

**Master Thesis, Department of Geosciences**

# **Failure of an Earth Dam**

*An analysis of earth dam break – Årbogen dam, Nedre Eiker municipality, Norway*

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**UNIVERSITY OF OSLO**

**FACULTY OF MATHEMATICS AND NATURAL SCIENCES**

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Master Thesis in Geosciences

Discipline: Environmental Geology and Geohazards

Department of Geosciences

Faculty of Mathematics and Natural Sciences

University of Oslo

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## **Abstract**

The consequence of dam failure is one of the major hazard to human life as well as to infrastructure. Huge flood waves produced by a failure of dam may have enough capability to destruct some stronger structures like power plants and industrial plants. Studies about a natural dam failure are very common, but the thesis included numerical modeling and detailed analysis of dam conditions when a dam is under abnormal flooding conditions.

The old industry dam Årbogen has been failed during an intense rainfall occurred in August 2010 with three major slides along the downstream slope. The local municipality requested to Norwegian Geotechnical Institute (NGI) to carry out some immediate temporary measures and a detailed analysis of dam condition.

The primary objective of the thesis is analyzing the seepage and stability of the Årbogen dam for the extreme flood condition caused by the intense rainfall. In addition, soil analysis of the deposits, determination of flood path and flow accumulation, and consequence of the dam break are considered as secondary tasks. Stability and seepage has been analyzed using the popular geotechnical software called GeoSlope. A contour map is used to explain the possible flood paths which may impact the nearest community.

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# CHAPTER 1

## 1.0 INTRODUCTION

The designing and construction of an Earthfill dam is one of the key challenging in the field of Geotechnical engineering, because of the nature of the varying foundation condition and the range of properties of the material available for construction (U.S. Army corps engineers 2004). The major advantages of the earthfill dams are easily adapting to the foundation and accommodate even in difficult site condition. The most common and basic earthfill dams are known as homogeneous. (Jansen et al. 1988). This type of dams entirely constructed with same material. However, at present designing of earthfill dam with relatively impervious core is increased for the purpose of controlling seepage through the dam(Jansen et al. 1988).

World-wide there are approximately 30,000 earth dams higher than 15m and more than half of them are constructed since 1950(HÖEG 2001).

*Table 1: World's highest earth dams(HÖEG 2001)*

<i>Name</i>	<i>Country</i>	<i>Height (m)</i>
<b>Rogun</b>	Tadjikistan	335
<b>Nurek</b>	Tadjikistan	330
<b>Manuel M. Torres</b>	Mexico	261
<b>Mica</b>	Canada	243
<b>Alberto Lleras C.</b>	Colombia	243
<b>La Esmeralda</b>	Colombia	237
<b>Oroville</b>	USA	235

Successful designing and construction of an earth dam should be fulfilled the following technical and administrative requirements(U.S. Army corps engineers 2004).

- a. Technical requirements
  - i. Dam foundation and abutment must be stable at all static and dynamic loading conditions.
  - ii. Should have a special design for control and collect seepage through the foundation, abutments and embankment.
  - iii. The outlet capacity of the spillway must be sufficient to prevent overtopping of embankment by reservoir
- b. Administrative requirements



- i. Environmental responsibility
- ii. Detailed operation and maintenance methods
- iii. Monitoring plans
- iv. Adequate instrumentation
- v. Records for all operation and maintenance activities
- vi. Emergency action plan (included identification and immediate response)

In Norway there are 250 dams are higher than 15m in height and 172 of them are known as earth dams. The designing and construction of earth dams were started in 1924, anyway after 1950 it have become important aspect because of the hydropower development(HÖEG 2001). According to the historic records none of the Norwegian embankment dams higher than 15 m have failed up to date. At the same time several small dams have failed because of inadequate dam engineering works involved. Therefore a careful attention and regular inspection should be focused on small earth fill dam to prevent possibilities of failure.

## 1.1 Purpose of the Investigation

On 13<sup>th</sup> August 2010, there was an intense rainfall occurred in Nedre Eiker area that resulted an increase of water level in a small – old industry dam called “Årbogen dam”, by 5cm above the water level corresponding to estimated Probable Maximum Flood (PMF) level. The spillway of the dam was partly blocked by the suspended particles carried out through water and caused overflow of the natural earth dam. On the side of the constructed concrete dam, at one stage the deposit was inundated and the ground water – direct overflow combination washed out the mass away and caused two major slides with several minor slides. There are some old scars observed in the deposit which may produced by some similar events in the past.

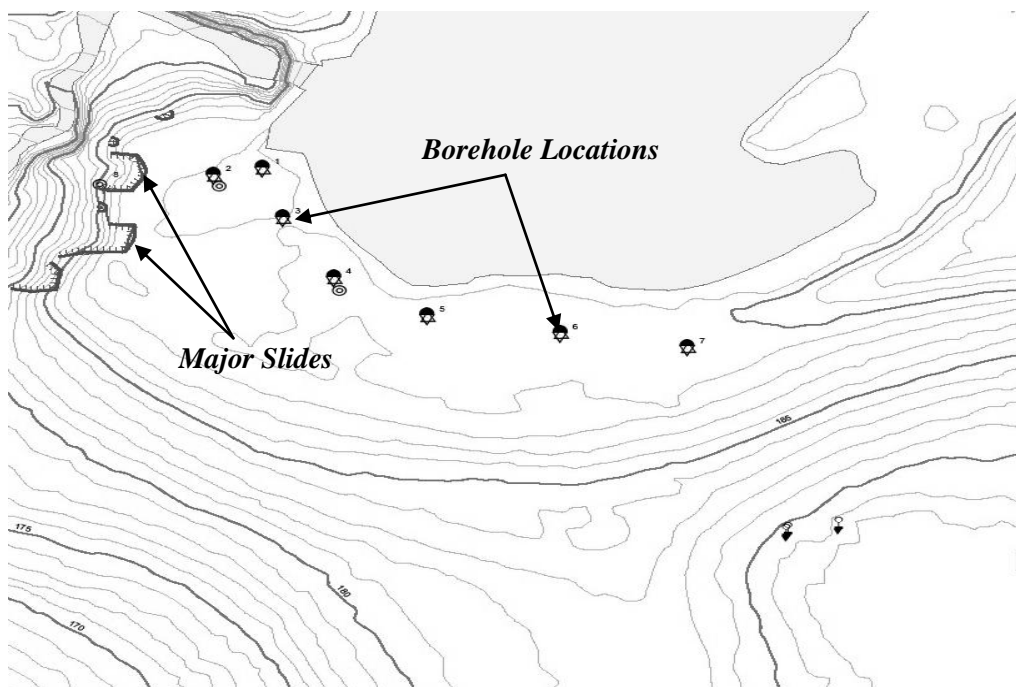


*Figure 1.1: Two main slides happened at the Årbogen Dam site.(Captured during the field visit on 28th October 2010)*

The slides were stabilized by Norwegian Geotechnical Institute (NGI) immediately after the event with large rock boulders (Drammen Granite). NGI has been asked to reanalysis of stability and seepage of the dam by the Nedre Eiker municipality.

As the first step of the project NGI has been drilled 7 boreholes along the deposit and collected some samples as well. In the master thesis the following main tasks are included.

1. Interpretation of drilling and samples
2. Seepage/Stability analysis
3. Flood analysis
4. Consequence of the dam break



*Figure 1.2: Major slides and borehole locations*

The figure 1.2 shows the topographical map of the locations of major slides happened during the flood and the boreholes which have been drilled by NGI.

The top of the deposit is at about 190 m.a.s.l. which is about marine limit (ML) ie: the deposit is a glacifluvial which deposited into the marine of a previous fjord.

## 1.2 Literature of the Dam site



Figure 1.3: Årbogen Dam location (Nedre Eiker kommunes kartside)

Dam Årbogen is located just above the Krokstadelva in Nedre Eiker kommune, Norway and was originally built around century in connection with water supply for industrial purposes in Krokstadelva. However, the dam is no longer for industrial purposes and being part of a popular outdoor area. The pond was originally a bricked homogeneous earthfill and the dam crown is later built with concrete.

The Nedre Eiker kommune, technical service group completed rehabilitation of dam Årbogen in the spring and summer of 2003. The quality of the implemented control, the work performed and the final result must be regarded as fully satisfactory. The execution is according to the description and drawings.

## 1.3 Rehabilitation of Årbogen dam (Willassen 2004)



Figure 1.4: Årbogen Dam during the rehabilitation in 2003

The rehabilitation of the brick/concrete part of the Årbogen dam is carried out with the basis of the following set of documentations.

1. Flood calculations Ulevann, Øyevann and Årbogen, 1998, Consultant.
2. Alignment of flooding calculations for Ulevann, Øyevann and Årbogen, June 2000, Consultant.
3. Review dam Årbogen, June 2001, Consultant.
4. Project rehabilitation dam Årbogen, 2002, Consultant.
5. Adjusted stability calculations, November 2002, Consultant
6. Proposal engineering works, January 2003, Consultant
7. Working drawings, January 2003, Consultant, INC.
8. Permissions to the temporary lowering of Årbogen, March 2003, NAF VEIBOK.
9. Blanket permission and permission for this project for initiated setting plan and construction law.
10. Plan for building management and control, rehabilitation of dam Årbogen
11. Various standards, guidelines and regulations those are relevant to the work.
12. The approval of plans for the rehabilitation of dam Årbogen, July 2003, NAF VEIBOK.
13. Note from the Engineering Geologist Morten Lund regarding securing downstream rock part, 10. June 2003.

In the rehabilitation works there are several measures carried out to improve dam stability, flooding lead capacity as well as secure against leaks. In addition, the actual spillway enhanced to improve flood lead. Along the downstream side a downstream reinforcement cast was constructed against the existing wall.

In order to implement the measures the pond was a lowered of water level in the reservoir began just before Easter. The water level was lowered by the use of existing gate through the dam, and right after Easter, the water level was lowered sufficiently for the work on the upper part of the dam could start.

It was dug to bedrock along the upstream side, cleaned and grouted permanent rock bolts. Completely against both connections were not the old dam built of carved stone that fit the remaining part of the dam. The dam here consisted of dry vegetation is stunted stone masonry with a thin front and crown of concrete screeds.

The dam is entirely founded upon a rock. The rock condition varies widely between different locations near the dam. Generally, rocks marked by significant gaps and are much fractured. On the upstream side, it was not made special arrangements in the rocks apart from ordinary clean.

Some construction joints were adjusted upon request from the contractor. In addition, the location of certain construction joints adjusted slightly after the rock was uncovered. By assembling the formwork was used consistently stays attached to the brickwork as expansion bolts. After casting the supports were removed and the rod and cone holes on the finished surface was sealed by the use of massive clotting of fiber-reinforced concrete that was glued with two component adhesives.

# CHAPTER 2

## 2.0 THEORY AND METHODS

### 2.1 Earthfill dams

Earthfill dams are simple structures which stand on their self-weight to prevent the sliding and overturning(Jansen et al. 1988). These dams are the most common type of dams known in the world. At the earlier time the earthfill dams are constructed to divert massive water body and protect the community. Later it was structurally improved and used to construct the reservoirs.

