated. No excess pore pressures are expected to be built up during the excavation. Therefore no consolidation phases are simulated; the calculation phases are performed as drained phases for the excavation.

4.2 Section B

The advanced constitutive model Hardening soil is chosen for this section, since it is suitable for this application and material parameters are available.

The modelling of the planned berm is performed in five steps, as shown in Figure 5.

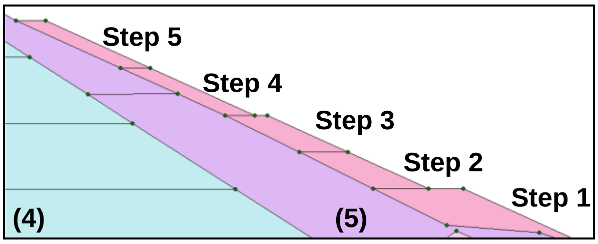


Figure 5 Construction steps for the new toe berm, material zones denoted according to Figure 2.

The level of the phreatic line, illustrated in Figure 2, for this section has been based upon flow computations performed in the programme GeoStudio SEEP/W by the consulting company ÅF. The levels are +440.5 m.a.s.l for the upstream side and approximately +400.0 m.a.s.l for downstream side.

No significant excess pore water pressures are expected to be built up during the berm construction, because of the high permeability of the adjacent material zones. The phases for the construction of the toe berm are modelled as drained phases.

4.3 Section C

The constitutive model Hardening soil is utilised for section C.

The excavation steps of the soil masses at the dam toe are shown in Figure 6.

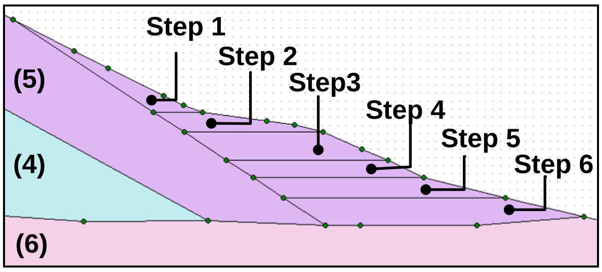


Figure 6 Excavation steps for the removal of the soil at the dam toe, material zones denoted according to Figure 3.

The levels of the phreatic line, seen in Figure 3, is also assigned based upon the results from flow computations in GeoStudio SEEP/W performed by ÅF. The upstream water level is at +440.5 m.a.s.l. and the downstream level is at approximately +400.0 m.a.s.l.

The phases for the excavation at the toe berm are modelled as drained phases.

5 RESULTS

Deformation and stability analyses have been performed in PLAXIS for all sections.

5.1 Section A

Values for the computed factors of safety and accumulated total deformations for each excavation phase are shown in Table 4; the failure surfaces corresponding to the factors of safety, FoS, are shown in Table 8.

Table 4 Section A, factors of safety and accumulated maximum total deformations.

Excavation phase	Factor of safety	Maximum deformation [mm]
Step 1	2.07	9.5
Step 2	1.67	22
Step 3	1.47	36
Step 4	1.37	47.5
Step 5	1.19	60

In Table 4 and Table 8, it is seen that the value of the factor of safety is decreasing for each excavation step. The computed values for the factors of safety gives that the trench slopes are still stable during the excavation phases. The smallest value of the factor of safety is 1.19, which can be considered as relatively low.

For the last excavation phase, step 5, the maximum total deformations in the excavation reached 60 mm. In Figure 7, for the last excavation step 5, the zone affected by the deformations is viewed by colour shadings. The position and direction of the maximum deformations is shown by the arrow. The smaller arrow indicates relative magnitude and direction of the deformation in a point of the right trench wall.

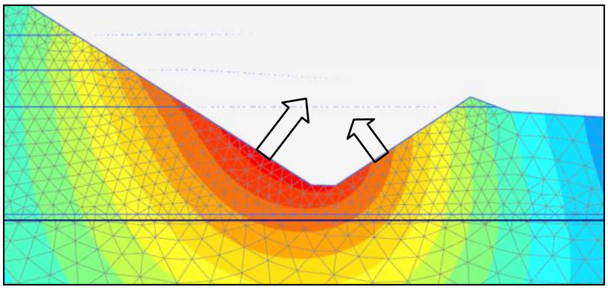


Figure 7 Section A, deformations in excavation step 5.

