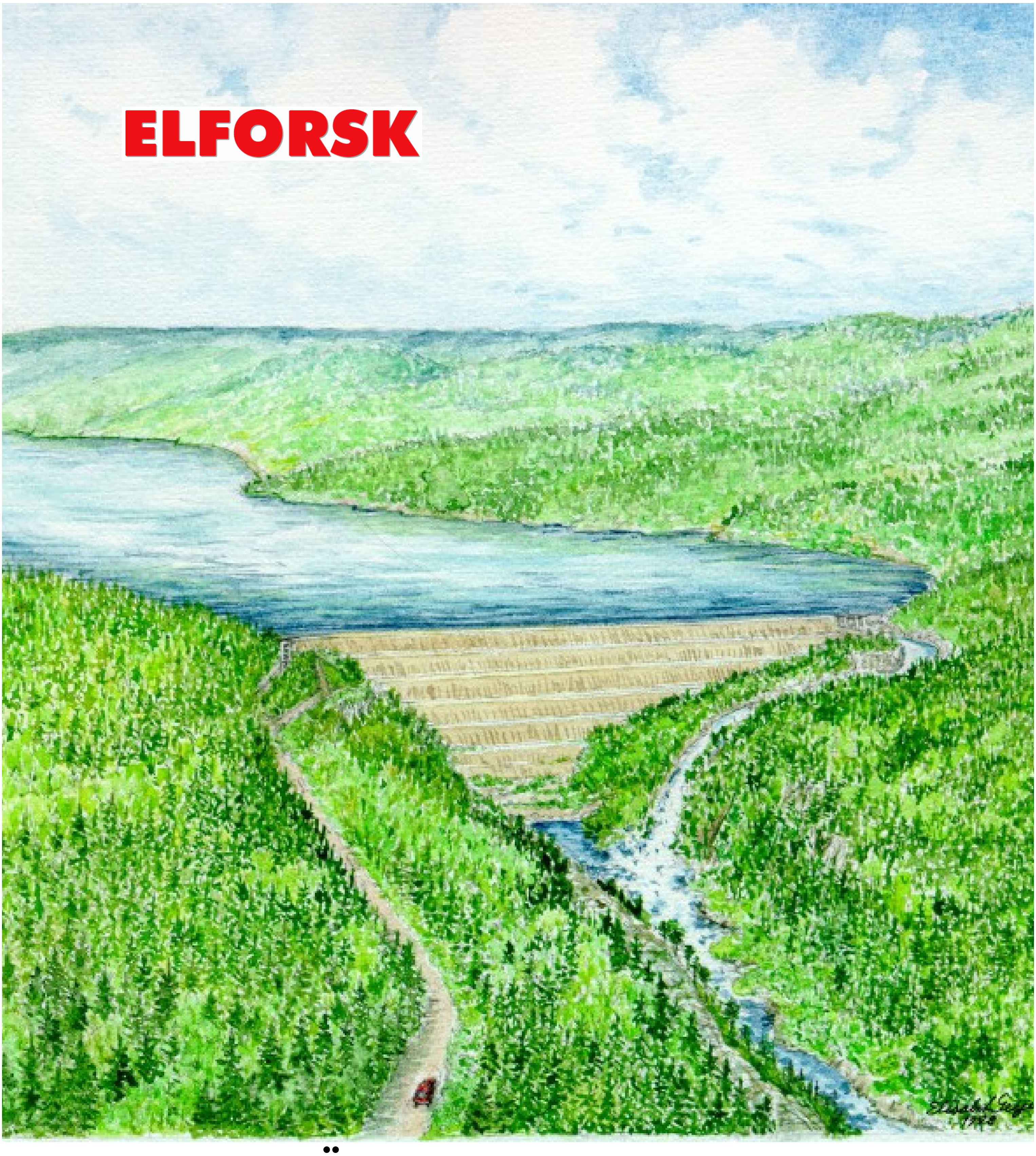

DAMMSÄKERHET

Dam-Break Project in Norway, Slope Stability Analysis on Rockfill and Gravel Test Dams in correlation with test results

Rapport 05:13

Dam-Break Project in Norway, Slope Stability Analysis on Rockfill and Gravel Test Dams in correlation with test results

Elforsk rapport 05:13

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Förord

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Denna rapport är ett delresultat inom Elforsk ramprogram Dammsäkerhet.

Kraftindustrin har traditionellt satsat avsevärda resurser på forsknings och utvecklingsfrågor inom dammsäkerhetsområdet, vilket har varit en förutsättning för den framgångsrika utvecklingen av vattenkraften som energikälla i Sverige.

Målen för programmen är att långsiktigt stödja branschens policy, dvs att:

- Sannolikheten för dammbrott där människoliv kan vara hotade skall hållas på en så låg nivå att detta hot såvitt möjligt elimineras.
- Konsekvenserna i händelse av dammbrott skall genom god planering såvitt möjligt reduceras.
- Dammsäkerheten skall hållas på en god internationell nivå.

Prioriterade områden är Teknisk säkerhet, Operativ säkerhet och beredskap samt Riskanalys.

Ramprogrammet har en styrgrupp bestående av: Jonas Birkedahl – FORTUM, Malte Cederström – Vattenfall Vattenkraft, Anders Isander – Sydkraft Vattenkraft, Lennart Markland – Vattenregleringsföretagen, Urban Norstedt – Vattenfall Vattenkraft, Gunnar Sjödin – Vattenregleringsföretagen samt Lars Hammar – Elforsk.

Lars Hammar
Elforsk AB

Sammanfattning

Med början år 2001 t.o.m. år 2003 genomfördes försök på testdammar i Norge nära staden Mo I Rana. Testdammarna utgjordes av ett flertal 5-6 m höga dammar av varierande utformning från homogena grus- och sprängstensdammar till zonerade dammar med vertikal tätkärna och omgivande stödfyllning. Målsättningen med de norska dammbrottsförsöken är att förbättra förståelsen om dammars stabilitet och kasta ljus över dammbrottets olika mekanismer. I föreliggande rapport analyseras släntstabiliteten på testdammarna från år 2001 och 2002 i jämförelse med testresultatet.

2001-års sprängstensdamm (testdammen ”1-01”) gick aldrig till brott trots över- och genomströmning på totalt ca 3 m^3/s. Försöket överensstämmer med stabilitetsanalysen som visar stabila förhållanden med säkerhetsfaktor väl över 1 under samtidigt högt portryck när dammen är vattenfylld och genomströmmad, vilket var förväntat med tanke på fyllningsmaterialet.

Stabilitetsanalysen undersökte även den stabilitetshöjande inverkan en tåförstärkning upp till halva dammhöjden kan ha på en sprängstensdamm; säkerhetsfaktorn för djupgående glidytor ökar med i storleksordningen 50-60 % jämfört med en oförstärkt slänt.

2002-års grusdamm (testdammen ”2C-02”) gick till brott i form av en bakåtgripande erosion från dammtån mot dammkrönet. När erosionen slutligen nådde magasinet var dammbrottet fullständigt.

Trots att testdammen av grus var belastad med hög magasinsvattenyta och utbildat källsprång i nedströmsslänten initierades ej erosion förrän övertoppningen påbörjades. Testdammen föreföll därför ha en oförväntad förhöjd hållfasthet med tanke på grusmaterialet. Den höjda hållfastheten kan ha berott på en rad faktorer, som t.ex. att portrycket inte hunnit ställa in sig innan ny belastning påfördes (vilket ger intryck av att dammen är utsatt för större belastning än vad den egentligen är) eller att grusmaterialet kan ha haft oförväntade hållfasthetshöjande egenskaper, som t.ex. bidrag från kohesion. Normalt är dock grus ett icke kohesivt material.

För att erhålla korrelerande resultat mellan testdammen och stabilitetsanalysen erfordrades att ökad hållfasthet modellerades, och en trolig orsak till hållfasthetshöjningen bedömdes vara mobiliserat negativt portryck över portryckslinjen, som förenklat uttryckt bildar ett hållfasthetshöjande ”sug” i materialet inom stighöjdens räckvidd. Utan inslag av förhöjd hållfasthet så visar analysen på mindre stabilitetsbrott som ej inträffade under själva testet.

Analysen visade för övrigt att en stabiliserande tåförstärkning kan vara påtagligt effektiv på en grusdamm. Om grusdammen antas helt vattenmättad och förstärks med en tåbank upp till krönet så kan det ge en fördubbling av säkerhetsfaktorn för djupgående glidytor.

Summary

Beginning in autumn 2001 through 2003 a number of large-scale field test were carried out in Norway at a test site near the town Mo I Rana. A number of test dams of various designs from homogenous dams of gravel and rockfill to zoned dams with central cores and support fills. The large-scale tests analyzed in this report comprise of the 2001 rockfill test dam and the 2002 gravel test dam which eventually was brought to failure.

The ***Rockfill test dam 1-01*** never failed although subjected to overtopping and through-flow in total about 3 m^3/s. The actual test results correlates well with the stability analysis that indicates stable conditions though high pore pressure is acting on the downstream shoulder. The safety factor of a “deep-going” slip failure in the rockfill dam, if occurring it would probably have resulted in breaching, is far above stable conditions as would be expected considering the fill material.

The stability analysis also studied the influence a stabilizing toe berm has on the stability. If the rockfill test dam is stabilized with a toe berm of approximately half the dam height it improves the stability (safety factor) with roughly 50-60 percent compared to the unsupported case.

The estimated rockfill strength model is a nonlinear Mohr-Coulomb failure envelope with friction angle 57 degrees at the surface gradually decreasing to 47 degrees at the base of the dam. The nonlinearity is based on Leps (1970) and its Norwegian supplementation (sub-project 1: “Shear Strength of Rockfill and Stability of Dam Slopes”).

The ***Gravel test dam 2C-02*** failed through ”head cut” advancement; the erosion works itself backwards to the crest little by little until it reaches the reservoir and a breach is completed. The erosion at the toe did not initiate prior to the raising of the reservoir enough to overtop the crest allowing water to flow down the slope.

The actual test does not correlate well at this point with the stability analysis, which indicates instability at lower reservoir levels than the level that overtops the crest. According to the analysis “superficial” shallow slides form just below the discharge point of the leakage. Such small slip failures at the toe below the exit leakage point did not take place at this point although it could be expected due to the mobilizing hydrostatic uplift pressure in the gravel shoulder especially below the discharge. Erosion first started when overtopping occurred and water flowed freely down the slope.

Another inconsistency between the analysis and actual test is that the stability analysis shows slip surfaces below stable conditions for “deep-going” failures with the scarp at the crest going down to the toe when overtopped. As described above the erosion process started with small erosion at the toe working itself backwards, not a large failure as indicated in the analysis.

The unexpected stability of the gravel test dam could be due to the pore pressure distribution in the dam had not yet had time to settle and the seepage line to position itself before the reservoir was increased applying a new case of load, indicating a tougher load case than actually applied. Had the seepage situation been allowed to settle erosion might have progressed according to the analysis. Another explanation to the supposedly increased shear strength of the fill can be that the gravel fill used in the dam had some cohesive strength although assumed to be non-cohesive. Alternatively the increased strength can be due to negative pore pressure creating added strength by suction.

To accommodate to added strength in the fill the influence of negative pore pressure above the seepage line was considered in the analysis. Negative pore pressure can create a (matric) suction within the capillary reach increasing the shear strength in the unsaturated zone. By this action more realistic results can be obtained correlating with the actual test results.

A fully saturated downstream side of the gravel could be expected after overtopping some time allowing the pore pressure to position. “Deep-going” slip failures are unstable under fully submerged conditions. Assuming that the gravel test dam is not fully submerged, subjected to strength enhancing suction, the analysis show stable conditions in good correlation with the actual test.

In conclusion regarding the analysis on the gravel test dam is as the submersion is continually increasing; closing in on complete saturation, the stability of “deep-going” slip surfaces is gradually decreasing towards unstable conditions and failure.