Small earthfill dams contain a variety of advantages in both technically and economically. They are(STEPHENS 2010);

1. Construction materials are easily available
2. Simple design criteria
3. Less foundation preparation required when compared with other dams
4. Quiet flexible than other rigid dam structures and suitable for seismic sensitive regions

On the other hand, there are some disadvantages when compared with other dam types(STEPHENS 2010).

1. Higher possibility to damage or slide than other dam types
2. Lack of compaction of material leads to increased seepage
3. Continuous monitoring and assessment needed to prevent the slope erosion, abnormal seepage condition and growing plants

#### 2.1.1 Design criteria

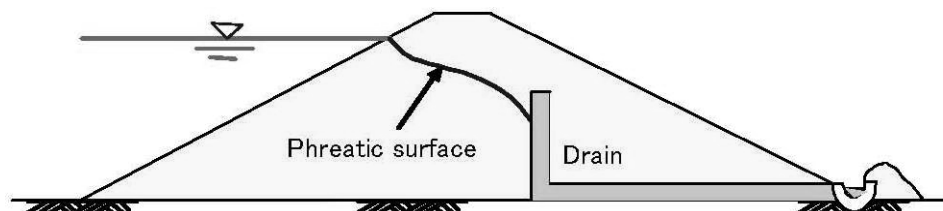


Figure: 2.1 A simple design of a Homogeneous earthfill dam(Narita 2000)

A homogeneous earthfill dam should be designed with relatively flat slopes to reduce the possibility of failure (Generally 1:3 in upstream side and 1:2 in downstream side)(STEPHENS 2010). Unlike other dams, the dam body is the only structure which provides structural and seepage resistance against failure and required drainage facilities for a homogeneous earthfill dam(Narita 2000). The design is unique for each earthfill dams because of the location of the dam and the variety of materials to be used for construction. Purpose of the dam also plays an important role on design criteria. The factors mentioned above bring hard to define a general design criteria(Kutzner 1997). However, every design criteria must be included the following fundamental design aspects(Jansen et al. 1988):

1. Stability of embankment and foundation in critical conditions such as Earthquake and flood.
2. Control of seepage and pressure in both embankment and foundation
3. Safety measures to control overtopping situation
4. Erosion control methods

A dam may lose its performance by time because of the long term changes in the properties of constructed materials. A typical example is the material may become more anisotropic than when it was at the stage of construction. Also, deposition, displacement and biological growth are some other considerable process which may impact on the performance of a dam(Jansen et al. 1988). To maintain the performance of a dam, critical conditions such as earthquake, overtopping and unexpected increase of seepage quantity are should be overcome with controlling structures such as filter – protected chimney drains, horizontal drain blankets, foundation cut – offs, relief wells and abutment drainage curtains(Jansen et al. 1988).

### 2.1.2 Materials

A good embankment soil material has to be water insoluble and should contain inorganic substances as long as possible. Hence, the clay with higher water content (more than 80%) and crushed rock powders are strongly avoidable materials(Brown 2004). Generally fine grained soils are very suitable for embankment construction but, those should be within a particular range of moisture content and fulfill the requirements for compaction(Brown 2004). Because in the case of fine grained soils with higher water content, the self weight of the embankment may develop the higher pore-water pressure within the embankment dam.

A well graded (wide range of particle size) soil is always preferable than a uniform soil when the other properties of the both soils are equal. Because the well graded soils are less susceptible to piping and liquefaction and soil erosion(Brown 2004). However, for any soils, the large boulders which have the particle size greater than the required thickness of compacted layer must be removed before compaction. This operation will raise the performance of compaction.

## 2.2 Permeability and Seepage

### 2.2.1 Permeability

Permeability is one of the most important properties that Hydrologist, Geotechnical engineers and ground water professionals always deal with (Cedergren 1989). Naturally all the soil materials are *permeable*, means water can flow through the soil by the interconnected pore spaces in the soil. The quantity of permeability is always denoted by the term *Coefficient of Permeability* ( $k$ ). A permeable material must have the ability to be penetrated by another material such as gas or liquid. Most of the soil and rocks with cracks and joints are some common permeable materials which deal with geotechnical works.

The *Coefficient of permeability (also called as Darcy's Coefficient)* is determined by Darcy's empirical Law:

$$Q = A.k.i \quad \text{OR} \quad k = Q/Ai \quad (2.1)$$

Where;  $Q$  is the volume of water flow through the soil per unit time,  $A$  is the cross-sectional area of the soil,  $k$  coefficient of permeability (m/s) and  $i$  is the hydraulic gradient.

The coefficient of permeability depends on particle size and shape. Because the size of the pore space determines the quantity of permeability. Generally smaller particles have low permeability because of the smaller pore space hence larger particles have higher permeability. However, presences of fine grains in a coarse-grained material pull down the permeability significantly because, the tiny particles considerably filling out the pore-spaces. The coefficient of permeability also depend on the temperature changes, as the viscosity changes in fluids by temperature (R.F.Craig 2004b). The relationship between viscosity and coefficient of permeability is given in the equation (2.2)

$$k = \left(\frac{\gamma_w}{\eta}\right)K \quad (\text{R.F.Craig 2004b}) \quad (2.2)$$

Where;  $\gamma_w$  is the unit weight of water,  $\eta$  is the viscosity of the water and  $K$  is an absolute coefficient depending only on the soil characteristics.



## 2.2.2 Determination of Permeability

### 2.2.2.1 Well pumping test (Field method)

Well pumping test is one of the most common method used for determine the permeability in the field. This method is most suitable for homogeneous coarse soil types(R.F.Craig 2004b). The procedure involves continuous pumping at a constant rate from a well (normally 300mm diameter). A filter screen placed at the bottom of the well to prevent soil particles also a casing is required to prevent collapse of well walls. Steady seepage will be started towards the well and the water table being drawn down to form a ‘corn of depression’. Water levels are observed in different boreholes located on radial lines at different distances from the well.

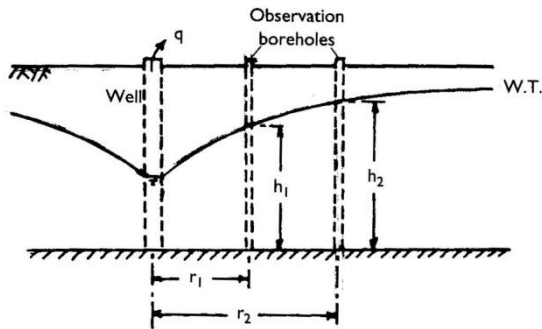


Figure 2.2: Well pumping test for unconfined soil stratum (R.F.Craig 2004b)

The figure 2.2 shows the well pumping test for an unconfined soil stratum at field. Where,  $q$  is the constant pumping rate,  $h_1$  and  $h_2$  are height of the water level in the observation boreholes;  $r_1$ ,  $r_2$  are the distance between the observation boreholes 1, 2 and the center of the pumping well respectively, and  $k$  is the permeability of the soil. From the obtains in the test, the permeability of the soil is determined by Darcy's equations(R.F.Craig 2004b).

$$q = 2\pi r h k \frac{dh}{dr} \quad (2.3)$$

$$\therefore q \int_{r_1}^{r_2} \frac{dr}{r} = 2\pi k \int_{h_1}^{h_2} h dh$$

$$\therefore q \ln\left(\frac{r_2}{r_1}\right) = \pi k (h_2^2 - h_1^2)$$

$$\therefore k = \frac{2.3q \log(r_2/r_1)}{\pi(h_2^2 - h_1^2)} \quad (2.4)$$

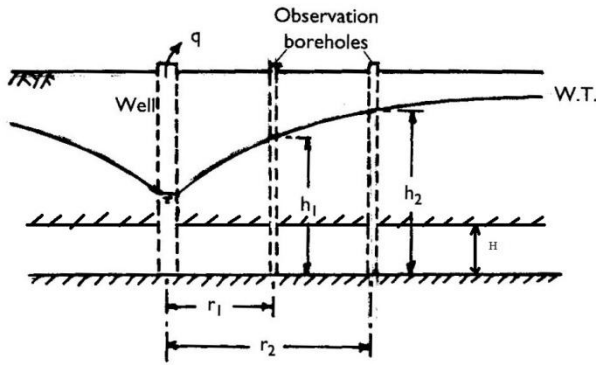


Figure 2.3: Pumping well test for confined soil stratum(R.F.Craig 2004b)

In the case of confined aquifer (Figure 2.3) the seepage take place through the thickness H. Total area of seepage is  $2\pi rH$ . For steady state pumping,

$$q = 2\pi rH \left( \frac{dh}{dr} \right) \quad (2.5)$$

Where; H is a constant and r is a variable.

$$q \int_{r_1}^{r_2} \left( \frac{dr}{r} \right) = 2\pi Hk \int_{h_1}^{h_2} dh \quad (2.6)$$

$$q \ln \left( \frac{r_2}{r_1} \right) = 2\pi Hk(h_2 - h_1) \quad (2.7)$$

$$k = \frac{2.3 q \log \left( \frac{r_2}{r_1} \right)}{2\pi H(h_2 - h_1)} \quad (2.8)$$

These procedures will be repeated with different value of  $h_1$  and  $h_2$ . The advantage of field method is more reliable than a laboratory test. Because, in the laboratory, a specimen of sample represents the whole soil mass.

#### 2.2.2.2 Laboratory methods

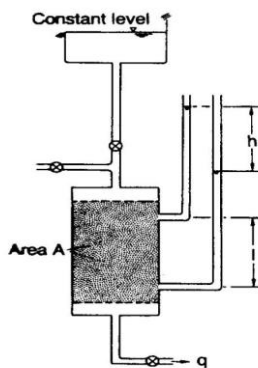


Figure 2.4 Constant head permeability equipment

The figure 2.3 shows the *constant head* permeability determination equipment at laboratory. This method is commonly used for coarse soil. A steady vertical flow of water, under a constant total head is maintained through the soil and the volume of water flowing per unit time ( $q$ ) is measured. From the Darcy's law equation, the coefficient of permeability will be determined.

$$k = \frac{Q}{At} \text{ (R.F.Craig 2004b)}$$

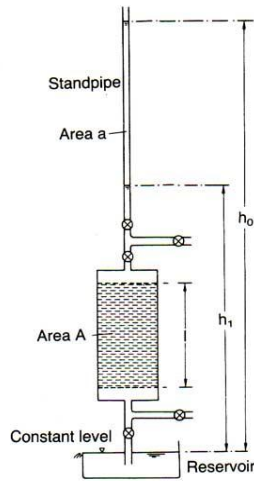


Figure 2.5: Falling head permeability test equipment

Another method called *falling head method* (Figure 2.5) is used for fine grained soil. A sample with the length of  $l$  and the cross-section of  $A$  is placed in a container and the top and the bottom of the sample end up with coarse filters. A standpipe with internal area of  $a$  is connected on top of the container. The water allowed to drain into the bottom reservoir. The time taken for the water level fall from  $h_0$  to  $h_1$  is measured ( $t'$ ). The height and the time for any in-between level are noted as  $h$  and  $t$  respectively.