5.2 Section B

The results are found in Table 5; the failure surfaces are shown in Table 9.

Table 5 Section B, factors of safety and accumulated maximum total deformations.

Toe berm phase	Factor of safety	Maximum deformation [cm]
Step 1	1.36	6.1
Step 2	1.39	6.2
Step 3	1.42	7.9
Step 4	1.45	9.1
Step 5	1.54	10.6

The value of the factor of safety, as seen in Table 5 and Table 9 are increasing with every added step of the berm.

For the final computational phase, step 5, the deformations reached 10.6 cm. The extent of the maximum total deformations is shown by colour shadings in Figure 8; the arrows are indicating the direction of the largest deformations.

When considering the degradation of the rockfill material, the maximum total deformations reached a value of 24 cm in the last phase, step 5. The distribution of the deformations is as shown in Figure 8.

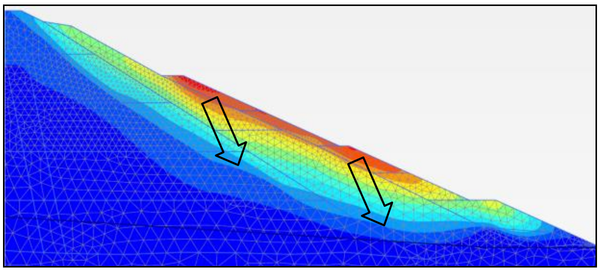


Figure 8 Section B, maximum deformations in part 5.

5.3 Section C

The results from the computations are found in Table 6.

Table 6 Section C, factors of safety and accumulated maximum total deformations.

Excavation phase	Factor of safety	Maximum deformation [mm]
Step 1	1.28	4
Step 2	1.26	5
Step 3	1.25	6
Step 4	1.24	7
Step 5	1.23	10
Step 6	1.20	12

The failure surface for the final excavation phase, step 6, is found in Table 10. This is representable for all excavation phases, step 1-6. The lowest factor of safety is relatively low, with a value of 1.20.

As seen in Table 6, the maximum total deformation reached 12 mm for step 6. The extent of the maximum total deformations at the toe of the dam is shown in Figure 9 by shadings; the main direction is indicated by the arrow.

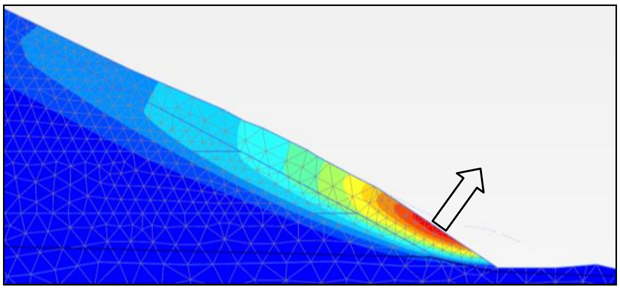


Figure 9 Section C, deformations in excavation step 5.

6 VERIFICATION COMPUTATIONS

In order to determine if the results from the PLAXIS computations are reliable and trustworthy, complementary analyses have been performed with the software GeoStudio SIGMA/W and SLOPE/W, see GeoStudio (2009) and GeoStudio (2008).

6.1 Stability

Verification of the slope stability computations have been performed in GeoStudio SLOPE/W with the limit equilibrium method Morgenstern-Price. The most critical slip surface and associated slip surface is searched for at the downstream side of the dam. For sections A and B, the factors of safety and failure surfaces are shown Tables 8-9.

Due to the fact that all slip surfaces for section C are very similar in location from the finite element analyses, only one limit equilibrium analysis is performed in the verification part for the sixth and final excavation step. Since the slip surface from PLAXIS is not circular, the most critical slip surface in SLOPE/W is specified as in PLAXIS. The factor of safety and corresponding failure surface is presented in Table 10.