If the gravel test dam is stabilized with a toe berm all the way up to the crest the stability (safety factor) improves with roughly 30-50 percent. 110 percent if the dam is overtopped and assumed fully saturated.

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1 Introduction

Beginning in autumn 2001 through 2003 a number of large-scale field test were carried out in Norway at a test site in proximity to the town Mo I Rana located in the Nordland County and the Hemnes municipality. The main purpose of the tests has been to provide better knowledge about stability of embankment dams of different design and the breaching process. The large-scale test dams were constructed downstream of the dam Røssvassdammen making it possible to control the inflow to the test dams by regulating the three flood gates of Røssvassdammen as the figure below indicates.

The test dams were embankment dams made up by various materials in order to make up different scenarios regarding stability and breaching. The constituents of the embankment dams analyzed in this report are rockfill and gravel, making up a typical filling material of Scandinavian embankment dam conditions.

The large-scale field tests are a part of the project “Stability and Failure Mechanisms of Dams”.

Figure 1.1 Location of the test site.

The purpose of this report is to provide better knowledge about stability issues concerning embankment dams composed of rockfill and gravel subjected to through-flow and/or overtopping. The stability has been analyzed for two homogeneous embankment test dams built 2001 and 2002 of rockfill and gravel separately to see if correlations with the actual test results can be shown. Furthermore an evaluation has been made on the effect on stability a stabilizing toe berm can have when placed on the downstream slope of an embankment dam.

The slope stability analysis is carried out in the commercial code SLOPE/W (http://www.geo-slope.com).

2 Background

A typical cross-section and longitudinal section of the test site is shown in figure 2.1 and figure 2.2. The test dam foundation was rock partly covered by gravel and stones. The rock was levelled and the permeable material was removed in order to uniform the foundation.

The homogenous rockfill test dam (1-01) never failed and the test subsequently crossed over to 2-01 by adding a stabilizing toe berm to the rockfill test dam.

The gravel test dam (2C-2) failed when overtopped some time. The prior test to the “2C-2” in year 2002, namely “2A-02 and “2B-02”, involved the same gravel test dam as “2C-02” but arranged with a toe berm.

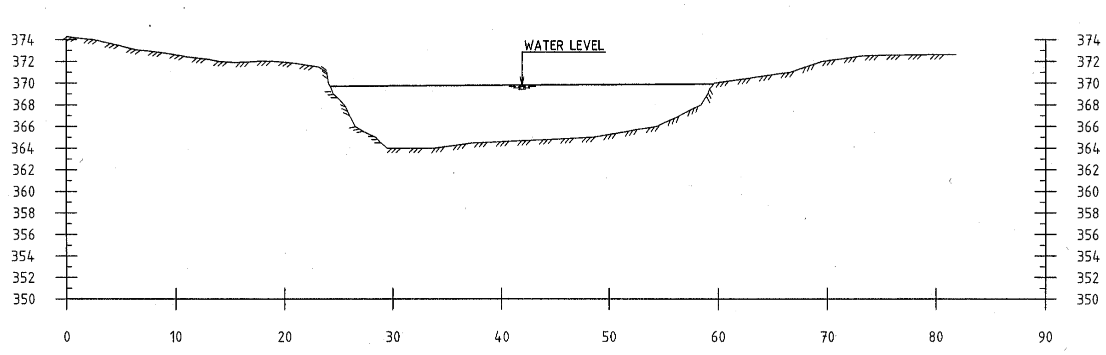

Figure 2.1 Location of the test site.

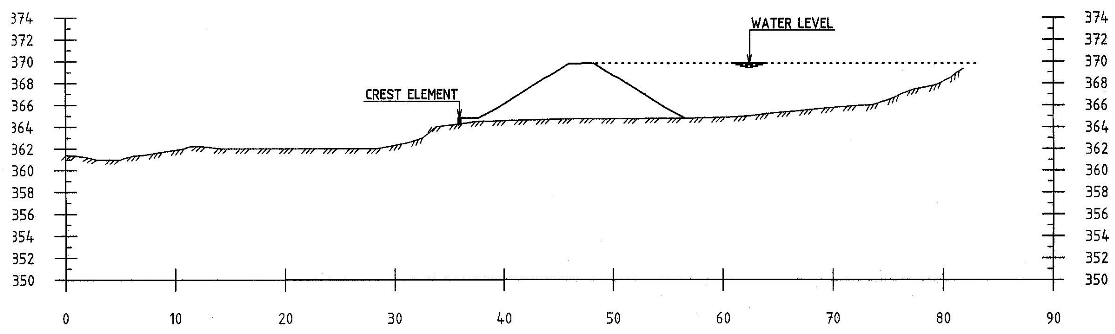

Figure 2.2 Longitudinal section of the test site.

3 Analysis Conditions

3.1 General

The in general used procedure in slope stability analysis is to investigate the relationship between forces and moments causing instability compared to such that resist instability. The general shape of the sliding surface is of a circular form but it can be a more complex shape like a composite surface of sliding, which is a noncircular form or wedged shaped. The shape of the slip surface is normally depending on the stratigraphy of the stability problem.

The dam structures of the test dams are fairly simple; homogenous dams made up by a single component (i.e. rock fill in test dam 1-01 and gravel in 2C-02). In the analysis a circular slip surface has therefore been assumed. The following stability calculations are in a 2D-cross section of the highest part of the test dams.

3.2 Strength parameters

The shear strength of soil materials is preferably determined from laboratory tests on a specimen. In this case no laboratory work relating to shear strength has been done so it has been necessary to estimate such parameters of the fill material.

By and large, the components of the two test dams analyzed in this report are granular non-cohesive materials. This is of course the case of the rockfill dam with its rockfill material. But one way of explaining the breach process of the gravel test dam, during which steep, stable slopes developed, is to assume that the finer graded parts of the gravel material must have had some cohesive tendencies, which increased its strength. Otherwise the developed steep slopes in the gravel would never have kept stable but crumbled down. Alternatively (as have been considered in this report) the effect of negative pore water pressure (matric suction) above the zero pressure line can have increased the shear strength of the gravely material.

Assuming a nonlinear failure plane (strength envelope) by a Mohr-Coulomb failure envelope has been the best way to go about in estimating the shear strength characteristics in the rock fill test dam 1-01 and the gravel test dam 2C-02. Each estimated strength parameter of the materials in the two dams is further described in the analysis part.

3.3 Analysis Method and Software

The software product SLOPE/W of GEO-SLOPE was used in the stability analysis of the test dams. SLOPE/W is software to calculate the factor of safety of slip surfaces in slopes using limit equilibrium theory.

The method of analysis is the Morgenstern-Price method, which satisfies both force and moment equilibrium and uses a selected interslice force function, in this case the Half-sine function. Interslice shear forces are required to calculate the normal force at the base of each slice in the slip surface. The Morgenstern-Price method is considered to be an adequately conservative method for stability calculations in this particular case.

To explain the inconsistencies in stability in the analysis in comparison to the actual test of the gravel test dam, the influence of negative pore water pressure has been considered. The rate of shear strength increase with change of negative pore water pressure, which usually is the case of materials with a notable capillary zone. In order to pay regard to this in SLOPE/W it is necessary to assume an unsaturated ϕ_b-value, which is an angle used to accommodate for unsaturated soil conditions.

3.4 Acting external and internal water load

The following stability analysis has been made with regards to different reservoir levels. The case of load has eventually been brought up to an overtopping of the test dams in order to develop a dam breach. What reservoir level used in the following calculations is clearly presented for each case.

The two test dams phreatic line (line of seepage) has been determined from the measured pore pressure in the installed sensors in the body of the dam. In order to get the correct value of the pressure head it is usually necessary to reduce the pore pressure measured in the sensors with the velocity head, but in this case the velocity of the seepage was low and thus resulting in a negligibly low velocity head.

4 Test Dam 1-01 – Homogenous Rockfill Dam

4.1 General

The as-built dimensions of the test dam 1-01 are 6.2 m height, 2.92 m crest width and 1V:1.6H slope. The design is roughly according to the standard cross-section in figure 4.1 and the plan in figure 4.2. The elevation of the dam toe was leveled to +364.81m.

The 1-01 test subsequently crossed over to 2-01 by adding a toe berm of rockfill.

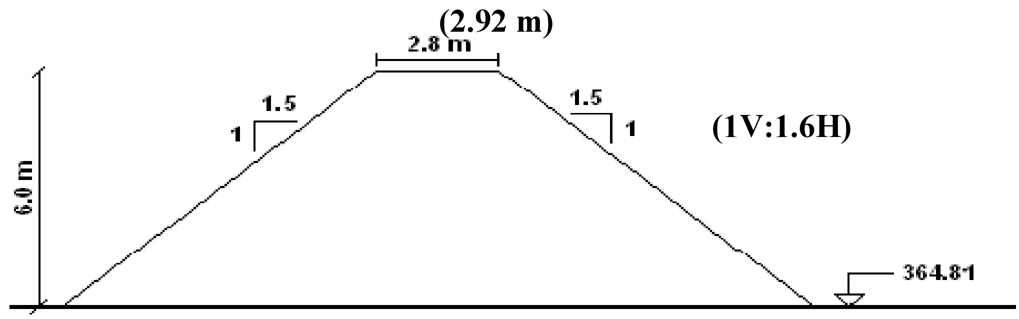

Figure 4.1 Standard cross-section of the 1-01 rockfill dam with initially given dimensions. The crest width, slope inclination and height were later on revised according to numbers in brackets.

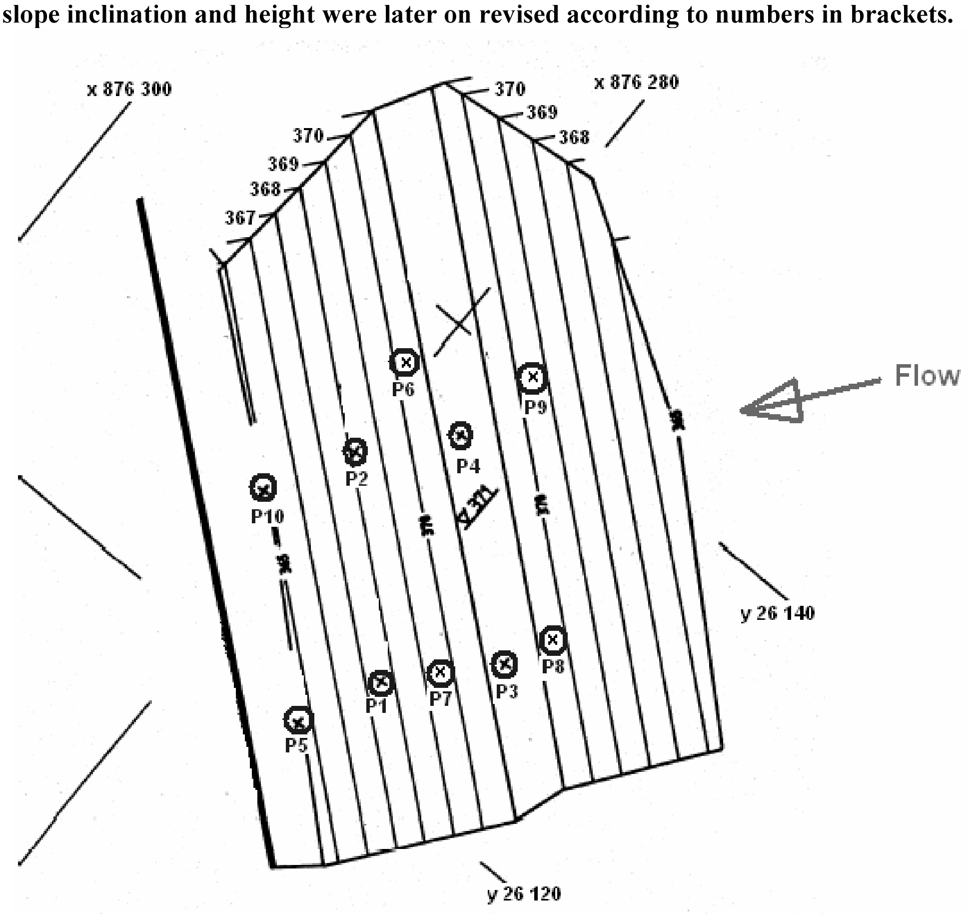

Figure 4.2 Plan view of the Rockfill dam with location of the pore pressure sensors.