By using Darcy's law;

$$-a \frac{dh}{dt} = Ak \frac{h}{l} \tag{2.9}$$

$$-a \int_{h_0}^{h_1} \frac{dh}{h} = \frac{Ak}{l} \int_0^{t'} dt$$

$$k = 2.3 \left( \frac{al}{At'} \right) \log \frac{h_0}{h_1} \tag{2.10}$$

### 2.2.3 Seepage

Seepage analysis for all structure in dam is important to detect internal erosion and designing of drainage structure to control hazards such as slides and flooding (Jackson 1997). Excessive seepage through the foundation of the dam causes the integrity of the structure. In an unconsolidated or fractured terrain when leakage velocities reach critical values, the erosion takes place in a higher rate.

This determines the importance of mapping the seepage paths and monitoring the changes in seepage as a function of time(T.V.Panthulu et al. 2000).

In general, seepage is considered in two dimensions ( $x$  and  $z$ ) for a homogeneous and isotropic soil with respect to permeability.(R.F.Craig) explaining the seepage theory for a homogeneous and isotropic soil as follows;

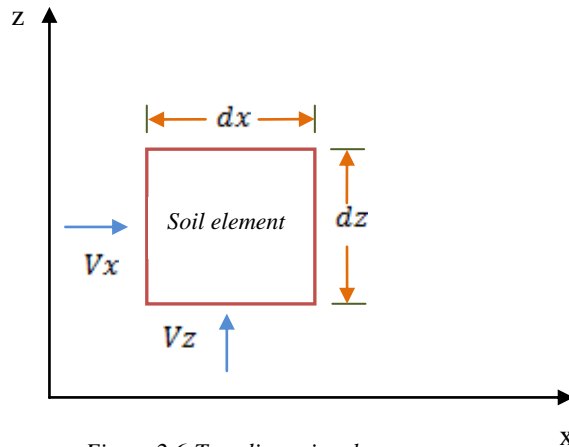


Figure 2.6:Two dimensional seepage

The figure 2.6 shows two dimensional seepage through a homogeneous and isotropic soil element with full saturation. By direct application of Darcy's law the seepage velocity through  $x$  and  $z$  directions can be written as

$$V_x = -k \frac{\partial h}{\partial x} \quad 2.11$$

$$V_z = -\frac{k \partial h}{\partial z} \quad 2.12$$

Where;  $h$  is the total head which decreasing with the directions of  $V_x$  and  $V_z$  and  $k$  is the permeability of the soil element. The total volume of water which entering the soil element per unit time will be given by the product of seepage velocity and cross-sectional area of the soil element.

$$V_x dy dz + V_z dx dy$$

As the soil element is fully saturated, the volume of water leaving from the soil element per unit time will be

$$\left( V_x + \frac{\partial V_x}{\partial x} dx \right) dy dz + \left( V_z + \frac{\partial V_z}{\partial z} dz \right) dx dy$$

Consider the soil element have no volume change and assume that the water is incompressible, and then the difference between volume of water entering and leaving per unit time is zero.

$$\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0 \quad 2.13$$

Now, let's assume the function  $\varphi(x, z)$  called as *potential function* and,

$$\left(\frac{\partial \varphi}{\partial x}\right) = V_x = -k \frac{\partial h}{\partial x} \quad 2.14$$

$$\left(\frac{\partial \varphi}{\partial z}\right) = V_z = -k \frac{\partial h}{\partial z} \quad 2.15$$

The combination of equations (2.13) and (2.14, 2.15) results

$$\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial z^2} = 0 \quad 2.16$$

Now, the function  $\varphi(x, z)$  satisfies the Laplace equation. From the integration of the equation (2.16)

$$\varphi(x, z) = -kh(x, z) + c \quad 2.17$$

Where;  $c$  is a constant

Let's assign a constant value  $\varphi_1$  for the function  $\varphi(x, z)$ , it produces a curve with a constant of total head ( $h_1$ ). When a series of constants ( $\varphi_1 \varphi_2 \varphi_3 \varphi_4 \dots$  etc) assigned for the function, a series of curves produced with constant total head which different for each curve. These curves are called as *equipotential lines*.

Now assume a second function  $\theta(x, z)$  called *flow function*,

$$-\frac{\partial \theta}{\partial x} = V_z = -k \frac{\partial h}{\partial z} \quad 2.18$$

$$\frac{\partial \theta}{\partial z} = V_x = -k \frac{\partial h}{\partial x} \quad 2.19$$

As proved earlier, that this function also satisfies the Laplace equation

$$d\theta = \frac{\partial \theta}{\partial x} dx + \frac{\partial \theta}{\partial z} dz \quad 2.20$$

$$d\theta = -V_z dx + V_x dz \quad 2.21$$

If the function  $\theta(x, z)$  gets a constant value  $\theta_1$ , then  $d\theta = 0$  and

$$\frac{dz}{dx} = \frac{V_z}{V_x} \quad 2.22$$

From the equation (2.22), it is concluded that the tangent of any point on the curve will be,

$\theta(x, z) = \theta_1$  and represent the direction of discharge at the point. This implies that the curve represent the flow path. When the function  $\theta(x, z)$  gets a series of constants ( $\theta_1, \theta_2, \theta_3, \theta_4, \dots$  etc), it generates another set of curves. These curves called as **flow lines**.

The total differential of the function  $\varphi(x, z)$  is

$$\begin{aligned} d\varphi &= \frac{\partial \varphi}{\partial x} dx + \frac{\partial \varphi}{\partial z} dz \\ &= V_x dx + V_z dz \end{aligned} \tag{2.23}$$

If  $\varphi(x, z)$  is a constant,  $d\varphi = 0$

$$\frac{dz}{dx} = -\frac{V_x}{V_z} \tag{2.24}$$

From the equation (2.22) and (2.24) we can conclude that the flow lines and equipotential lines are intersect at right angles.

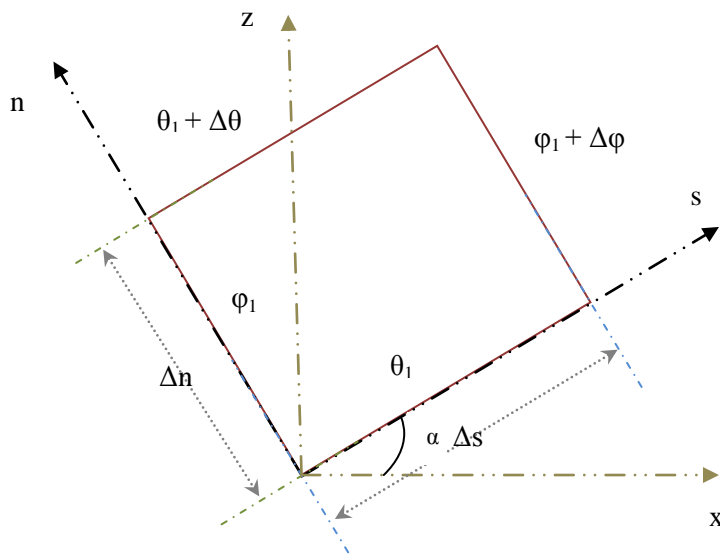


Figure 2.7: Two adjacent flow and equipotential lines

Now, consider two adjacent flow lines ( $\theta_1$  and  $\theta_1 + \Delta\theta$  separated by  $\Delta s$ ) and two adjacent equipotential lines ( $\varphi_1 + \Delta\varphi$  and  $\varphi_1$  separated by  $\Delta n$ ) those intersected at right angle. The  $n$  and  $s$  are inclined at an angle  $\alpha$  to axes  $z$  and  $x$  respectively. The seepage velocity components in the directions of  $x$  and  $z$  can be written as;

$$V_x = V_s \cos \alpha$$

$$V_z = V_s \sin \alpha$$

Also

$$\frac{\partial \phi}{\partial s} = \frac{\partial \phi}{\partial x} \frac{\partial x}{\partial s} + \frac{\partial \phi}{\partial z} \frac{\partial z}{\partial s} = V_s \cos^2 \alpha + V_s \sin^2 \alpha = V_s \quad 2.25$$

$$\frac{\partial \theta}{\partial n} = \frac{\partial \theta}{\partial x} \frac{\partial x}{\partial n} + \frac{\partial \theta}{\partial z} \frac{\partial z}{\partial n} = -V_s \sin \alpha (-\sin \alpha) + V_s \cos^2 \alpha = V_s$$

$$\text{So, } \frac{\partial \theta}{\partial n} = \frac{\partial \phi}{\partial s} \approx \frac{\Delta \theta}{\Delta n} = \frac{\Delta \phi}{\Delta s} \quad 2.26$$

The equation (2.26) is used in the flow net construction for problem solving in seepage.

### 2.2.4 Flow net construction

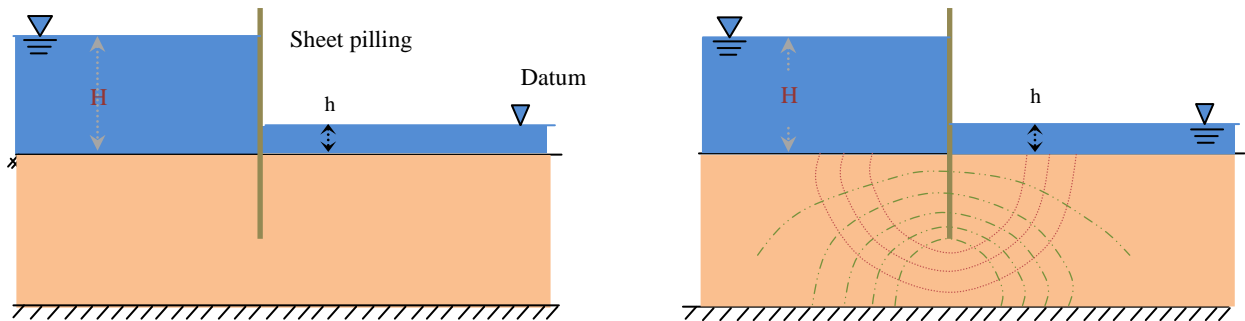


Figure 2.8: Flow net construction

Flow nets are used to solve the practical seepage problems with the functions  $\phi(x, z)$  and  $\theta(x, z)$  as boundary conditions. Finite element method or finite difference method with computer software is commonly used solution methods(R.F.Craig 2004b).

The most important condition to be satisfied that the intersection of every flow line and the equipotential line must be in right angle. The quantity of flow between any of two adjacent flow lines ( $\Delta \theta$ ) will be same and the potential drop between any of two adjacent equipotential line ( $\Delta \phi$ ) also equal. As a convenient way, it's important to define that the  $\Delta s = \Delta n$ . From the equation (2.26)

$$\Delta \phi = \Delta \theta$$

And  $\Delta \phi = \Delta q$ ,  $\Delta \theta = k \Delta h$  therefore;

$$\Delta q = k \Delta h \quad 2.27$$

The equation (2.27) indicates the flow through one particular square covered by two adjacent flow lines and equipotential lines.

For the entire flow net;

$\Delta h = \frac{h}{N_d}$  and  $\Delta q = \frac{q}{N_f}$  Where;  $N_d$  and  $N_f$  are the number of equipotential lines and flow lines respectively.