6.2 Deformations

In order to verify the computations of deformation from PLAXIS, the programme GeoStudio SIGMA/W is used. The constitutive model Mohr Coulomb has been utilised during the verification computations for section A. For sections B and C, the constitutive model Nonlinear elastic Hyperbolic is chosen in SIGMA/W, since both are based upon the nonlinear formulation by Duncan and Chang (1970). The modelling is performed under the same conditions as in PLAXIS. The maximum deformations are presented in Table 7; the distributions of the deformations are shown in Figures 10-12.

Table 7 Verification of the deformations.

Section	Maximum deformation
A	42 mm
B	13.4 cm
C	15 mm

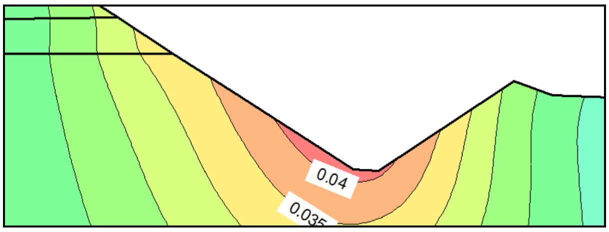


Figure 10 Section A, deformations from SIGMA/W.

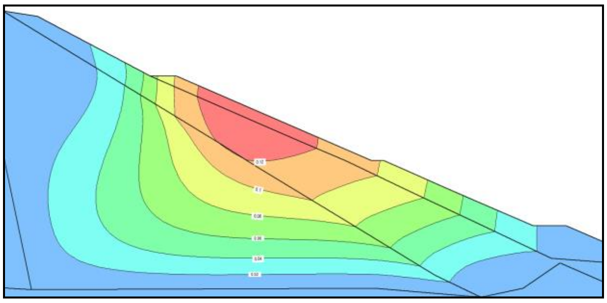


Figure 11 Section B, deformations from SIGMA/W.

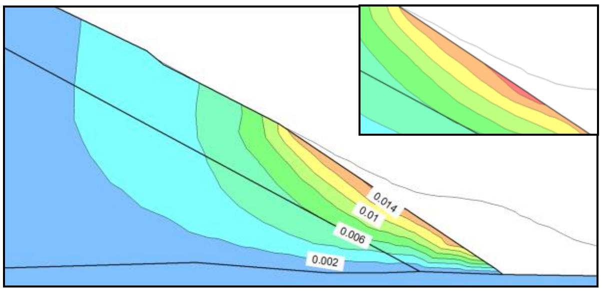


Figure 12 Section C, deformations from SIGMA/W.

The obtained results from the verification computations are well conforming to the results from PLAXIS. Therefore the conclusion is drawn that the results from the numerical analyses in PLAXIS are reliable.

7 DISCUSSION

Even though all the planned measures have indicated stable conditions, meaning a factor of safety above 1.0 during the simulations, it can not be determined if that represents a sufficient stability. This is up to the dam owner to decide. However, in order to assure a safe work environment, deformation monitoring can be conducted during the work in section A, B and C. Alarm values used to determine if the deformations are within reasonable limits, can be chosen prior to the monitoring. This can, for example, be done by inspecting trends of the relationship between computed factors of safety and deformations. This kind of monitoring is intended for the dam in the case study.

Monitoring of the deformations is valuable, since the results have shown relatively low factors of safety for sections A and C. By using the results from the finite element analyses for establishing alarm values, a contribution is made to improve the dam safety. More understanding is gained about the expected dam behaviour. This shows some advantages of using a numerical analysis instead of a more traditional approach as a limit equilibrium method; where only a factor of safety is produced and not deformations.

For section C, the excavation at the toe is limited in the transversal direction. This implies that modelling the problem as a plane strain case in two dimensions is not very appropriate. In a three dimensional case, soil masses at the sides cause resisting forces. Smaller deformations can therefore be expected from such a case compared to a two dimensional case. Since the deformations are already relatively small in plane strain, modelling in three dimensions was not considered as necessary.

8 CONCLUSIONS

Based upon the results, the following conclusions can be drawn:

Section A: The deformations are of largest magnitude in the direct vicinity of the trench. The maximum side of the deformations is 60 mm. The direction of the movements is mostly upward.