4.2 Composition of Fill Material

The fill material in the dam is a narrowly graded rockfill (gravelly stone material) with a grain size distribution varying in size up to 200 mm. Only 10 % of the material is smaller than 30 mm according to the sieving curve. The rock material was taken from the excavation blasting of the tunnels of the upstream hydropower station during its construction.

The rockfill material in test dam 1-01 was not subjected to any particular laboratory work apart from screening as was the rockfill material in subsequent test dams 1-2003 to 3-2003 the following year. It is in the following calculation assumed that the rockfill used in test dam 1-01 is from the same supply as test dams of 2003 used for the downstream shoulder. The rockfill is further described with the parameters in table 4.1, which defines the bulk unit weight to 21.7 kN/m^3 and the fully saturated unit weight can be derived to 26.6 kN/m^3.

Table 4.1 Rockfill parameters and characteristics.

Rock fill	Parameter		
Downstream	Density Bulk	kN/m^3	21.73
	Density Dry	ton/m^3	21.2
	Density Grain	ton/m^3	27.7
	Porosity		0.235
	Cohesion	kPa	0
	Friction angle	tg φ	0.9 (≈42°)

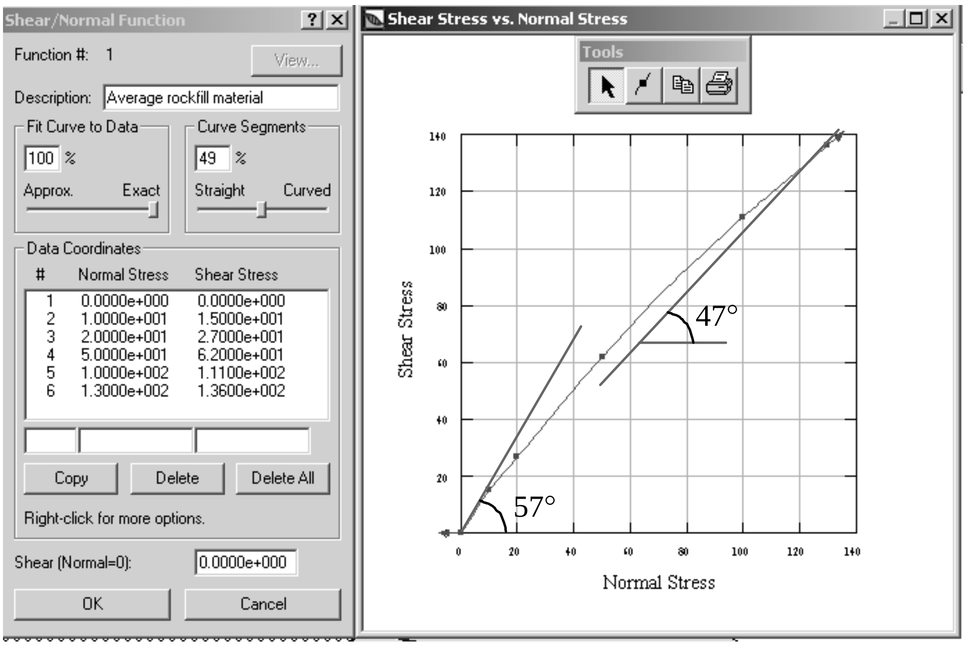

Figure 4.3 Estimated nonlinear shear/normal effective stress (kPa) relationship of the rockfill.

Leps ([5]) found that rockfill material show nonlinearity in the Mohr-Coulomb failure envelope. This nonlinearity is especially distinct at relatively low normal stresses (close to the surface) where the angle of shearing resistance decreases significantly. There is still nonlinearity at higher normal stresses but it is not as clear as at low depths. The rockfill shear strength of the material in test dam 1-01 is related to the effective normal stress on the failure plane in estimation to the Norwegian development and supplementation (see [1]) to Leps ([5]).

The estimated shear/normal stress relationship for the rockfill material is shown in figure 4.3. This function represents a decrease in friction angle from 57° right at the surface to about 47° at the bottom of the dam, approximately 6 m below the surface.

4.3 Pore pressure test readings

The dam was equipped with a total of 10 pore pressure sensors (P1-P10), see location in figure 4.2. On the upstream and downstream side of the dam there were gauges arranged for monitoring of water levels during the test.

Table 4.2 Pore pressure readings in sensor P1-P10 in test dam 1-01.

Date (dd.mm.yy)	Time (hh:mm)	Water level (m.a.s.l)	q ($m^3/s/m$)	P1 (m.a.s.l)	P2 (m.a.s.l)	P3 (m.a.s.l)	P4 (m.a.s.l)	P5 (m.a.s.l)	P6 (m.a.s.l)
19-10-01	13:01	368,88	0,091	365,97	366.43	367,68	367.81		368.16
31-10-01	16:34	370,00	0,112	366,32	366,80	367,92	368,29	366,00	368,47
11-11-01	11:15	370,15	0,115	364,89	365,09	364,96	365,11		367,43
11-11-01	12:36	370,93	0,188	365,06	365,24	365,33	365,49		367,59

Table 4.2 shows the measured total head (energy line) in each pore pressure sensor in the dam, displaying the head loss between the different points. The flow q in table 4.2 show that the discharge during the test varied up to 0.2 m^3/s,m and in gauge no 5 a total discharge in excess of 3 m^3/s was measured.

A rough estimate shows that these flows result in a velocity head of less than 1 mm at a discharge of 0.2 m^3/s,m. The velocity head is thus negligibly low and its influence will not be accounted for in the following calculations.

4.4 Analysis

Figure 4.4 shows the analysis model cross-section for the rockfill test dam during test day 19-10-01, time 13.01.

The reservoir level was set to +368.88m, roughly 1.9 m below the dam crest. The phreatic line through the dam has been estimated with basis on the head measured in pressure sensor P4, P6 and P2 (in May 2004 the measured head was revised but with negligible impact on the shape of the seepage line). This establishes a line of seepage in the dam. The elevation of discharge was measured during the test to about 1.5 m above the toe.

No breach was developed in this test, which correlates with the analysis result. The analysis show that superficial slides (thin failure plane, barely visible with a potentially small impact on the slope) have a factor of safety of approximately SF = 1,2 which is just over stable conditions (SF = 1) and these occur just below the seepage face as would be expected due to the effect of the pore pressure uplift.

Figure 4.5 shows a slightly larger slip surface, developed at the seepage point, with SF = 1,45 (still one of slip surfaces with the lowest "safety").

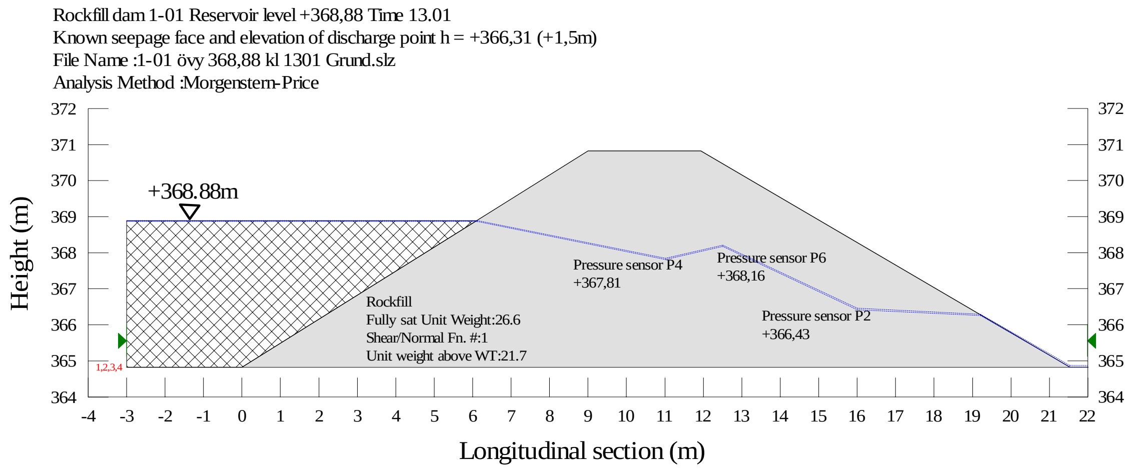

Figure 4.4 Rockfill test dam 1-01 model.

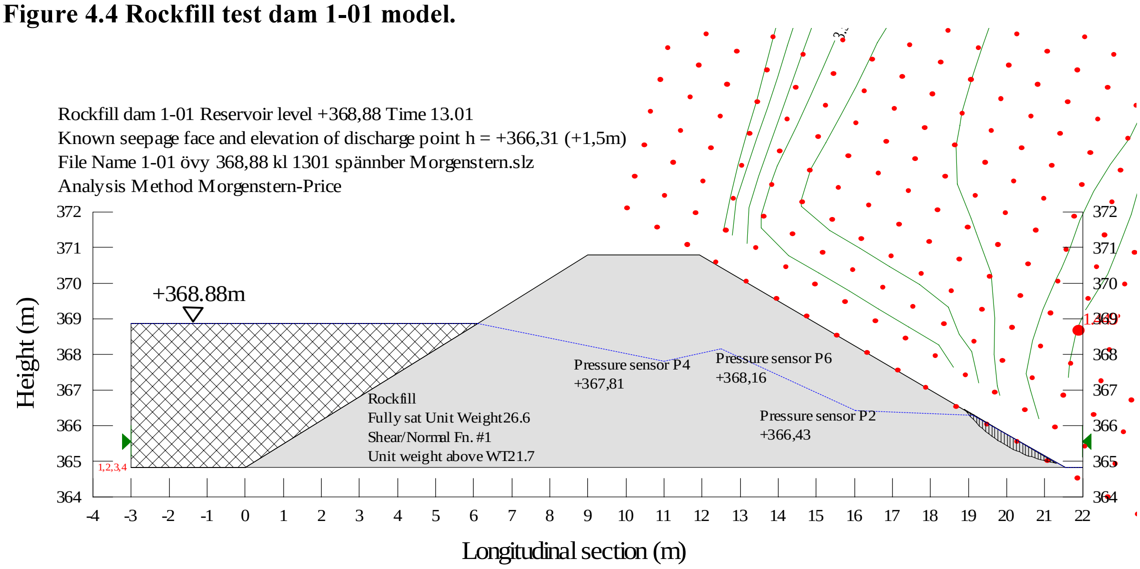

Figure 4.5 Superficial slide just below the discharge point of the slope. F = 1,449.

The factor of safety for a deep slip surface (see figure 4.6) with the scarp starting at the top of the slope and slip surface going down to the toe is SF = 2.145, which is well over stable conditions. This correlates well with actual conditions as a breach never initiated.

The test continued (crossing over to 2-01) with placement of a toe berm comprised of a transition layer of uniformly graded stone (100-200 mm) with a top layer of stone 200-500 mm. The berm was 3.4 m broad and 2.8 m high.

Under assumption that the reservoir level still is set at +368.88m and that such a toe berm will drain the upstream layer and lower the pore pressure at the interface, it will improve the factor of safety (increase the stability of the slope) of thin superficial slides, that without the berm had a factor of safety just over 1,2, with in excess of 100 % (SF = 2,4). The factor of safety for a deep slip surface with the support of a berm increases from SF = 2.145 to 3.23 (50 % improvement), see figure 4.7.