From the equation (2.27)

$$q = kh \frac{N_f}{N_d} \quad 2.28$$

Where  $q$  is the volume of water flow per unit time

### 2.2.5 Seepage modeling in *SEEP/W* Computer program

*SEEP/W* is a numerical modeling software which used to solve the practical seepage problems. This is a part of the most popular geotechnical software called GeoStudio. The *SEEP/W* program is created with the combination of *seepage theory* and *finite element* method and working on saturated/unsaturated soil region.

The practical seepage problems are never easy to convert into a numerical modeling because of the heterogeneity of the natural soils and the varying boundary condition. Generally the boundary conditions for a seepage problem never being as same as found in the initial stage. Therefore the seepage analysis in *SEEP/W* program is divided into two categories.

#### 1. Steady - state analysis

In the steady state the fundamental water flow properties such as water pressure and water flow rates never going to be changed. Practically achieving steady state is impossible. The purpose of the steady-state analysis is only to know how the initial input parameters respond to a given boundary condition.

This analysis never state that how long it takes to reach a steady state. It returns a set of solved values for water pressures and water flow parameters for particular boundary conditions. A constant pressure (H) and a constant flux rate are the important boundary conditions used for a steady-state analysis.

#### 2. Transient analysis

Transient analysis is used to know how long the embankment takes to responds for a given boundary condition. Therefore the fundamental flow properties (pressures and water flow rate) will vary with time. The analysis required an initial boundary condition as well as a destination boundary condition.



The *SEEP/W* program has an ability to read the initial condition from another analysis (may be *SEEP/W* or *SIGMA/W*) and generally obtained from a steady-state analysis (John 2010).

### 2.3 Stability analysis

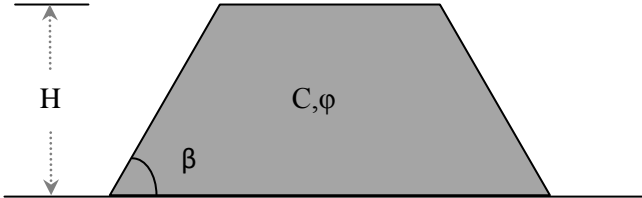


Figure 2.9: Embankment slopes

Generally the stability of a n embankment slope depend on the *height* of the slope ( $H$ ), slope *angle* ( $\beta$ ) and the shear strength parameters such as *cohesion* ( $C$ ) and the *friction angle* ( $\phi$ ). Among these three parameters, the height and the slope angle reduces the stability with respect to increased amount but, increasing shear strength parameters giving a more stable slope(Sivakugan and Das 2009).

In most of the homogeneous embankment dams failure occurred along the most *critical slide surface* with corresponding lower value of *factor of safety*.

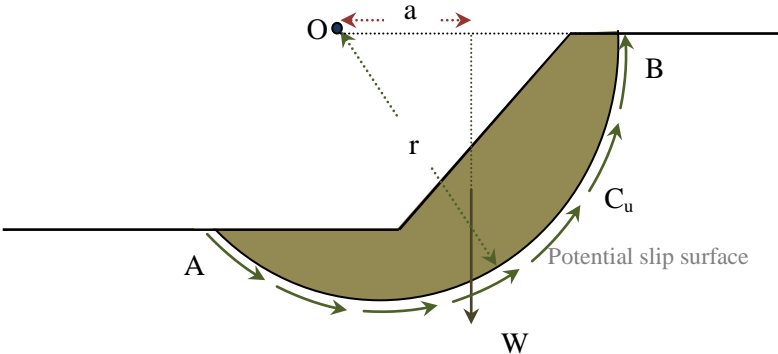


Figure 2.10: Momentum equilibrium of circular sliding surface(Duncan 1996)

Slide in soli material always has a distinct sliding surface. The sliding body moves relative to the underlying material. The figure 2.10 shows a simplified block diagram of momentum equilibrium of a circular sliding surface. The *factor of safety* ( $F$ ) of a sliding surface is defined as the number which the resistance force divided by sliding force.

$$F = \frac{C_u \cdot l}{W} \cdot \frac{r}{a}$$

Where;  $C_u$  is the shear strength along the slip surface,  $l$  is the length of the circular slip surface and  $W$  is the weight of the sliding body. When the factor of safety is less than 1, sliding will be occurred whereas a number of 1.3 to 1.5 is fairly well indicator of stability of dam(HÖEG 2001).

The forces involving in the stability equilibrium are occurred from the weight of the material, reservoir water pressure (External load), pore water pressure (Internal load), shear resistance along the sliding surface and the effective normal forces on the sliding planes(Kutzner 1997). These external and internal forces for a particular embankment is vary with time. Therefore stability analysis should be carried out for various situations. Generally this analysis made on three different stages of dam construction(Kjærnsli et al. 1992).

- a. End of construction
- b. Steady state seepage at full reservoir
- c. Rapid drawdown of the reservoir level

The shear strength is not always same in different sliding surfaces along a dam. Therefore the stability of an embankment dam must be analyzed in several sliding planes in different cross section of the dam.

### 2.3.1 Stability analysis methods

#### 2.3.1.1 Methods of slices

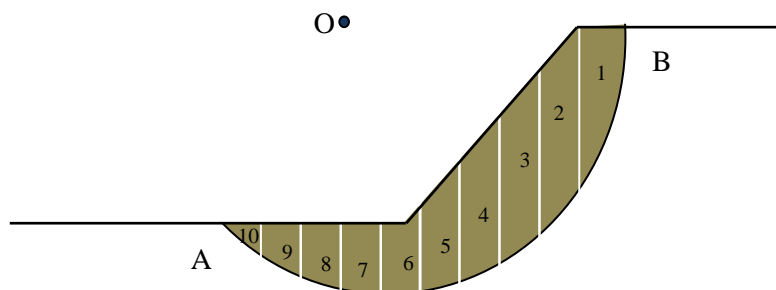


Figure 2.11: Methods of Slice(Duncan 1996)

In some cases, the sliding soil mass may not be homogeneous, i.e some part of the soil mass could included with various composites of soil. Therefore it's not possible that to apply the direct

*momentum equilibrium method* for the stability analysis. In such cases, the *method of slice* is an effective method to solve the practical stability problems.

The figure 2.11 shows a homogeneous undrained slope which several small slices for the ordinary method of slice calculation. The forces involved in the stability of a particular slice are

1. Shear strength forces ( $C_i \cdot l_i$ )
2. Self weight of the slices ( $W_i$ )
3. Pore-water pressure ( $U_i$ )

Now, consider the *normal* ( $N_i$ ) and *tangential* ( $T_i$ ) forces acting on a particular slice

$$N_i = W_i \cos \alpha - U_i \cdot l_i$$

$$T_i = W_i \sin \alpha$$

The factor of safety (F) for the entire slope is defined as,

$$F = \frac{\sum_{i=1}^n C_i \cdot l_i + \sum_{i=1}^n (W_i \cos \alpha - U_i \cdot l_i) \tan \varphi_i}{\sum_{i=1}^n W_i \sin \alpha}$$

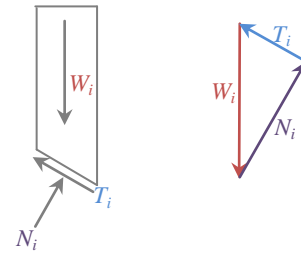


Figure 2.12: Free body diagram and force polygon for ordinary method of slice (Sivakugan and Das 2009)

Where;  $l_i$  is the arc length of the slice and the  $\varphi_i$  is the internal friction angle.

### 2.3.1.2 Bishop's modified method

This method is suggested by Professor Bishop in 1950's. The method is a modified version of the ordinary method of slice and the normal forces between interslices are included (Sivakugan and Das 2009). But Bishop did not include the shear forces between the slices and developed a new equation for the factor of safety. The new equation was a non-linear equation because, the normal force between two slices has obtained using the factor of safety hence, the equation contains the variable factor of safety in both side (Krahn 2004). Therefore an iterative method is compulsory to solve the equation. The final equation which Bishop derived is (Krahn 2004),

$$F = \frac{1}{\sum_{i=1}^n W_i \sin \alpha} \cdot \sum_{i=1}^n \left( \frac{C_i l_i + (W_i \cos \alpha - \frac{U_i l_i}{F} \sin \alpha) \tan \varphi_i}{m_\alpha} \right)$$

In the equation Bishop has included a new term  $m_\alpha$  and defined as;

$$m_\alpha = \cos \alpha + \frac{\sin \alpha \cdot \tan \varphi_i}{F}$$

In the Bishop's method the initially a value of factor of safety is necessary to start the iteration. This value always calculated from the ordinary factor of safety calculation.

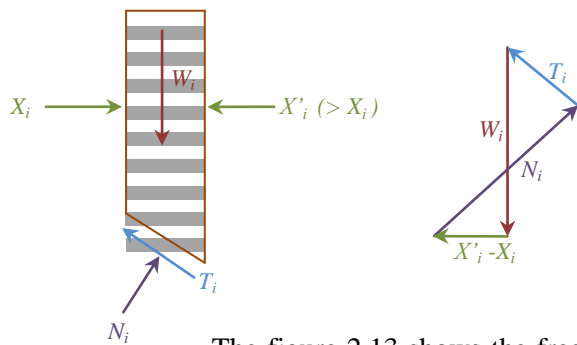


Figure 2.13: Free body diagram and force polygon for Bishop's method (Krahn 2004)

The figure 2.13 shows the free body diagram and the force polygon for the *Bishop's* modified method. Compared with the ordinary method of slice, the horizontal force ( $X_i, X'_i$ ) exerted by the adjacent slices are additionally included in both free body diagram and the force polygon. In the force polygon, the resultant horizontal force ( $X'_i - X_i$ ) is placed with the direction.

### 2.3.1.3 SLOPE/W (GeoStudio)

*SLOPE/W* is the most common and popular software application which used for the stability analysis of a slope. This is a part of GeoStudio software application. This application is created based on *limit equilibrium* method and included several types of methods like Fellenius, Bishop and Morgenstern – Price methods (Sivakugan and Das 2009). The stability analysis using *SLOPE/W* is included following components (Krahn 2004).

1. Drawing geometry
2. Defining soil properties and assigning for the corresponding soil layer
3. Defining the water table
4. Selection of analysis method (Eg: Bishop)
5. Problem solving and display the results

The results of stability analysis from the *SLOPE/W* can be obtained as both visuals and numbers. The visually interpreted results make it possible to easy understand of the results in numbers. The very important advantage of the *SLOPE/W* analysis is it allows handling all possible slides in a same model with the corresponding factor of safety.