Section B: Considering a case where no degradation of the rockfill material is occurring, the deformations will take a maximum size of almost 11 cm in the top of the new berm. The direction of the deformations is mainly awry downwards. For a case including the rockfill degradation, the maximum value of the computed deformations is 24 cm.

Section C: The maximum deformations caused by the excavation at the bottom of the toe berm reaches 12 mm. The directions of the deformations are mostly perpendicularly outwards from the excavation face.

The planned measures can be performed according to the suggestions projected. No deformations of such size that instability is occurring have been shown during the simulations.

The finite element method is a useful tool for this kind of evaluation; considering the physically adequate base of the method, the possibility of determining both deformations and factors of safety as well as the practical application of the results for establishing alarm values.

9 ACKNOWLEDGEMENTS

The author would like to express sincere thanks to Vattenfall vattenkraft AB for giving opportunity to carry out the presented study. Consultancy company ÅF AB, is to be acknowledged for providing the case and all relevant information regarding the dam.

The research presented has been carried out within the environment of "Swedish Hydropower Centre - SVC" at LTU. The support from the SVC environment is highly appreciated and acknowledged for.

10 REFERENCES

Brinkgreve, R.B.J., Engin, E. & Swolfs, W.M. (2014). PLAXIS 2014. Delft, the Netherlands: Plaxis bv.

Bowles, J.E. (1988). Foundation Analysis and Design. Berkshire, England: MacGraw-Hill.

Duncan, J.M. & Chang, C.Y., 1970. Nonlinear analysis of stress and strain in soils. Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 96, no. SM5, pp. 1629-1654.

GeoStudio (2008). Stability Modeling with SLOPE/W 2007 Version. Calgary, Canada: GEO-SLOPE International Ltd.

GeoStudio (2009). Stress-Deformation Modeling with SIGMA/W 2007. Calgary, Canada: GEO-SLOPE International Ltd.

Potts, D.M. & Zdravković, L. (1999). Finite element analysis in geotechnical engineering: theory. London, England: Thomas Telford Books.

Muir-Wood, D. (1990). Soil Behaviour and Critical State Soil Mechanics. New York, USA: Cambridge University Press.

Svensk energi. (2002) RIDAS: kraftföretagens riktlinjer för dammsäkerhet. Stockholm, Sweden: Svensk energi. (In Swedish)

Vahdati, P. (2014). Identification of Soil Parameters in an Embankment Dam by Mathematical Optimization. Licentiate thesis. Luleå, Sweden: Graphic Production.

Yu, H. (2006) Plasticity and Geotechnics. Nottingham, England: Springer.

Table 8 Section A, failure surfaces from PLAXIS and SLOPE/W.

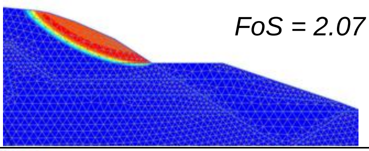
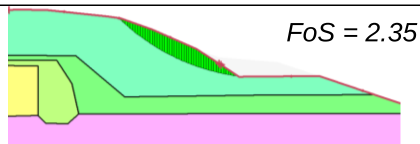
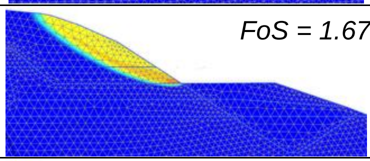
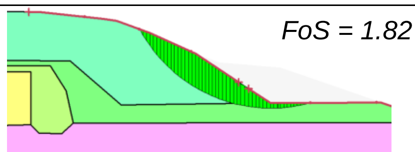
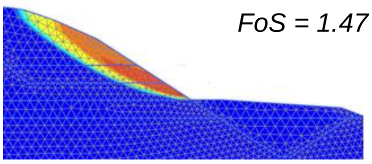
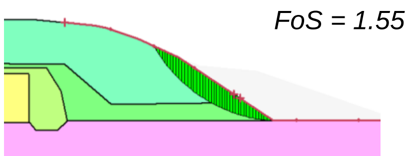
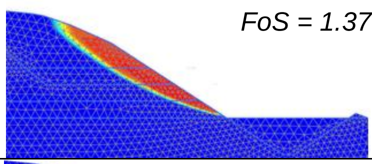
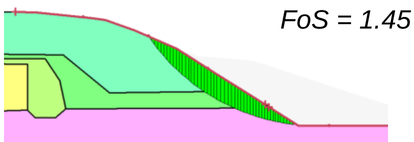
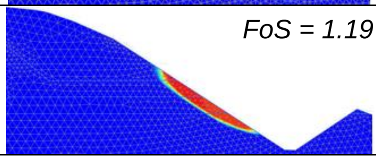
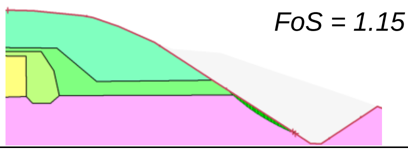
Excavation phase	PLAXIS	SLOPE/W
1		
2		
3		
4		
5		