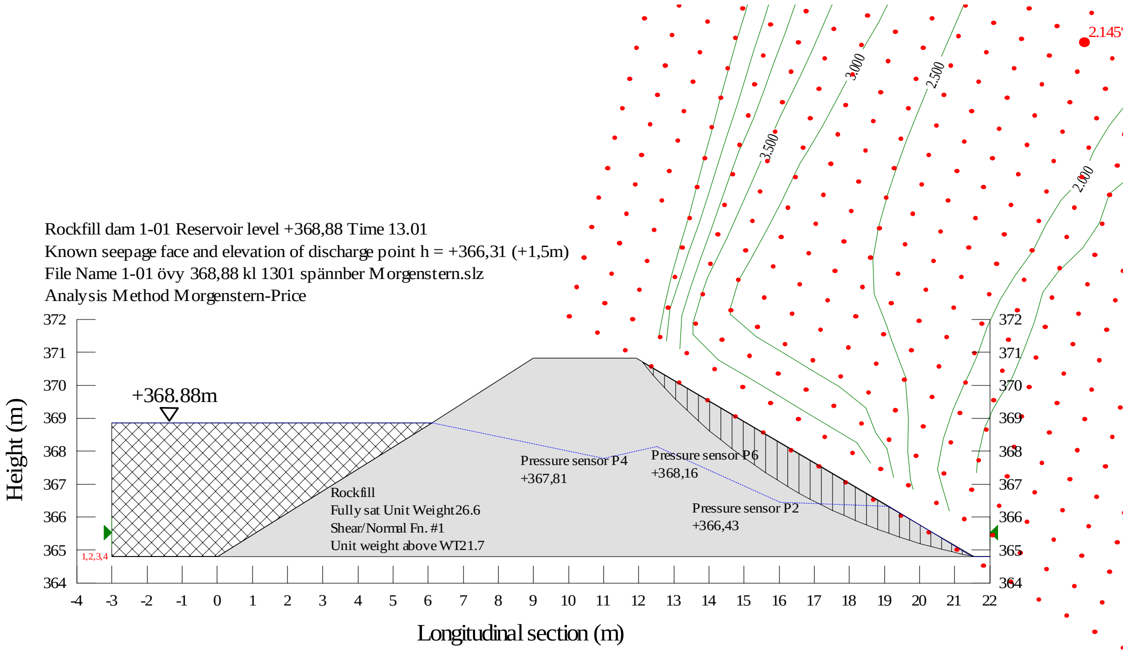

Figure 4.6 Deep slip surface from the top to the toe of the slope. F = 2.145.

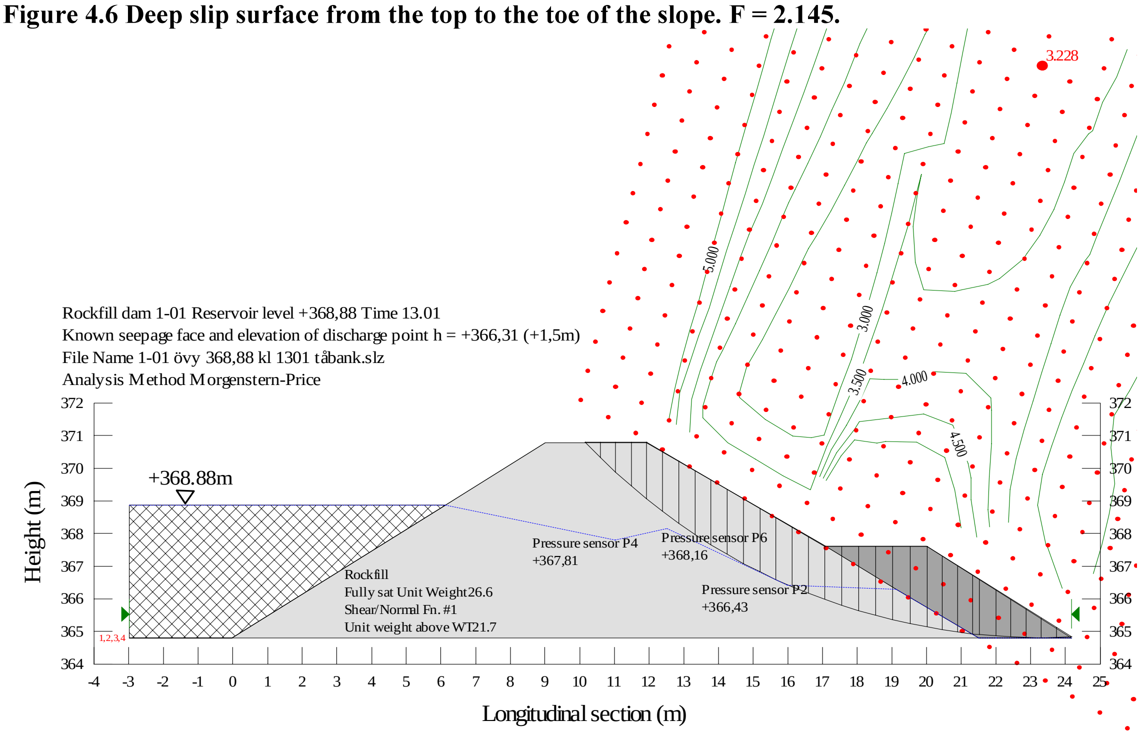

Figure 4.7 Deep slip surface with support of a toe berm. F = 3.23.

During the continuation of the test the upstream level was increased to +370.00m (Oct 31st 2001), roughly 0.8 m below the crest. According to photos taken this resulted in a heightening of the discharge face on the downstream slope, which lead to a seepage face above the berm, depicted by figure 4.8, with another point of discharge approximately at half of the berm height. This does not agree well with the pressure sensors, which measured lower head (displayed with X in figure 4.8). An explanation to this can be that the material had not had time to completely saturate during the increase in reservoir level, showing lower pressure or simply a measuring error. With regards to these circumstances the phreatic line in the dam has been estimated with basis on the photos taken and the pressure sensors are disregarded.

Thin slides, developed on the slope of the berm just below the seepage point, has a safety just over stable conditions, SF = 1,0 (such a slip surface on an unprotected slope that allows seepage running freely down the slope is unstable, SF = 0.8). Figure 4.8 shows that a deep slip surface with the support toe berm has a SF = 2.4 which is far above stable conditions and an improvement of 65 % compared to the factor of safety of such a slip without the berm (SF = 1,45).

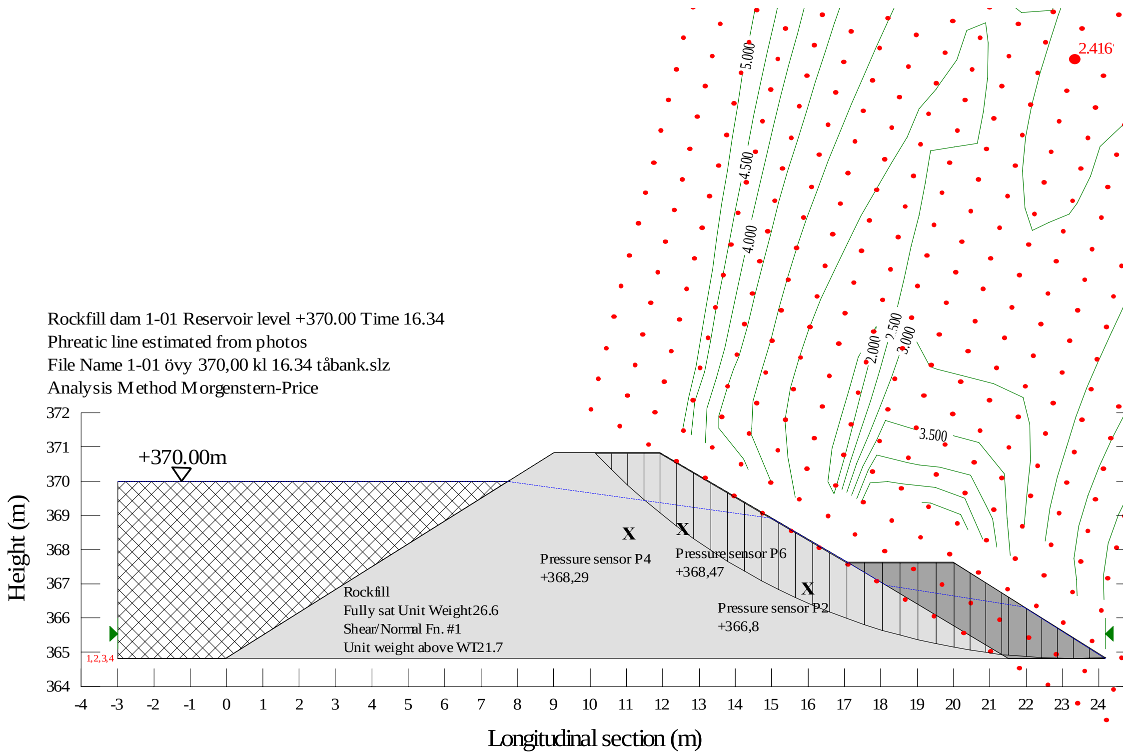

Figure 4.8 Deep slip surface with support of a toe berm. Higher reservoir level with high pressure in the dam. F = 2.4.

The dam was overtopped Nov 11th 2001 during which the pressure sensors measured unreasonable pore pressure (marked with X in figure 4.9), not taken into account in the following calculations. Similar to previous stability calculations thin slides (barely visible) occur with a factor of safety under stable conditions on the slope of the berm (SF = 0.9) in agreement with the actual test where small erosion took place at the toe at this point. Thin slip surfaces (with SF = 1.43) develop above the toe berm on the upper, unprotected part of the test dam, see figure 4.9.

A deep slip surface during overtopping (and complete saturation) has a factor of safety of 2.15 (figure 4.10) with the support of a toe berm. The analysis shows that if the rockfill test dam overtops unsupported the factor of safety of a "deep" slip surface is SF = 1.3, which indicates a 65 % increase in the factor of safety and improvement of the stability when saturated and with through-flow.

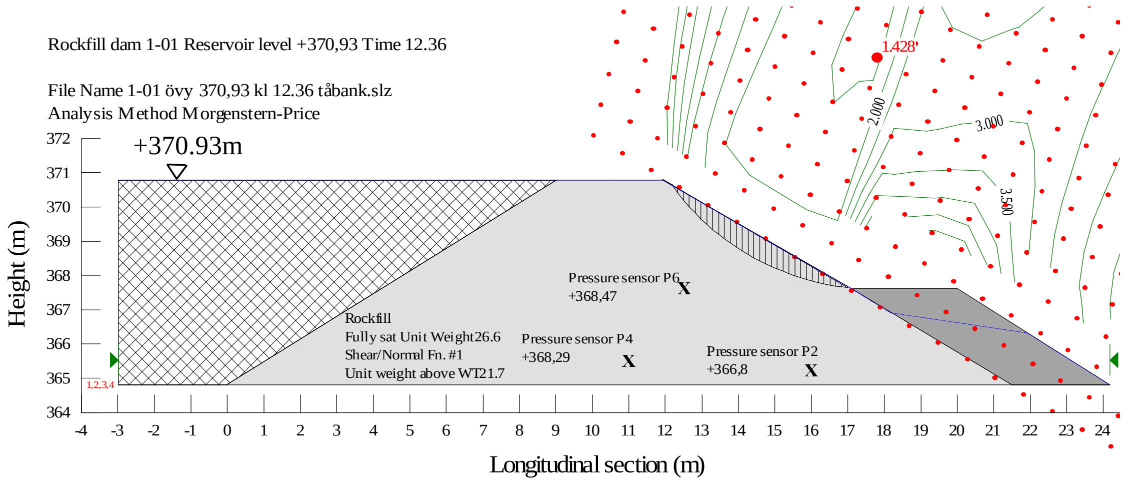

Figure 4.9 Thin slip surface during overtopping on the upper unprotected part of the slope above the toe berm. F = 1,43.