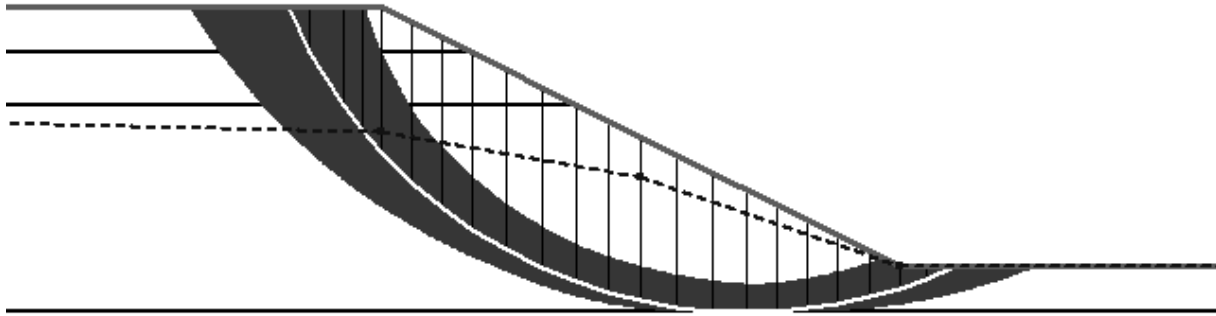


Figure 2.14: Several slip surfaces for a range of safety factor(Krahn 2004)

Figure 2.12 showing a graphically interpreted result of a stability analysis. The region filled with red color indicates the range of slip surfaces for the all possible slides. In SEEP/W it is possible to extract individual slip surfaces and their properties. When we select a particular slip surface, the corresponding factor of safety will be displayed.

## 2.4 Causes of embankment dam failure

The failure mode of an embankment dam is directly connected with the type of cause of failure and the type of the dam. Biswas and Chatterjee (1971)(Singh 1996b) examined the case of 300 dam failure and they have concluded that the 35% of the world's dam failure is caused by the direct overflow. Other 25% of failure is caused because of foundation problems such as excessive seepage, abnormal increases of pore-pressure and internal erosion. Improper design and construction caused the remaining 40% of the failure.

Incase of Årbogen dam, the direct overflow (causes the external erosion) and the seepage water (causes the internal erosion) are the main causes of failure.

### 2.4.1 External erosion

External erosion is caused by flow over embankment (overtopping).The overtopping situation is occurred when(E.Costa and L.Schuster 1988);

1. Insufficient capacity of spillway design
2. Partly or fully blocked spillway
3. Losses of storage capacity of the dam
4. Huge water displacement due to earthquake

In case of excess rainfall, the upstream water level increases instantly. When this level exceeds the maximum drainage capacity of the dam, water started to flow over embankment. This over flowing water causes the breaching followed by slide at downstream slope of the embankment as a consequence of external erosion(Kjærnsli et al. 1992).



*Figure 2.15: Damaged spillway of Årbogen dam, Norway (Pictures from site visit with NGI on 28<sup>th</sup> October 2010)*

The figure 2.15 shows the damaged spillway of a small embankment dam (Årbogen dam) in Norway. When an intense rainfall occurred, the spillway was partly blocked by suspended particles causes the increase of water level higher than the estimated probable maximum flood (PMF) level and overtopping.

### 2.4.2 Internal erosion

Internal erosion causes relatively higher number of the embankment dam failure. When compared with the external erosion, it is a long term process and several factors involved. Abnormal increases of seepage quantity and leakage of turbid water are the visual indication of ongoing erosion. In some cases, internal erosion and piping may appear similar because, the induced force is common for both that obtained from the water flow with higher hydraulic gradient(Fell et al. 2003). But, both have completely different mechanisms. Piping effect is a result from the intergranular flow of water. Internal erosion is a very common cause of embankment failure in hydraulically fractured structures such as cracks and joints(Singh 1996b).



*Figure 2.16: Traces of internal erosion at downstream side of the Årbogen dam (Pictures from site visit with NGI on 28<sup>th</sup> October 2010)*

### 2.4.3 Piping

Piping is a result of soil erosion which takes place through the embankment because of the seepage water flow(Fell et al. 2003). The water flow exerts force on particles and washes out them through an unexpected seepage discharge point. This discharge point undergoes further erosion towards upstream side and form an open like “pipe” through the embankment.

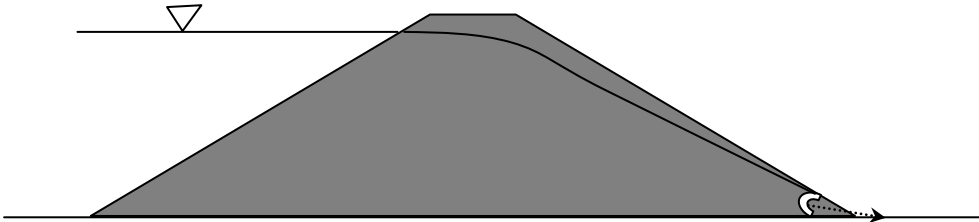


Figure 2.17 a: Initial stage of piping

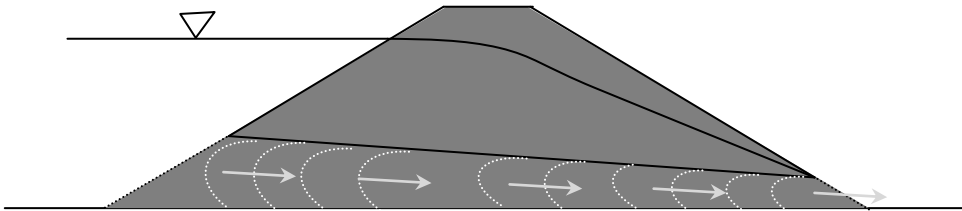


Figure 2.17 b: Erosion towards the upstream side

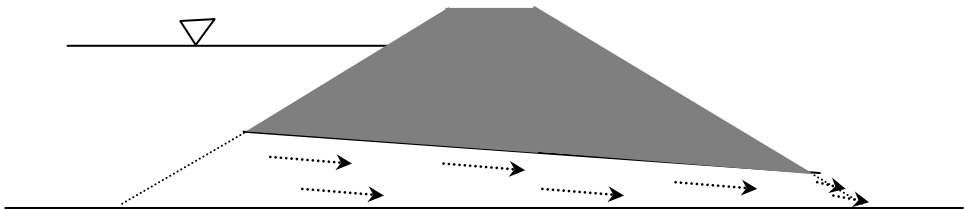


Figure 2.17c: Completed piping and created a tunnel

The figures above show the various stages of a piping process. The figure 2.15a is a initial stage of the piping. Soil masses started to wash out at the toe of the downstream slope. This erosion progresses gradually towards the upstream side (Figure 2.15b). Once the progress reached the upstream slope the tunnel will be completed and collapse may occur. After completed the tunnel, the flow water erode the

top and bottom soil in the tunnel and tends to become wider. However, there are some conditions those should be exist to initiate a piping process(Singh 1996b).

1. A flow path and a source of water
2. Hydraulic gradient should be exceeded a certain value which corresponds the embankment soil
3. The exit should be wide enough to pass the material which washed out from the embankment
4. The soil above the pipe must have enough strength to act as roof of the pipe.



# CHAPTER 3

## 3.0 RESULTS AND DISCUSSION

### 3.1 Interpretation of Drilling and samples

#### 3.1.1 Avalanche pits

On Friday 13<sup>th</sup> August 2010, there was a series of intense rainfall (62mm between 7<sup>th</sup> and 13<sup>th</sup> of August) occurred in the upstream side of the Krogstadelva, in Nedre Eiker area resulted the small old industrial dam Årbogen was overflowed (0.5m above the top). The seepage water through the deposits combined with the direct overflow caused three major and several small slides along the glaci fluvial sand deposit.

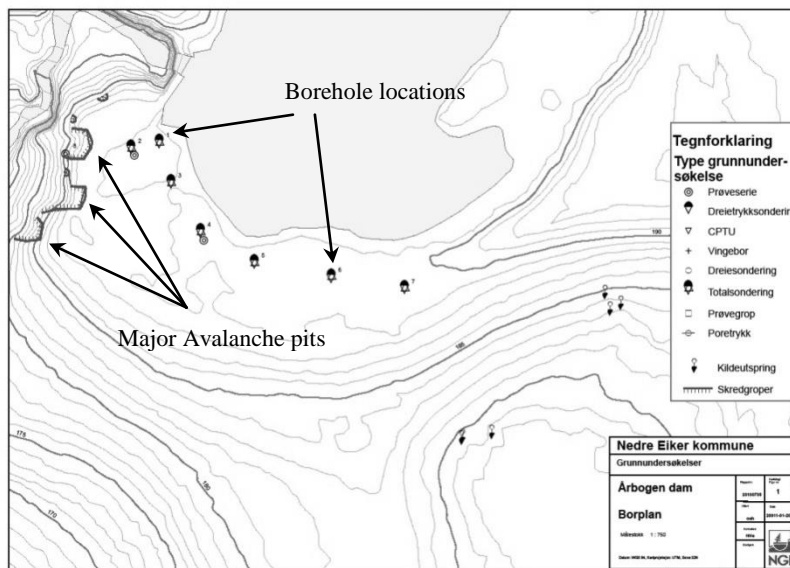


Figure 3.1 Major avalanche pits and the locations of the boreholes

##### 3.1.1.1 Pit 1

Pit 1 is the largest pit and is about 4 m depth in the back, 10 meters wide and has cut about 8 m into the soil. The pit has sloping bottom of the river and consists of ~ 2 m of gravel and stony clayed silt and fine sand (see Figure 3.2). In the lower part of the slope there are some exposed rock (red sandstone). The fine-grained layer is darker and looks from a distance like a clay layer. At the bottom of the interface occurred, there is a sandy channel. From the observation, the outflow of water continued for a long time from the sandy channel.



*Figure 3.2: Rear part of the avalanche pit 1 with sand layer at the bottom*

#### 3.1.1.2 Pit 2

Pit 2 is about 7.5 m wide and cut through the soils down to the river. It does not have a similar dark layer exists as in pit 1, but between 1-2 m depth, fine grained sand is found. There is no exposed bedrock at the upper part of the pit outlet.

#### 3.1.1.3 Pit 3

Located quite longer in southwest direction from the Pit 1 and Pit 2. The pit is about 7 m wide and of older origin. Both sides of this pit has been exposed to water flow during this event on 13<sup>th</sup> August 2010, so that fields in 2 ~ 3 m has been exposed on the sides of the pit.

During the flood, it was filled in a mass to raise the ground side so that one did not flow directly out of the deposit to the south of the dam. It is unclear to how much the direct surface flow influenced the formation of avalanche pits compared with the basic water flow. Both types of flow may influence, but the recent activity in the pit 3 is undoubtedly the only reason the water flow which has been the cause. This is also studied in the seep model.

#### 3.1.1.4 Temporary measures

During the inspection carried out by Norwegian Geotechnical Institute (NGI), it was proposed that some immediate temporary measures to be done. In oral proposal the client was agreed to keep the drainage open to maintain lower water level and to partial refill the avalanche pits. The pits were filled with gravel with an outer layer of coarser stone about 2-3 meters. Both of the layers should represent filters for the inner natural deposit.

The larger stones are sorted out and placed in the top layer with a slope of 1:2 but, the pit did not filled entirely. The refill ended at about 2m inside the toe of the avalanche pit. A rough sketch of the refill structure shown in the figure 3.3 and figure 3.4 shows a partially filled avalanche pit.