Table 9 Section B, failure surfaces from PLAXIS and SLOPE/W.

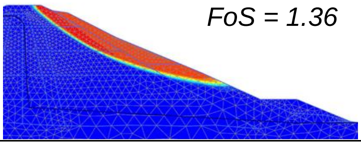
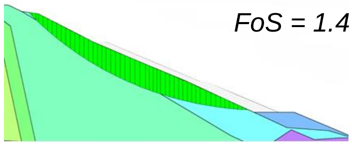
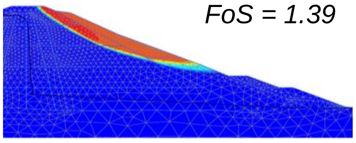
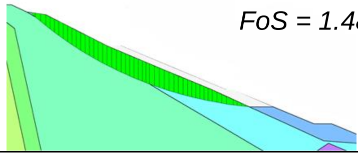
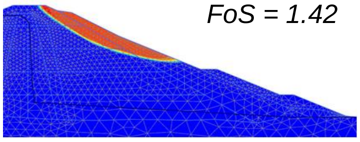
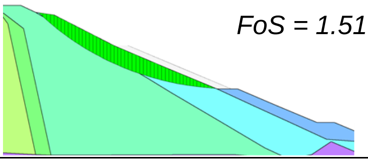
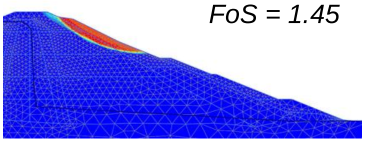
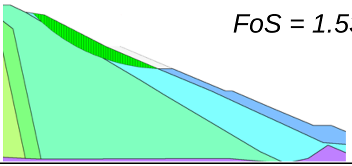
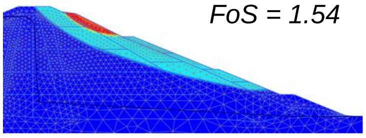
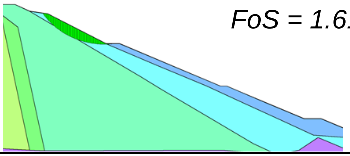
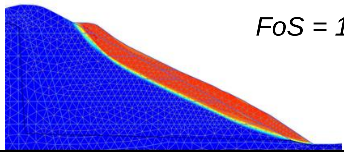
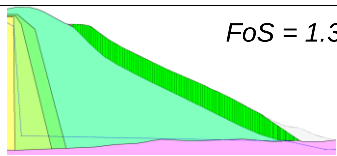
Toe berm phase	PLAXIS	SLOPE/W
1	 *FoS = 1.36*	 *FoS = 1.47*
2	 *FoS = 1.39*	 *FoS = 1.48*
3	 *FoS = 1.42*	 *FoS = 1.51*
4	 *FoS = 1.45*	 *FoS = 1.53*
5	 *FoS = 1.54*	 *FoS = 1.61*

Table 10 Section C, failure surfaces from PLAXIS and SLOPE/W.

Excavation phase	PLAXIS	SLOPE/W
6	 *FoS = 1.20*	 *FoS = 1.31*