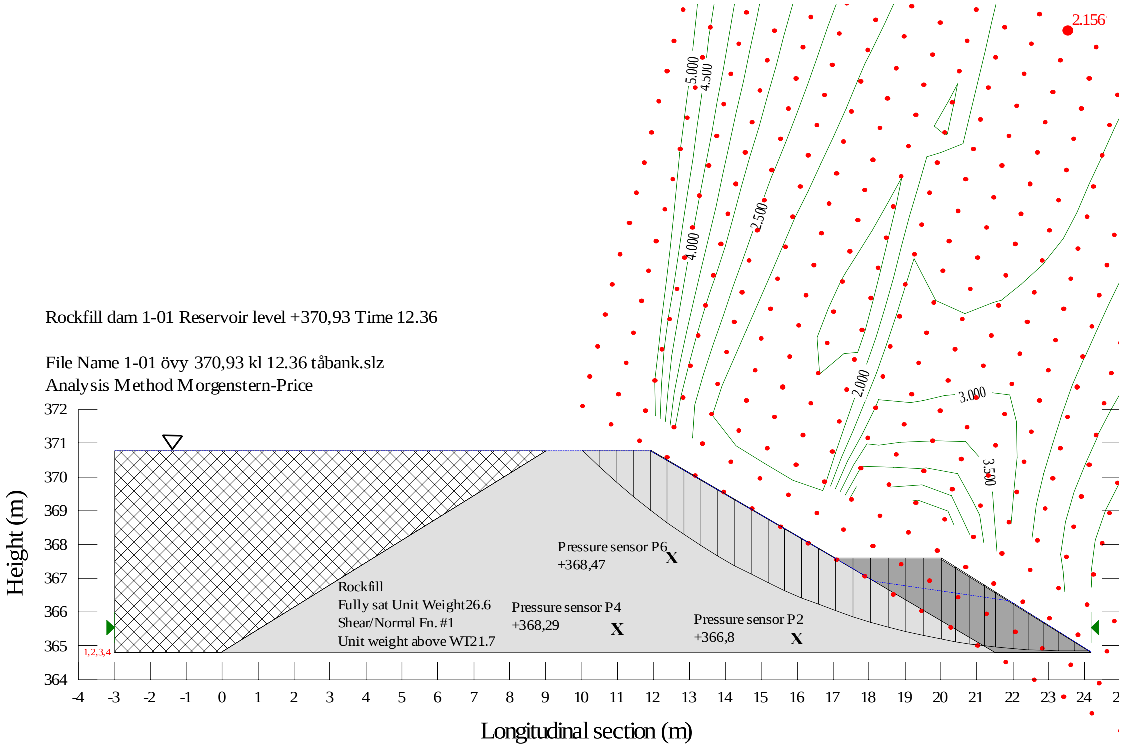

Figure 4.10 Deep slip surface during overtopping with supporting toe berm. F = 2,15.

5 Test Dam 2C-02 – Homogeneous Gravel dam

5.1 General

The gravel test dam was roughly 5 m high and the crest 2.2 m wide. The typical cross-section is displayed in figure 5.1 and the plan in figure 5.2. The slopes of the dam were 1V:1.57H. The dam crest had elevation +369.84m and the toe of the dam +364.81m.

In test 2C-02, carried out 2002-10-15 and 2002-10-16, the purpose was to study the breaching process of a dam with a material of minimum cohesion. The reservoir was raised to a level where the dam was overtopped in an arranged 2 m wide and approximately 0.12 m deep "channel" and the breach process started.

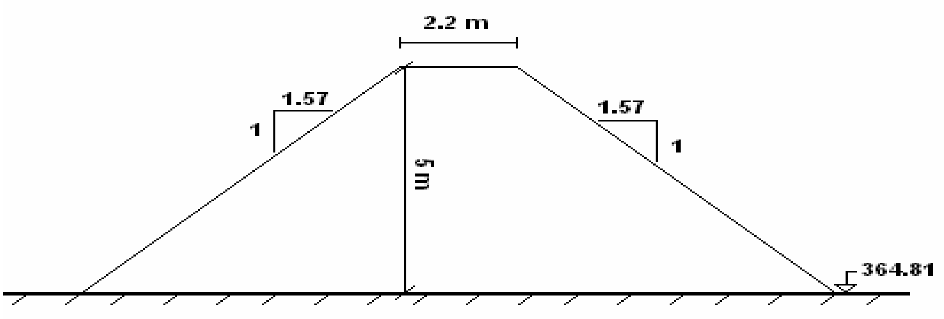

Figure 5.1 Standard cross-section of the 2C-02 gravel dam.

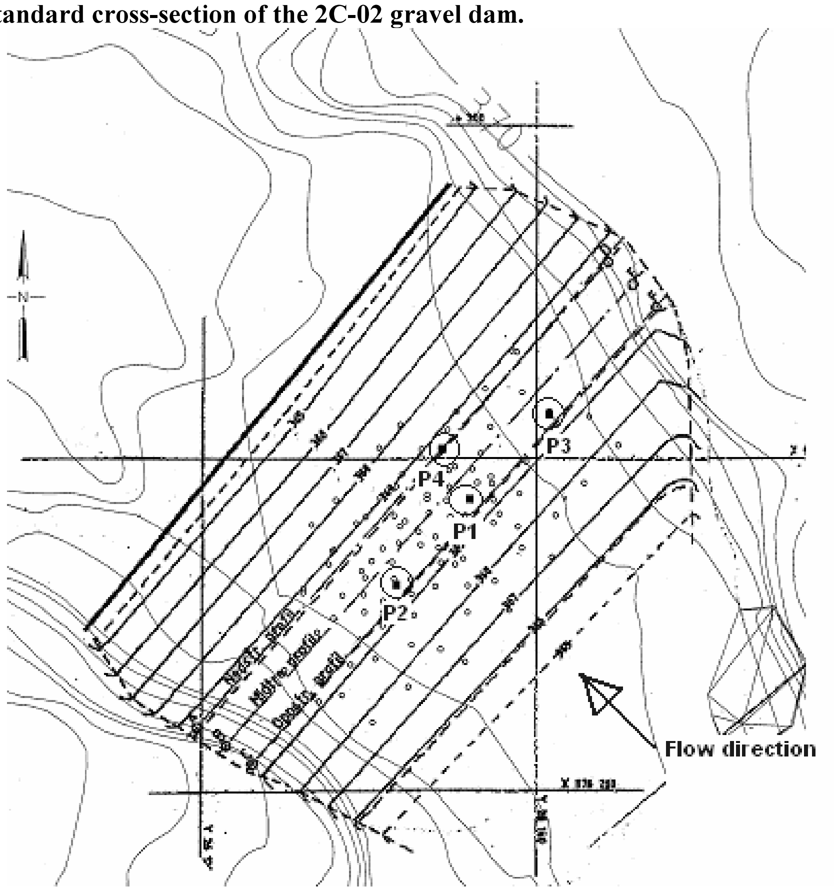

Figure 5.2 Plan view of the Gravel dam with location of the pore pressure sensors.

The test started at 10:00 by raising the reservoir to the channel. From 10:50 to 11:35 the water level stayed at el. 369.765, 0.045 m above the "channel" elevation. A wooden board avoided the water from running in the "channel". At 11:35 the breach initiation phase was started. The breach initiation phase (11:35 - 12:22) was mainly erosion downstream of the crest, making a "water fall" which developed its height, while working itself upstream towards the reservoir. The erosion process on the downstream side showed head cut characteristic. The seepage face and the elevation of the discharge point are estimated from photos taken during the test.

The test prior to 2C-02, namely 2A-02 and 2B-02, involved the same gravel dam as 2C-02 but arranged with a toe berm.

5.2 Composition of Fill Material

The test dam was made of gravel taken from a borrow area 600 m from the test site. Figure 5.3 shows the sieve curves for the material at different levels of the dam, which indicates that the dam is made of relatively broadly graded sandy gravel. The curves show some variation on different levels, but the Cu-factor seems to be equal. Level 4 has a Cu=19.7. This is well-graded masses and typical for Norwegian dams and Scandinavian embankment dam conditions in general.

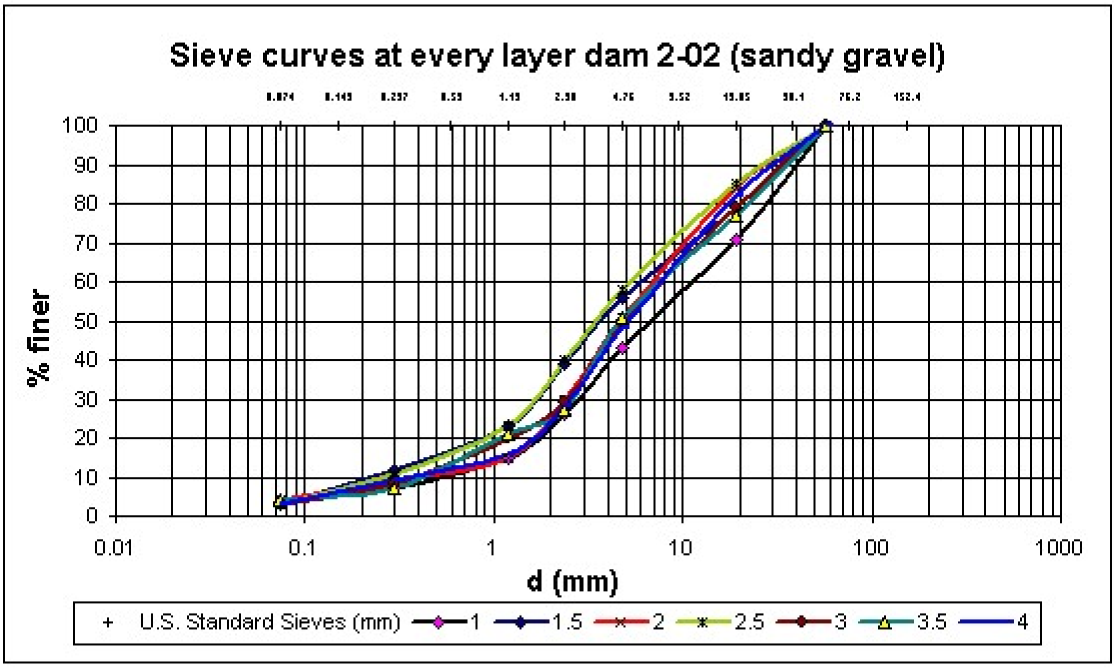

Figure 5.3 Sieve curves for dams 2A--C-02.

The laboratory tests conducted on the gravel material show characteristics according to table 5.1. The bulk unit weight is 22-22.5 kN/m^3 (average 22.3 kN/m^3) and from this fully saturated unit weight can be derived to vary between 22.6-26.5 kN/m^3 with an average of 24.6 kN/m^3.

The nonlinearity between the shear strength and the normal effective stress has for the material in the gravel test dam been estimated in a previous project (see [1]). The shear strength and friction angle were back figured according to table 5.2.

Based on the back calculation the shear/normal stress relationship for the gravel material is displayed in figure 5.4. This function represents a decrease in friction angle from 49° right at the surface to about 45° at the bottom of the dam, 5 m below the surface.

In order to accommodate for unsaturated soil conditions in the gravel material a ϕ_b-value of 20° has been set, meaning that above the seepage line (due to the influence of suction mobilizing through negative pore water pressure) the negative pore water pressure will have such enhancing effect on the shear strength. Otherwise the shear strength above the line of seepage will get no influence from suction. A ϕ_b-value of 20° falls within the commonly used bounds.

Table 5.1 Given gravel parameters and characteristics.