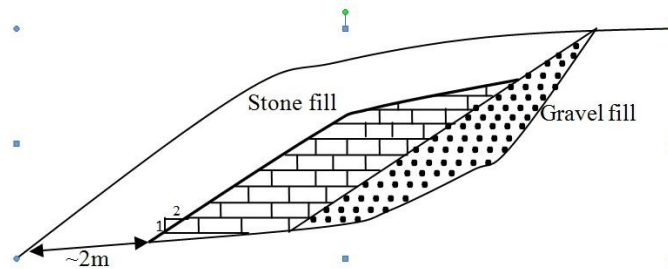


Figure 3.3: Refill structure of avalanche pit



Figure 3.3: Partially filled avalanche pit (Picture captured during the field visit on 28<sup>th</sup> October, 2010)

### 3.1.2 Boreholes

Immediately after the inspection NGI drilled 7 boreholes on the top of the deposit with rotary press sounding. Boreholes 2 and 4 were sampled by Naver drilling because of the stones and collapse of borehole walls. Figure 3.1 shows the borehole locations in a contour map. Some samples were collected and tested for grain size analysis from boreholes as well as the avalanche pits. Every

borehole were logged and described based on the description of the drilling foreman, Erlend Edvardsen, NGI (Borehole logs are attached as appendix).

#### 3.1.2.1 Borehole 1

The masses are gravel and coarse material down to the bedrock exposed and the flushing was not turned off. Flushing pressure does not increase with the increasing power supply which shows the masses are well drained. Bedrock exposed at ~ 3m and red drill cuttings came up with the spray pressure.

#### 3.1.2.2 Borehole 2

First attempt (2A) was abundant at 3.8 m deep, because of the exposed hard rock. Therefore another borehole (2B) was drilled near to the existing borehole 2A.

Borehole 2B: Gravel down to 2.4 m, then more silty - sand exposed. Flushing Water came up. When the battle turned off by ~ 2 m depth comes out a response in the spray pressure, indicating a closer and more silty - sand layer. Drill cutting of the folded rock was the first gray, then red. Naver samples down to 3 m shows grit material.

#### 3.1.2.3 Borehole 3

Gravel and soil (organic content) exposed from 0 - 2.3 m and then the sandy soil from 2.3 to 4 m. The spray pressure decreases from 3-4 m with the same flush, ie increased permeability. From 4-6 m the masses again were gravel. Bedrock turned in by 6.5 m deep. From 8.8 m rock was harder.

#### 3.1.2.4 Borehole 4

Gravelly soil and rock were down to 2 m deep. Then it was assumed silty sand down to 4 m. and coarser gravel down to 6 m with a transition to the estimated silty soils. Before the bedrock, it was believed that the moraine. Bedrock turned in at 10.6 m depth, and drilling completed at 13.9 m depth. Flushing Water did not come out of the hole being drilled. There were no large pore pressure response indicates the good drainage for whole mass.

Naver samples were taken until 4.6 m deep. The samples were predominantly gravel. From 2-3 m the sample was more sandy. From 4 to 4.6 m is the gravel sample also was more sandy. The sample flowed out of the bit by raising. Naver drill bit got stuck several times.

Both before 2m and 4m it was pre-drilled with core cutter. The hole was collapsed continuously. Therefore the samples are probably not representing the various depths.

### 3.1.2.5 Borehole 5

Gravel masses from 0 to 2.7 m deep, then drill with no stroke and flush down to 9.2 m deep. Silt or sand from 2.7 - 3, 8 m, then increased rotation down to 6.8 m; indicates gravel soil, silt response from 6.9 to 7.8, then increased rotation with rougher mass down to 9.2 m. The mass down to bedrock at 14.6 m depth characterized as moraine. Flushing water does not come up during the drilling.

### 3.1.2.6 Borehole 6

Gravel masses down to ~ 6m deep and then sand and silt sediments were exposed. Flushing pressure was increased from 10 m to 12 m indicates the increased silt content. Moraine or alternate silty gravel was occurred until 14.7m where the bedrock was exposed. Drilling was terminated at 17.1 m.

### 3.1.2.7 Borehole 7

Gravel lots from 0-9 m deep, then move to finer mass (sand, silt, or clay separate lots) from 9 to 16 m. The mother Response from 16 m, Mountain turned in at 16.8 meters. Boring terminated at 20 m depth.

There are four samples were collected from the avalanche pits and described as gravel and sand.

## 3.1.3 Particle size distribution (PSD) analysis

The particle size analysis has been done in the geotechnical laboratory at NGI. Totally seven samples were analyzed for PSD with four samples were taken from the avalanche pits and rest of them were collected from the boreholes (two samples from borehole 2 and one from borehole 4). 'Wet sieving' method was applied for all the analysis and the PSD curves were plotted for each corresponding samples.

The PSD curves are used to calculate the *Coefficient of Uniformity* ( $C_u$ ) and the results have been used to describe the samples under the standard soil classification system. The figure 3.4 shows the PSD curves for all seven samples. For easy identification, samples are named from A-G. Samples A, B and C are representing the borehole samples and D, E, F and G are representing the samples from the avalanche pits. The Calculated  $C_u$  values and the description of the samples are listed in the table 3.1

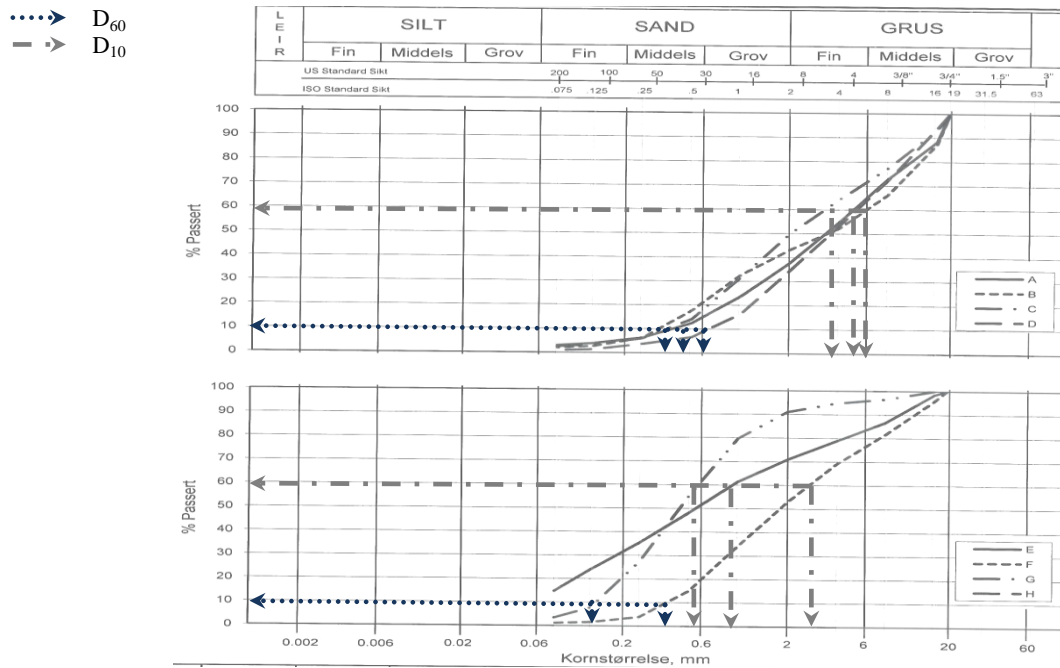


Figure 3.4: Particle Size Distribution (PSD) curves (Taken from NGI laboratory)

Table 3.1: Sample description from the PSD curve

Curve	Hole Number	Sample Number	Depth (m)	$C_u$ ( $D_{60}/D_{10}$ )	Clay Content (%)	Sample Description	Method of Sieve
<b>A</b>	4	1	0.5	12.8	-	Sandy GRAVEL	Wet
<b>B</b>	2	2	1.5	18.7	-	Sandy Gravel material	Wet
<b>C</b>	4	3	2.5	9.3	-	Sandy Gravel material	Wet
<b>D</b>	Avalanche pit	1	-	8.0	-	Sandy GRAVEL	Wet
<b>E</b>	Avalanche pit	2	-	-	-	Gravelly SAND	Wet
<b>F</b>	Avalanche pit	3	-	7.5	-	Sandy gravel material	Wet
<b>G</b>	Avalanche Pit	4	-	4.2	-	SAND	Wet

There are no curves which entered into the silt region. Therefore, no samples did contain any silt or clay particles. Most of the samples have very small amount (less than 10%) fine sand. The

samples B, C, E and F contain almost equal amount of sand and gravel. These soils are described as sandy gravel material because there is no domination of either sand or gravel. Samples A and D are dominated by gravel (more than 60%) and contain coarse sand as minor particles. These soils are described as Sand GRAVEL. Sample G has sand (almost 70%) as the major particles and which described as SAND.

Based on the *Unified soil classification* system, most of the soil samples have Coefficient of Uniformity ( $C_u$ ) above 4 and classified as *well graded* coarse soil (R.F.Craig 2004a). In case of sample E, there are no particles less than 10% of the total soil. Therefore the soil sample is not suitable for a classification under the unified soil classification system.

### 3.2 Seepage and stability analysis

Seepage is believed to be the most important cause for failure of the embankment dam Årbogen. Abnormal seepage conditions occurred during the intense rainfall and flooding effected significantly in the stability of the embankment slopes. Therefore, it is important that the stability and seepage analysis for the potential failure slopes with some extreme conditions.

First of all, three profiles were created across the embankment with enough distance between each other and then each of them was analyzed for seepage and stability with some extreme flooding condition of upstream water level. SEEP/W and SLOPE/W computer programs were used to analyze the seepage and stability conditions respectively.

The figure 3.5 shows a contour map of the Årbogen dam site with the profiles which were created for stability and seepage analysis. Profile 1 consist quiet steep slopes in both upstream and downstream sides when compared with the two other slopes. Also this profile is situated in the area where the major slides were happened.

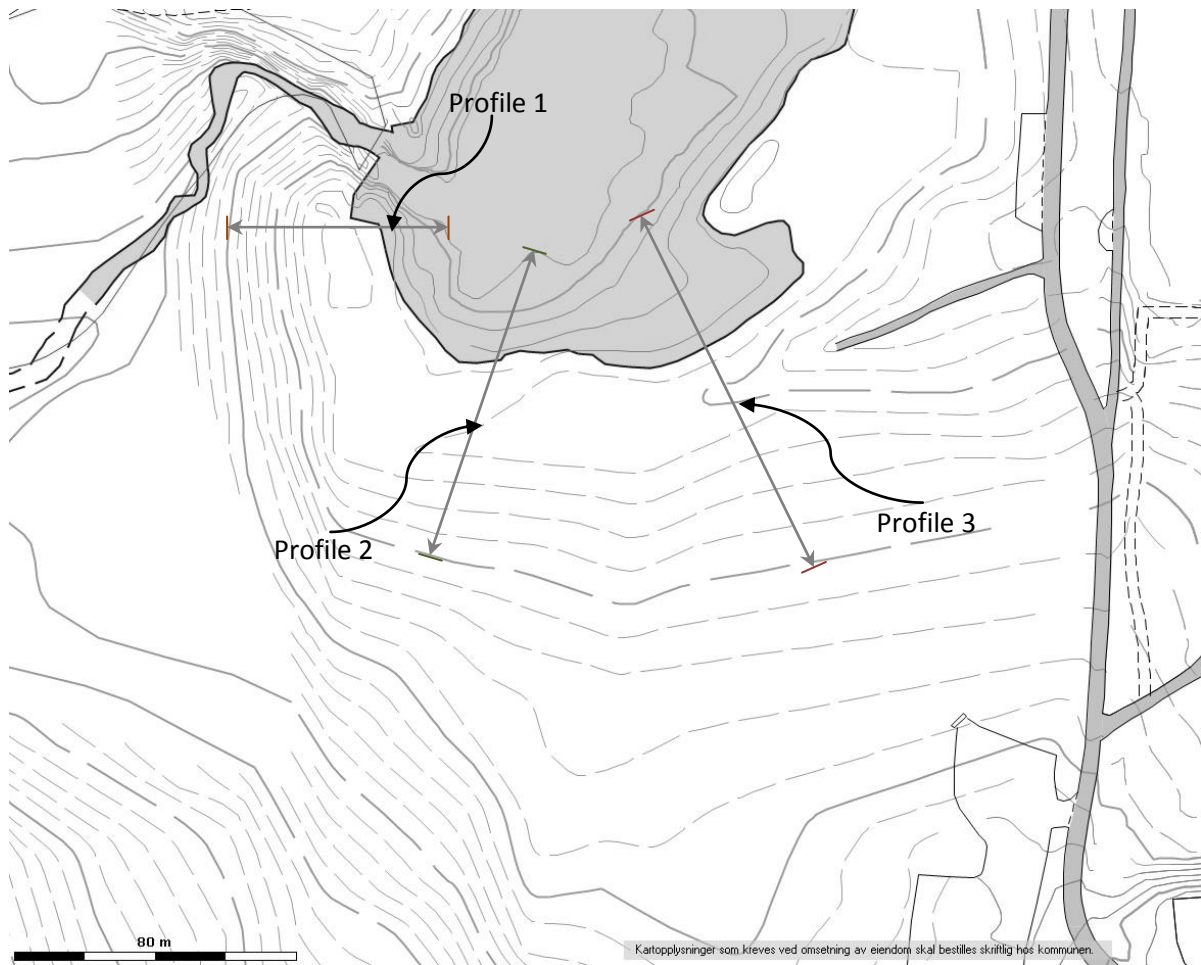


Figure 3.5: Selected profiles for stability and seepage analysis

### 3.2.1 Profile 1

Profile 1 has quiet steep slope in both upstream and downstream sides and seems a narrow profile. The length of the profile is 45m and the maximum height is 15m. The water level corresponds to the profile lays 1m below the top surface of the profile and the downstream slope is steeper than the upstream slope. This makes a large head difference between the upstream and downstream sides and which causes seepage through the embankment. Therefore it's obvious that the major slides or piping happened in this region.

At start, a steady-state analysis of seepage and corresponding stability analysis were carried out for the normal dam condition when the dam was before the extreme event with the total head as a boundary condition. The pressure and water flow conditions obtained from the steady-state analysis used as boundary conditions for the transient analysis.



### 3.2.1.1 Analysis of Normal condition

Before start the analysis of flooding condition, the normal dam conditions were analyzed with steady-state seepage analysis and the initial conditions for the transient analysis were obtained. Boundary conditions are defined by the total head along the upstream slope, zero pressure at the toe of the downstream slope and the potential seepage face. Hydraulic conductivity and the volumetric water content functions (Figures 3.7(a) and 3.7(b)) are directly imported from the *GeoSlope* resources files and the embankment soil is defined as uniform fine sand.

Stability analysis has been done with Mohr-Coulomb method and the strength parameters are defined as follows; Unit Weight = 18kN/m<sup>3</sup>, Cohesion = 5 kPa and Phi = 36°. Factor of safety is calculated using *Morgenstern-Price* method.

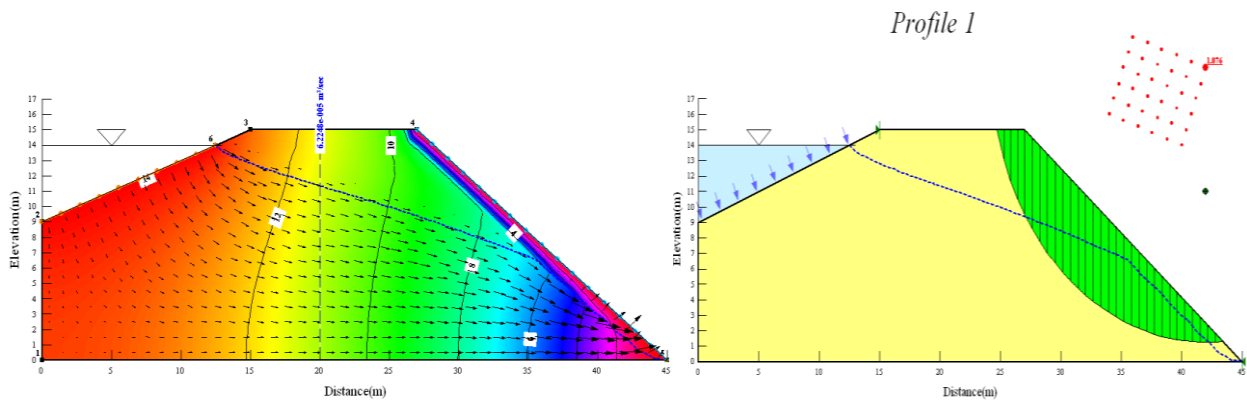


Figure 3.6(a): Steady-State seepage analysis

Figure 3.6(b): Stability analysis before flood

In the steady-state analysis, the total flux through the cross section is  $6.22 \times 10^{-5} \text{ m}^3/\text{sand}$  the factor of safety is 1.076. From the value of safety it can be concluded that the slope may failure for some extreme flood condition. Let's check in the analysis of flooding condition. Figure 3.6(a) and 3.6(b) show the visual interpretation of the results.

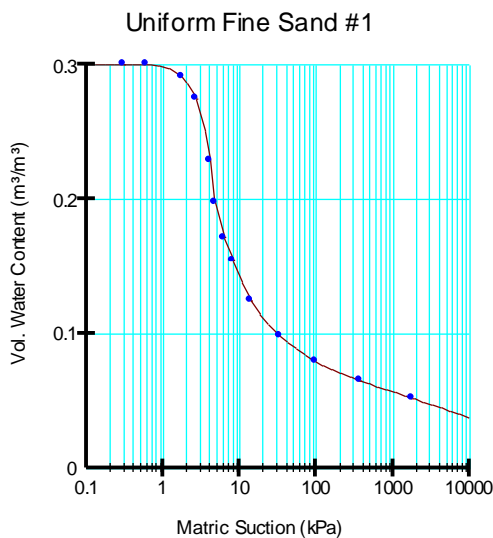


Figure 3.7(a): Volumetric water content function for uniform fine sand

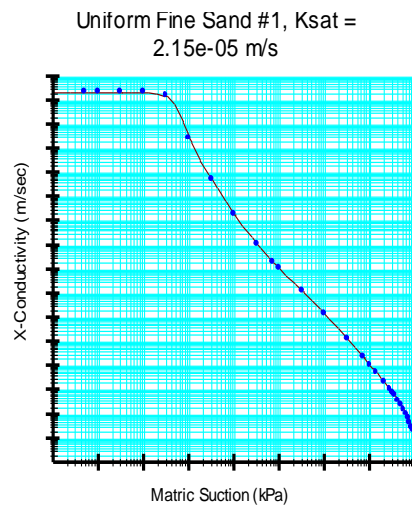


Figure 3.7(b): Hydraulic conductivity function for uniform fine sand

3.2.1.2 Analysis of flooding condition

During the flood, the water level of the dam is assumed 2m above the normal conditions (16m, but the figures below show only the normal water level) and the front of the water table extend up to the point 7 which lies on the crest of the dam. Therefore the boundary condition defined by the pressure head is extended by adding the lines 6-3 and 3-7 with the line 2-6 which represents the normal condition (see figure).

In a particular analysis, the GeoSlope program allows to import the results which obtained from another analysis result to define the functions as well as the boundary conditions. So, the transient analysis could be done based on the steady-state analysis as the parent analysis. Therefore the pressure head and the pore water pressure at each node which obtained from the steady-state analysis are transferred to the transient analysis as the boundary condition. The properties of the soil such as permeability and the volumetric water content which defined in the steady-state analysis also imported to the transient analysis.

Initially the time duration for the analysis was defined as 98 days with 25 time steps and the time increment was selected as exponential manner. Every time step in the model was saved and the times which corresponding to the significant changes in the flow properties were taken as the results.

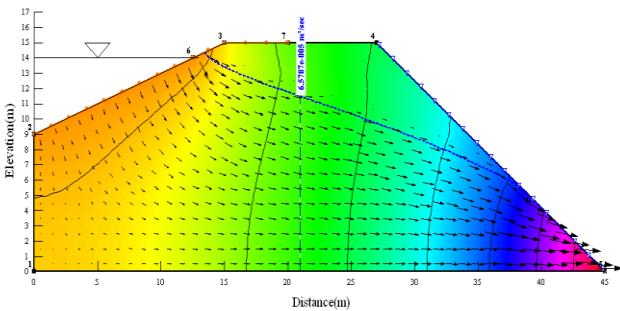


Figure 3.8(a): Seepage condition after 3 days and 21 hours of flooding

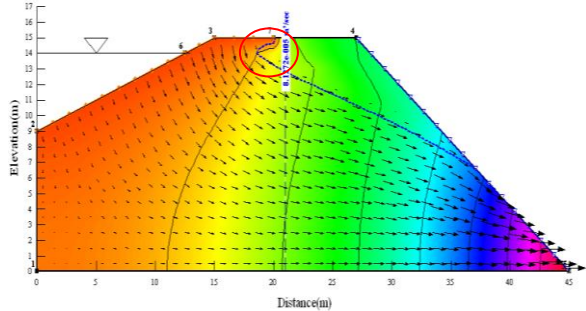


Figure 3.8(b): Seepage condition after 11 days and 16 hours

Figures 3.8(a) and 3.8(b) shows the seepage condition at 3 days and 21 hours and after 11 days and 16 hours of flooding respectively. The reservoir level indicated in the figures is the water level corresponds to the normal conditions. A new water table is formed at the initial stage as a result of the combination of flooding water level in the dam and the water table at normal dam condition (before flooding).

After 3 days and 21 hours the water table is increased ~0.5m and after 11 days and 16 hours it increased to 1m above the normal water table and interact the top of the profile at point 7 (figure 3.8(b)). A curve towards the upstream side is observed because of the quick saturation of the top layer.

Between the above time intervals the water flux increased from  $6.578 \times 10^{-5} \text{m}^3/\text{s}$  to  $8.1272 \times 10^{-5} \text{m}^3/\text{s}$ . it shows the seepage through the deposit is increased because of the increasing head difference.