Gravel	Parameter		
Test dam	Density Bulk	kN/m^3	22 – 22.5
2C-02	Porosity		0.15 – 0.20
	Water content		0.03 – 0.05

Table 5.2 Back calculated φ'-values from model gravel dam.

σ'_n	φ'-value
(kPa)	(degrees)
10	49
50	47
100	46
200	45

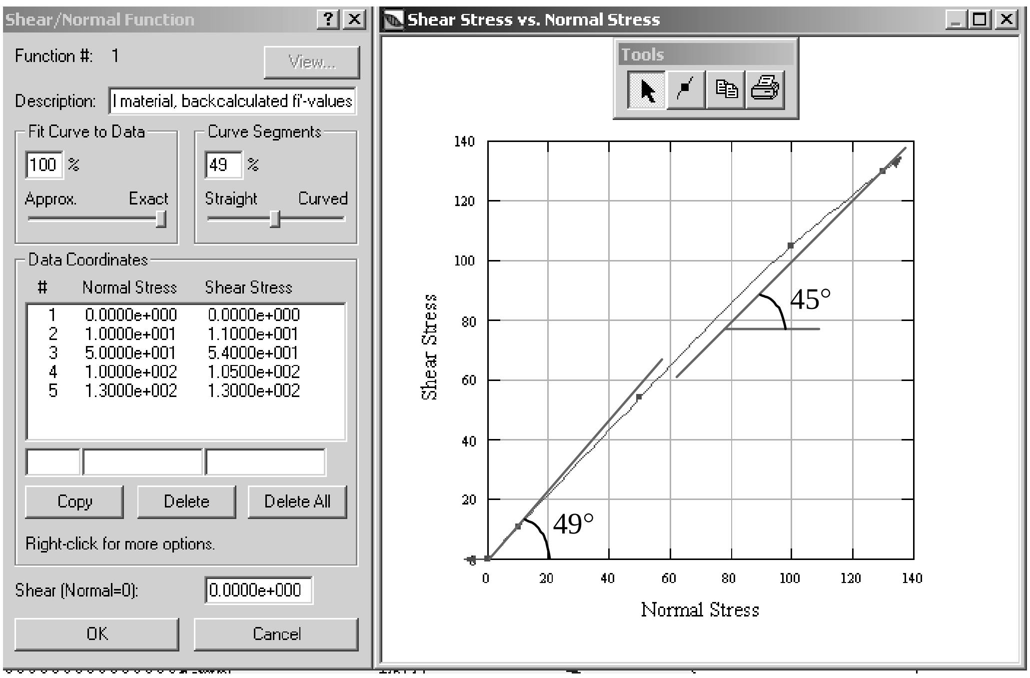

Figure 5.4 Estimated nonlinear shear/normal effective stress (kPa) relationship of the gravel.

5.3 Pore pressure test readings

The dam was equipped with pore pressure sensors for continuous measuring of pore pressure. In addition water level gauges were used to measure discharge. Figure 5.5 shows the water level upstream of the test dam during the test.

Table 5.3 shows the total head in the pore pressure sensors in the dam, compare with figure 5.6. The low flow q results in a negligibly low velocity head to which will not be accounted for in the following calculations.

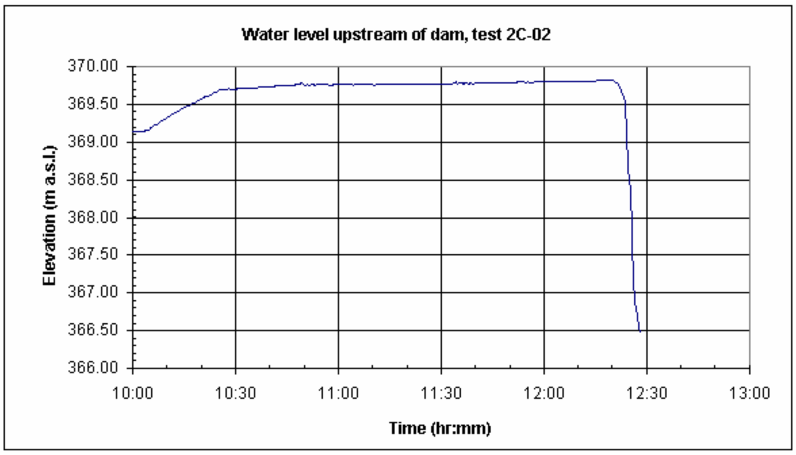

Figure 5.5 Reservoir level upstream test dam during test day.

Table 5.3 Measured pore pressure (total head) in sensor P1-P4 in test dam 2C-02. The velocity head has not been reduced.

Date (dd.mm.yy)	Time (hh:mm)	Water level (m a.s.l)	q (l/s/m)	P1 (m a.s.l)	P2 (m a.s.l)	P3 (m a.s.l)	P4 (m a.s.l)
16-10-02	09:37	369,14	0,0306	368,51	367,92	367,76	367,32
16-10-02	11:30	369,76	0,0389	368,95	368,36	368,00	367,71

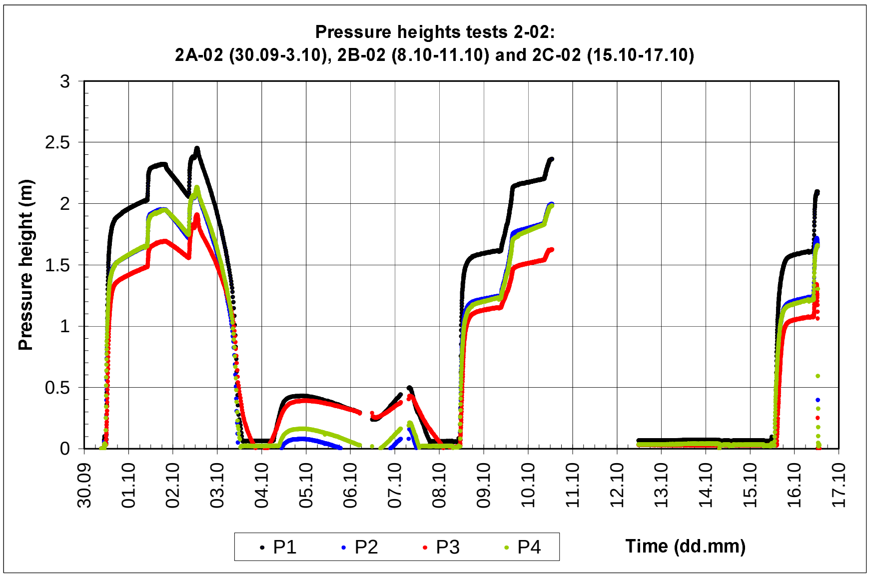

Figure 5.6 Pressure head in pore pressure sensors during the different tests.

5.4 Analysis

Figure 5.7 shows the analysis model cross-section during the initial test of gravel dam 2C-02 Oct 16th 2002. Due to the dense and tight quality of the gravel material changes in reservoir level was made gradually and then letting it set in order to permit adjustments in the pore pressure while it positioned.

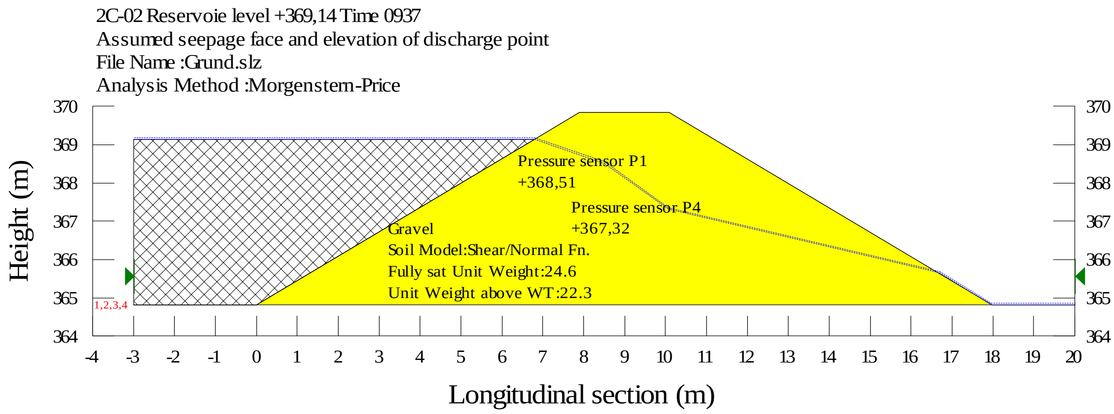

Figure 5.7 Gravel test dam 2C-02 model.

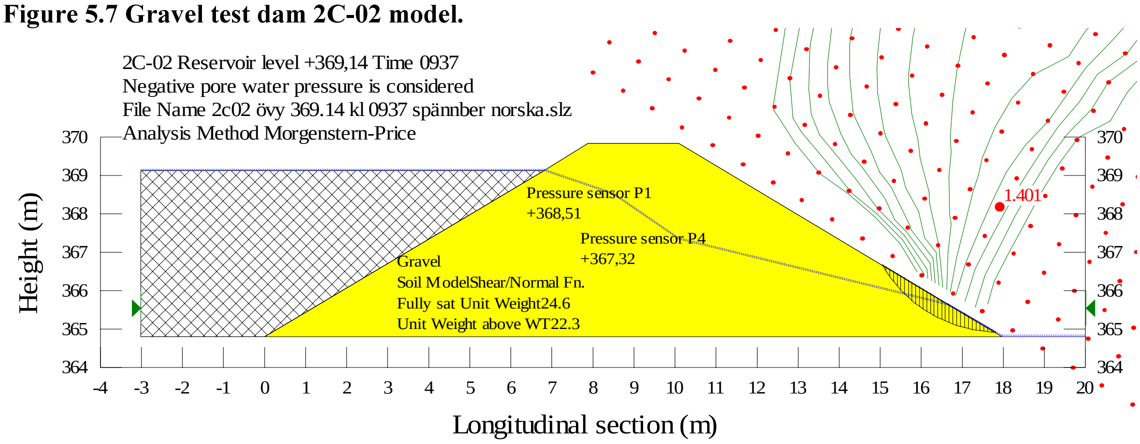

Figure 5.8 Thin slide on unprotected slope at +369.14. F = 1.4.

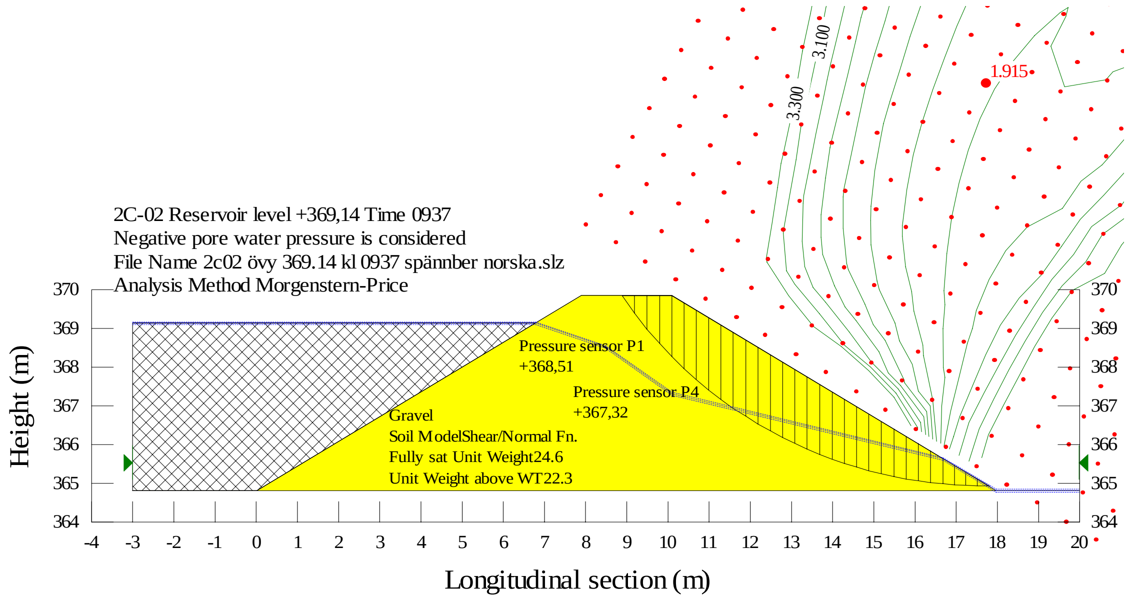

Figure 5.9 Deep slip surface on unprotected slope at +369.14. F = 1.92.

At the initial phase of the test the reservoir was set to +369.14, 0.7 m below the crest. Pressure sensor P1 and P4 are used in the model to estimate the phreatic line in the dam.

The analysis show that the factor of safety for thin barely visible slides is below stable conditions (SF = 1) and these slides occur just below the seepage point under the influence of uplift. So according to this some erosion should have initiated under the seepage face during the first part of the test, this never happened. The influence of suction is this case is minimal because the failure occurred below the point of discharge. Maybe the erosion process would have started in the actual test dam if this case of load had continued for a longer time allowing the seepage situation to settle. A more tangible, but still small slip surfaces, has SF = 1.4 (according to the analysis in figure 5.8). Without the influence of suction 1.33.

The factor of safety of a deep slip surface that probably would have resulted in a breach under these conditions is 1.92 (1.83 if negative pore water pressure is not considered, no suction), see figure 5.9. This is far above stable conditions.

The first gravel test dam 2A-02 was not brought to failure and it was equipped with a rockfill downstream berm all the way up to the crest. This dam was overtopped which initiated some erosion on the rockfill slope. Before the subsequent test (2B-02) the rock fill in the middle of the dam was removed, exposing the gravel that proved to be very erodible. The test continued with 2C-02 before which the entire rockfill was removed and the dam was brought to failure. Assuming that a 2 m wide rock fill berm was placed on the downstream slope (similar to what was placed on to the previous test dam 2A-02), with the same characteristics as the rock fill in test 1-01, this would result in a 33 % improvement of the stability (factor of safety), se figure 5.10. SF = 2.54 with the toe berm. The force polygon is attached to figure 5.10 showing the interslice forces, normal forces and shear force of a slice subjected to pore pressure uplift in the slip surface.

When the dam was overtopped the breach initiation phase started with small erosion on the downstream side evolving to a head cut working its way backwards. The pressure sensors measured what seem to be unrealistic pressures (marked with X in figure 5.11) during the ongoing overtopping. Disregarding the pressure sensors and assuming fully saturated body of the dam the stability analysis show that factor of safety is less than stable (0.7-0.8) for thin slides on the downstream slope, which correlates well with the test.

Such barely visible slides can evolve into a more or less continuous process with progressive erosion. Even deep slip surfaces acting on an unsupported slope (figure 5.11), which probably would develop to a fast breach, have stability below SF = 1 although the influence of negative pore water pressure is considered. This is an inconsistency with the actual test result. Note that in this case it is assumed that the entire body of the test dam is saturated (submerged), which naturally makes the influence of negative pore pressure nil.

If the pore pressure readings are considered correct it means that the upper part of the dam is not fully saturated though overtopping is taking place and water is flowing freely down the slope. Under such conditions there can be a mobilized suction, which increases the shear strength of the unsaturated zone within the capillary reach rendering a factor of safety of 1.53, displaying a more realistic result comparable to the actual field test (figure 5.12). Assuming that this is the case then the submersion is continually increasing as the overtopping goes on, closing in on fully saturated conditions and by this the stability is gradually decreasing towards unstable conditions (see figure 5.11).

Figure 5.13 disclose the influence of a toe berm, which increases the factor of safety from 0.96 to 1.94 assuming the upper part of the dam is completely saturated (no suction), which means a 110 % improvement of the stability. The toe berm is placed all the way up to the crest.

If the sensors are assumed to register pore water pressure correctly (analogous to previous case without toe berm) the influence of suction will increase the factor of safety to 2.32, which is a 51 % increase in comparison to unprotected conditions (see figure 5.14).

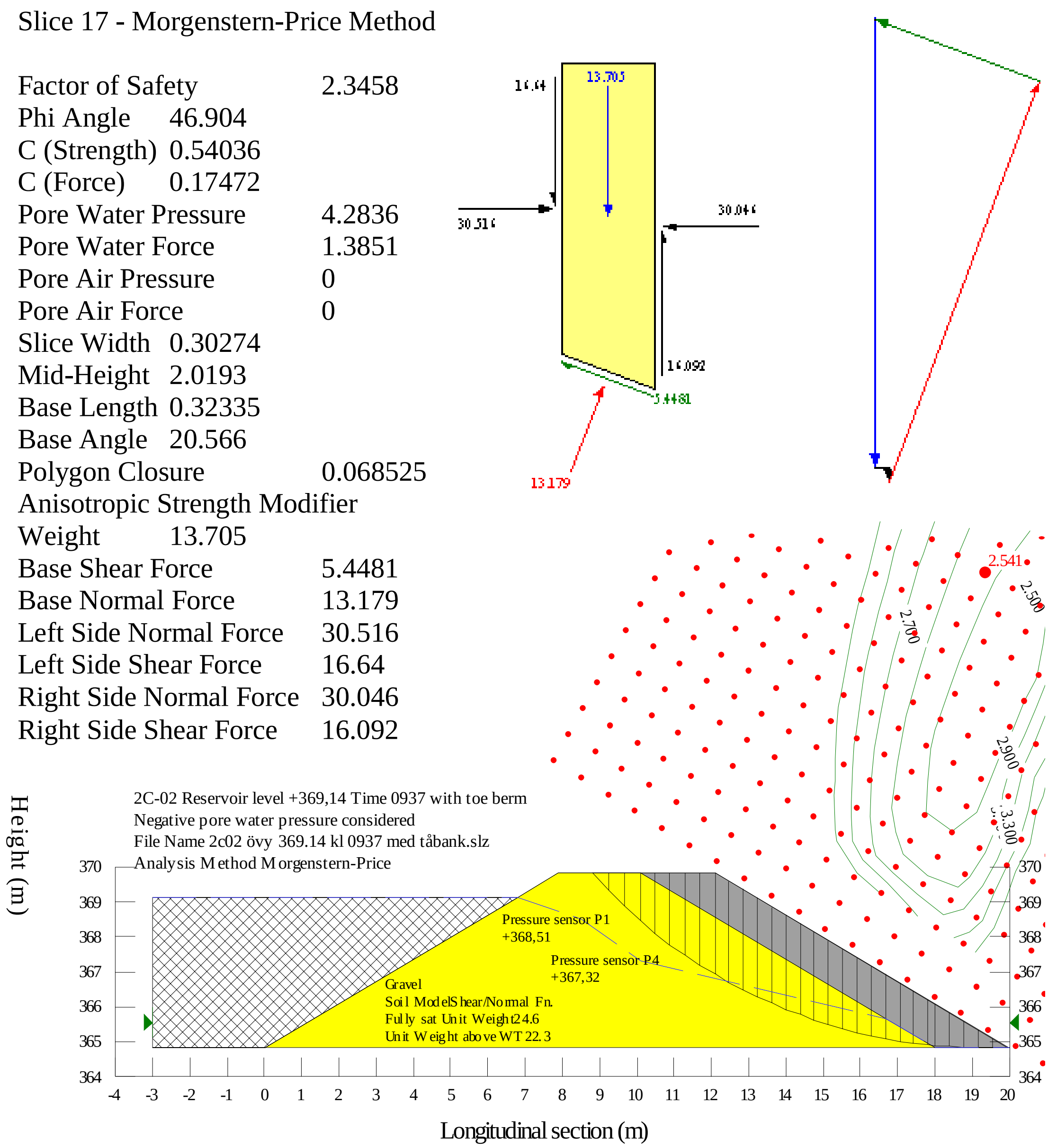

Figure 5.10 Deep slip surface with toe berm supporting the slope at a reservoir level at +369.14. F = 2.54.

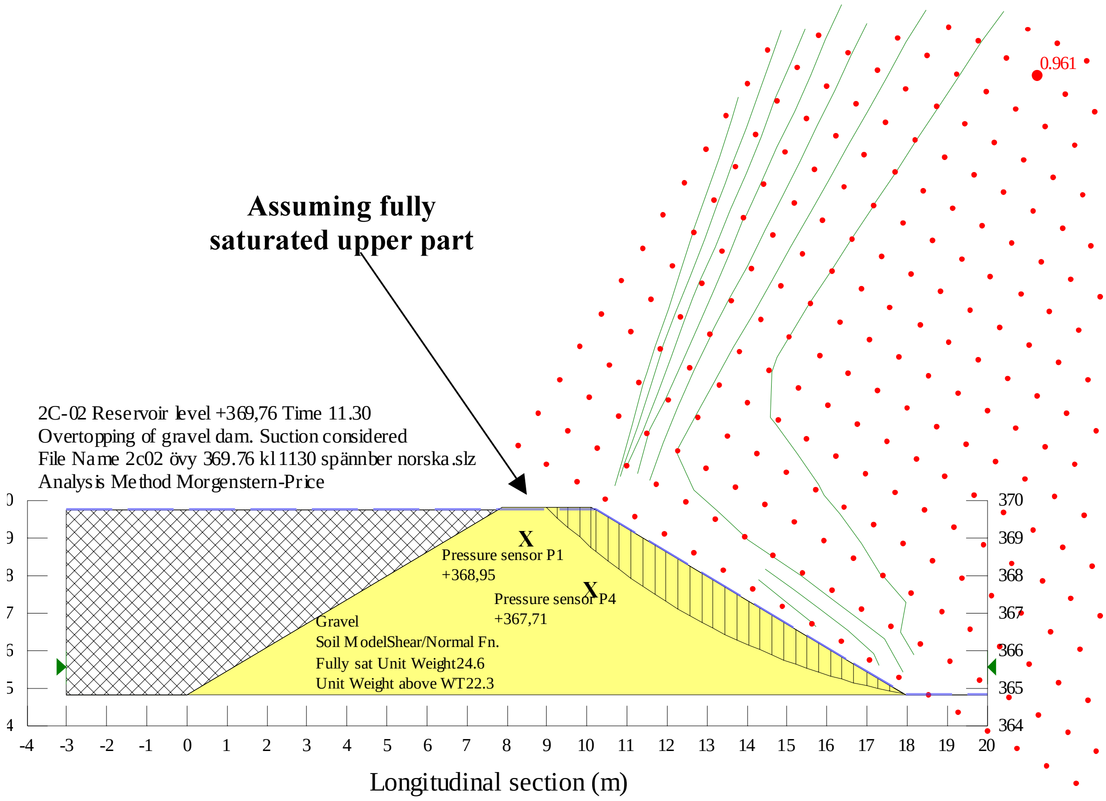

Figure 5.11 Deep slip surface during overtopping with unsupported slope. F = 0,961. Totally saturated material.

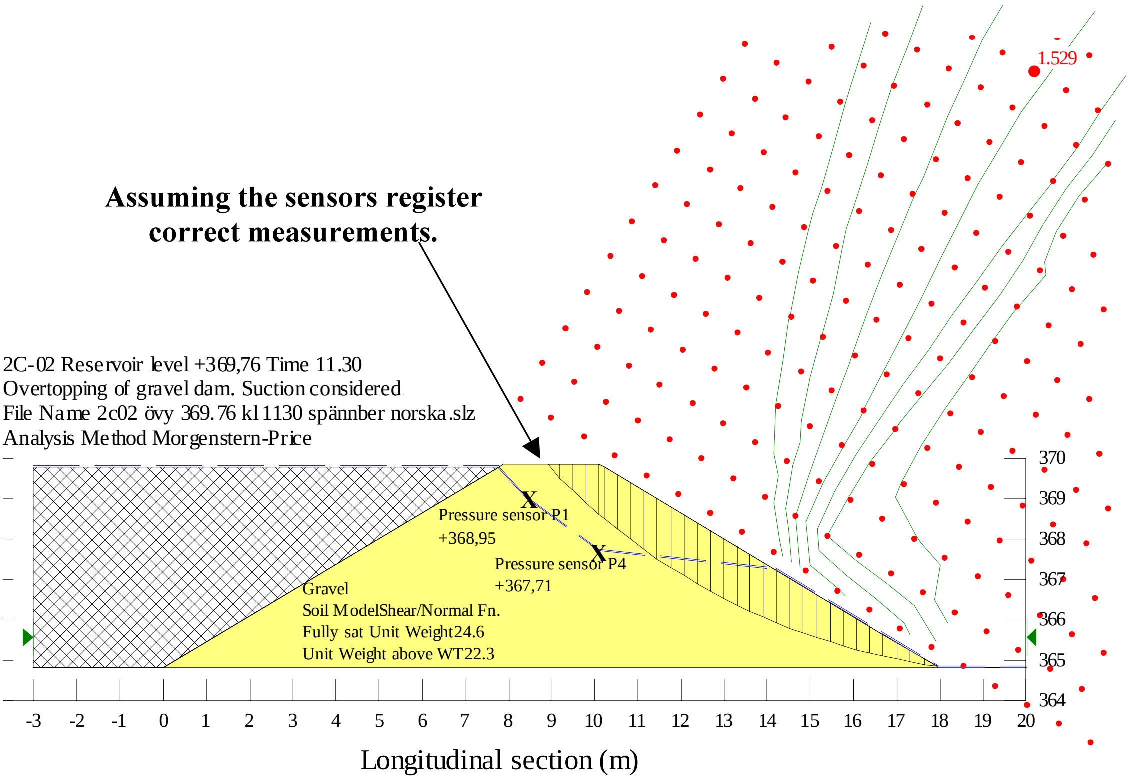

Figure 5.12 Deep slip surface during overtopping with unsupported slope. F = 1.53. The upper part has not yet been fully saturated permitting suction to mobilize.

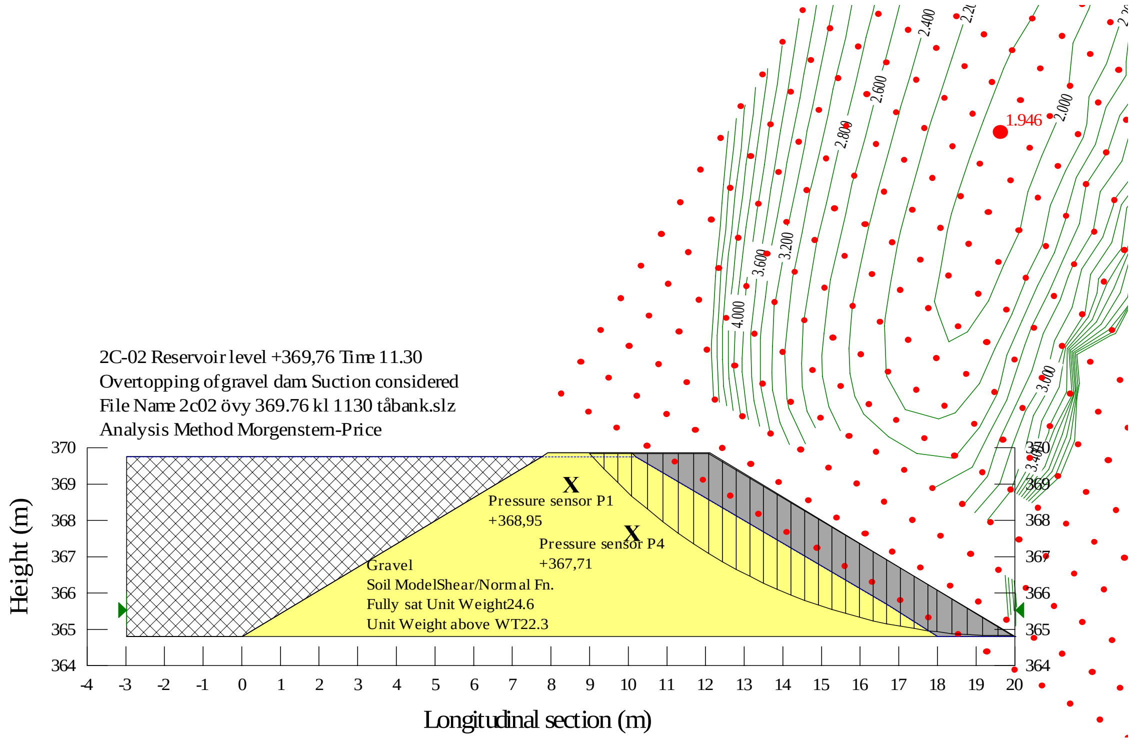

Figure 5.13 Deep slip surface during overtopping with toe berm. F = 1.94. Upper part fully saturated.

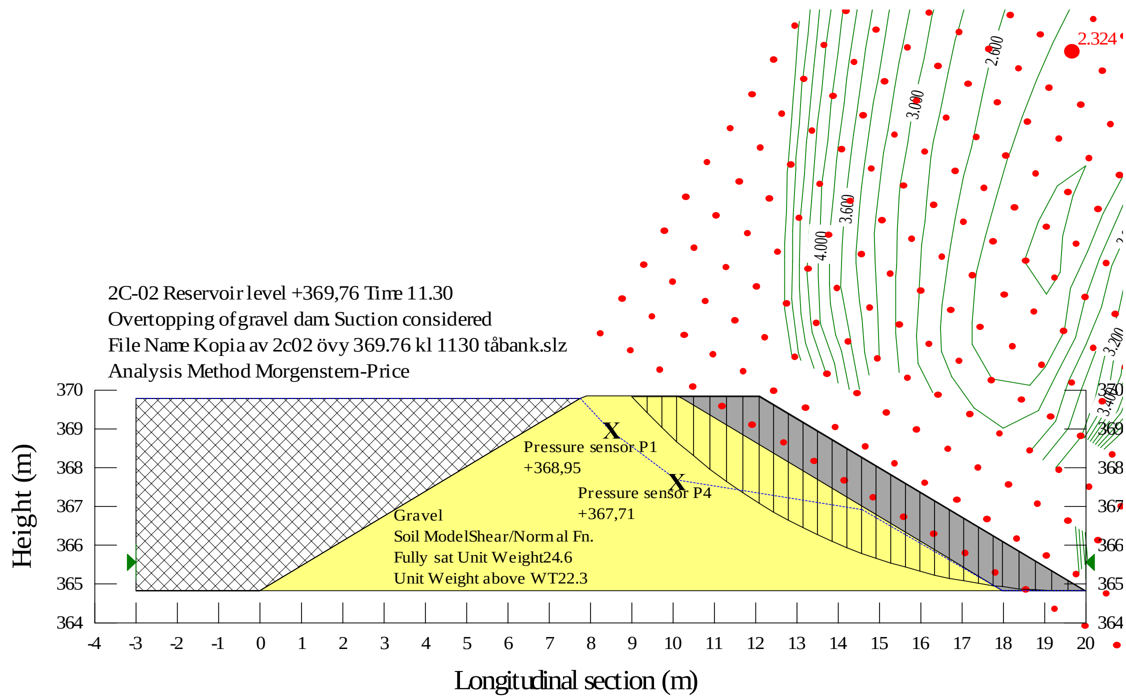

Figure 5.14 Deep slip surface during overtopping with toe berm and influence of suction. F = 2.32.

6 Conclusions

6.1 Test Dam 1-01 – Rockfill Dam

Based on the strength model chosen for the rockfill test dam the result of the analysis correlates quite well with actual test.

At unprotected conditions small slides have a factor of safety just above 1, and these slides generally develop just below the discharge point according to the calculations. Even a deep going slip surface when the dam is undergoing overtopping has a factor of safety above 1, such slope instability would probably have initiated a breach. A toe berm (of the same characteristics as the one placed in test 2-01), when the rockfill dam is overtopped, increases the factor of safety (stability) with 65 % for such a case of load.

Table 6.1 displays the change in factor of safety of “deep going slip surfaces” with a 2.8 m toe berm placed on the downstream side under different case of load.

Table 6.1 Change in factor of safety of “deep” slip surfaces after placement of a 2.8 m high toe berm on the rockfill test dam.

Reservoir Level (m.a.s.l.)	Factor of safety		
	Without toe berm	With toe berm	Improvement of stability
+368.88m (crest - 1.9 m)	2.15	3.23	50 %
+370.00m (crest - 0.8 m)	1.45	2.4	65 %
+370.93m (overtopping)	1.3	2.15	65 %

6.2 Test Dam 2C-02 – Gravel Dam

The result of the gravel test dam analysis correlates well with the actual test if the influence of negative pore pressure is considered.

The factor of safety for thin slides, when the dam is unprotected, is below stable conditions and these form just below the seepage point on the downstream slope as would be expected. In this case the there is no effect of suction assuming the soil is fully saturated.

Table 6.2 displays the change in factor of safety of “deep going slip surfaces” with a 2 m wide toe berm all the way up to the crest under different case of load.

When the breach process initiated during the test small erosion started working its way backwards until failure occurred and this happened when the dam was undergoing overtopping. The analysis shows that thin slides are unstable during overtopping, which agree with the actual breach process. Even deep slip surfaces, which would have evolved to a fast failure, have factor of safety just below 1 under the assumption that the upper part of the dam is fully saturated, i.e. no influence from suction. This was not the case during the test and does not agree well with the actual result.

Table 6.2 Change in factor of safety of "deep" slip surfaces after placement of a 2 m high toe berm on the gravel test dam all the way up to the crest.

Reservoir Level (m.a.s.l.)	Factor of safety		
	Without toe berm	With toe berm	Improvement of stability
+369.14m (crest – 0.7 m)	1.9	2.5	33 %
+369.76m (overtopping)	0.96 [2] (1.53) [1]	1.95 [2] (2.32) [1]	110 % [2] (53 %) [1]

[1] The effect of suction mobilizing from negative pore water pressure is applied if the pore pressure sensors are considered to register correctly.
[2] Fully saturated body of the dam.

If the suction effect is considered a more realistic result is obtained, correlating with the actual test. This means that the negative pore pressure mobilizes a suction, which increases the shear strength of the material above the phreatic surface. By and large negative pore pressure has an increasing effect on the shear strength above the zero pressure line as the (positive) pore pressure below the seepage line has an adverse effect.

7 References

[1] Norwegian Electricity Industry Association EBL; “Stability and Breaching of Dams. Report on Sub-project 1: Shear strength of Rockfill and Stability of Dam Slopes”, EBL Kompetanse, Publikasjon nr.: 123-2003.

[2] Norwegian Electricity Industry Association EBL; “Stability and Failure Mechanisms of Dams. Sub-project 2: Stability of the downstream shell and Dam Toe during Large Throughflows. Data report 3: Large Scale field-tests downstream of the dam Rosvassdammen 2001”, Technical report.

[3] “Stability and Breaching of Dams. Data report 4: Large scale field tests 2002”, Technical report.

[4] US Army Corps of Engineers; “Slope Stability – Engineer manual. Engineering and Design”. EM 1110-2-1902. 31 Oct 2003.

[5] Leps, T.M.; “Review of shearing strength of rockfill”, ASCE Journal of Soil Mechanics and Foundations, Vol. 96, SM4, pp. 1159-1170, 1970.

[6] Høeg, K, Løvoll, A and Vaskinn, K. A., “Stability and Breaching of Embankment Dams; field tests on 6 m high dams”, Hydropower and Dams, issue 1, 2004.

[7] SLOPE/W Version 5 User´s Guide GEO-SLOPE Office. (http://www.geo-slope.com).

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