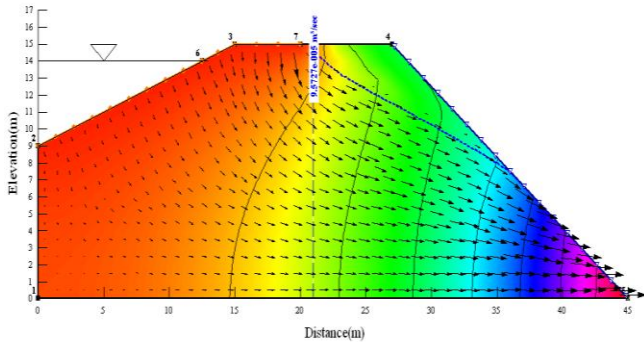


Figure 3.8(c): Seepage condition after 15 days and 13 hours of flooding

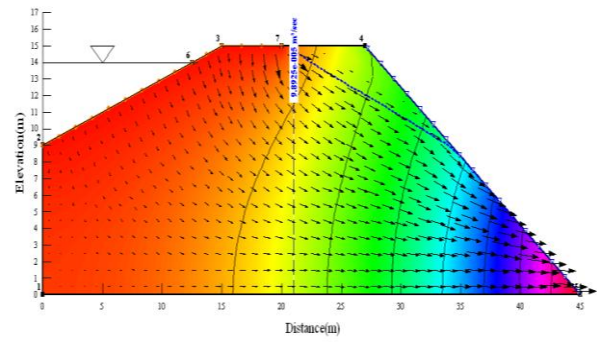


Figure 3.8(d): Seepage condition after 35 days of flooding

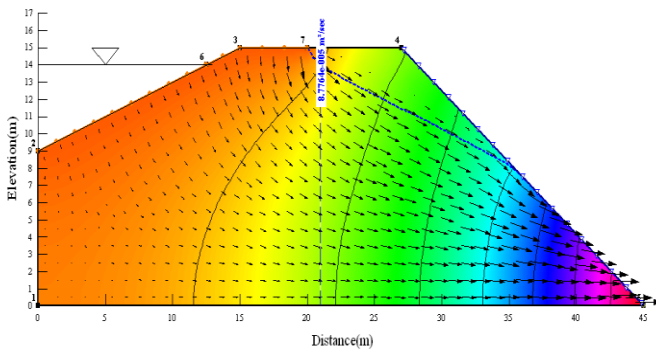


Figure 3.8(e): Seepage condition after 38 days and 31 hours of flooding

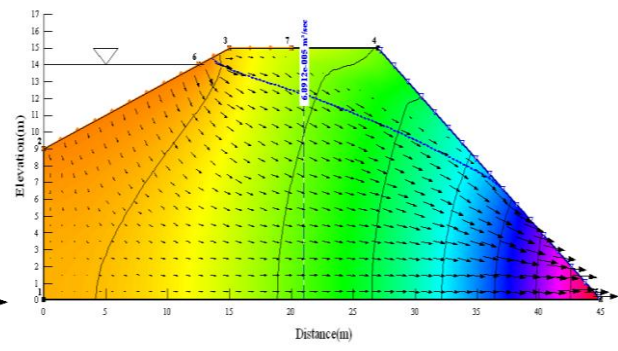


Figure 3.8(f): Seepage condition 77 days and 18 hours of flooding

Figures 3.8(c) to 3.8(f) show the seepage conditions for various time periods after the flooding. The water table tends to elevate continuously until a certain time period and then lowered to initial position. The seepage through the deposit also shows same variation as the water table. These changes happened because of the increasing head at the upstream side. Forward and backward movement of the total head contours clearly indicates the pressure changes.

The total flux changes summarized in the table 3.2

Table 3.2: Flow properties for different time periods

Time Period	Total Flux ( $\times 10^{-5} \text{m}^3/\text{sec}$ )
3 days and 21 hours	6.478
11 days and 16 hours	8.127
15 days and 13 hours	9.573
35 days	9.892
38 days and 21 hours	8.776
77 days 18 hours	6.891

After 3 day and 21 hours of flooding, the water flux was just above the water flux which corresponds to the initial condition. After a certain period it reached a maximum and then started to reduce.

It is because that during the intense rainfall, the head at the upstream side increased rapidly caused an abnormal increase in the seepage. Therefore the normal steady – state condition of the dam has been lost. Once the rainfall disappeared, the dam intends to reach the steady-state condition back. When the steady-state reached, the total flux never changes with the time.

The *SEEP/W* program helps to analyze the various pressure conditions, flow conditions and the changes in the material properties at any point or region of the embankment. The pressure condition could be analyzed in different forms such as total pressure, pressure head, pore-water pressure and the hydraulic gradient (Y gradient) separately. Here, some nodes from the geometry item called *potential seepage face* have been selected for the analysis.

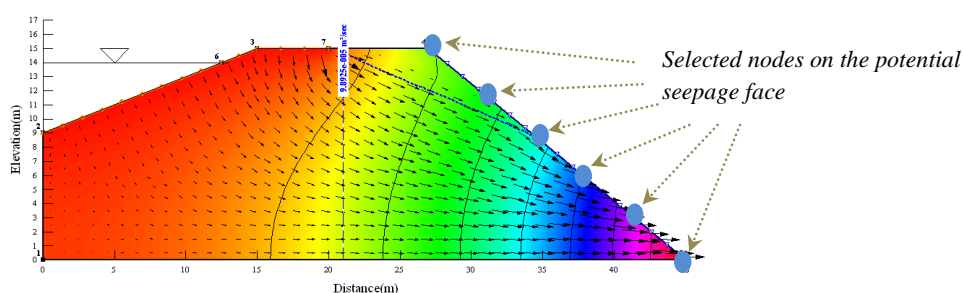


Figure 3.9: Selected nodes on the potential seepage face

Figure 3.9 shows the nodes on the potential seepage face which are selected for the analysis of pressure variation. *SEEP/W* program allows generating the graphs with distance and time as independent variables.

### Pore-water pressure Vs Time

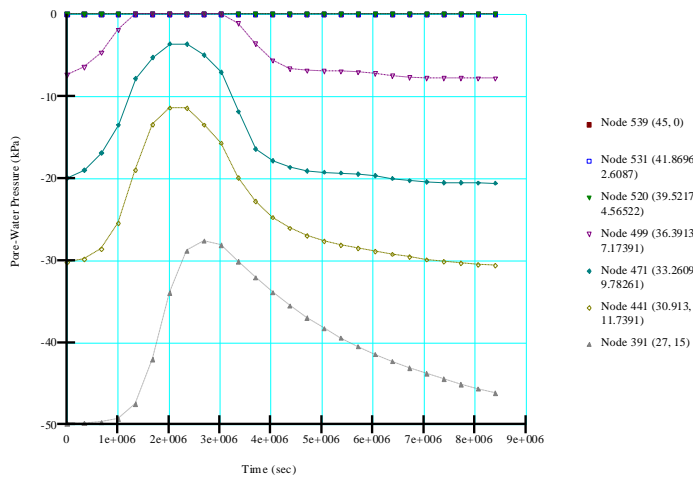


Figure 3.10: Pore-water pressure changes with time at potential seepage face

Pore – water pressure changes with time in 7 nodes on the potential seepage face is shown in the figure 3.10. Pore-water pressure changes all nodes are similar way. The node at the bottom of the geometry (node 539) always has higher pore-water pressure than other nodes, because the node is saturated at all. But the increasing seepage caused an elevation in the graph. The pore-water pressure which corresponding the other nodes will be below at any time of the analysis and arrange themselves according to the degree of saturation at each nodes.

### Total Head Vs Distance

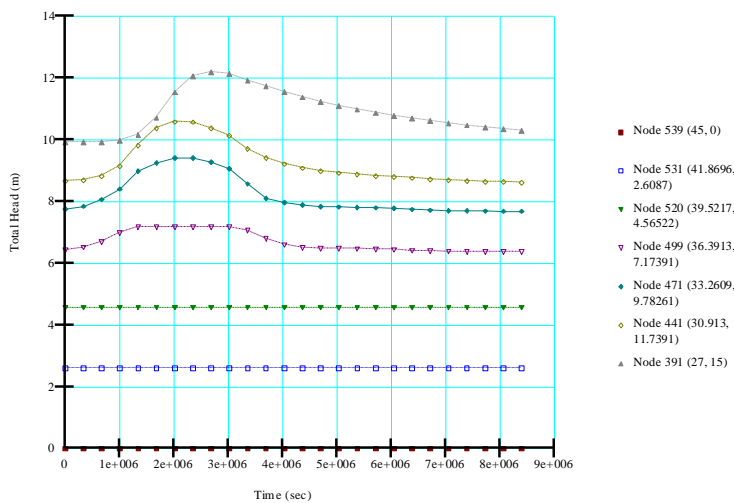


Figure 3.11: Total head changes with time and distance at potential seepage face

Figure 3.11 shows the changes of total head with time and distance at same nodes on the potential seepage face. The node at the bottom of the downstream slope (node 539) is defined as a boundary condition with zero pressure. Therefore the total head at the node 539 is always zero. Total pressure at the nodes 531 and 520 are not change with time because; it always being below the water table. At all other nodes, the total pressure continuously increase with time until the water table reaches to the maximum and then started to reduce.

Another important thing to be analyzed in the SEEP/W program is material properties. The volumetric water content is one of the most important material properties in a seepage problem. There are 6 nodes (different from previous) selected on the potential seepage face and analyzed for material properties. Figure 3.13 shows the change of water flux with time on the selected nodes.

Volumetric Water content VS Time

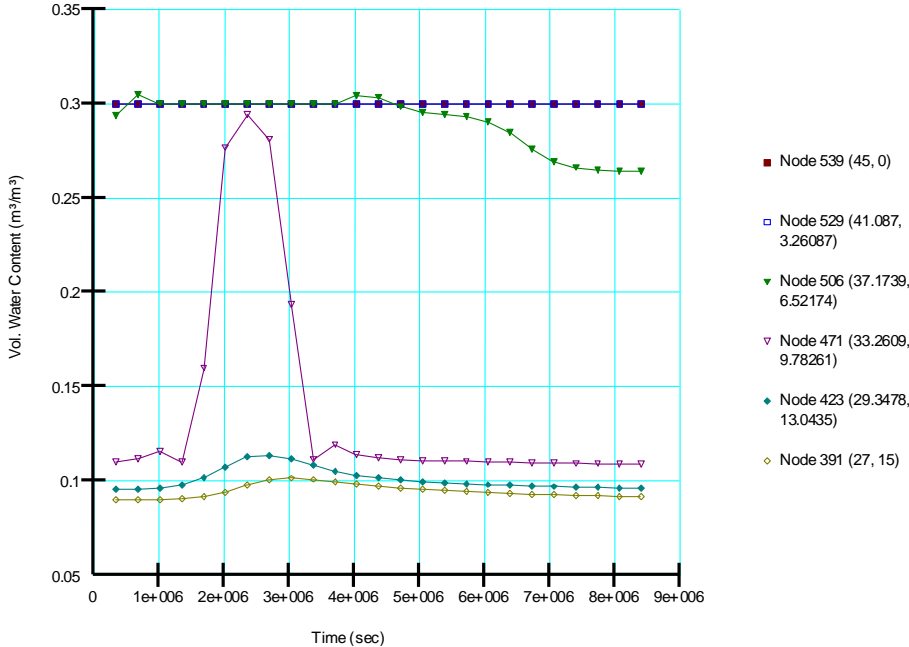


Figure 3.13: Change of volumetric water content with time

The figure 3.13 shows the changes in the volumetric water content of the soil at selected nodes. Nodes 506 and 529 are representing the soils which have higher volumetric water content. Because the nodes always lay below the water table, hence they are fully saturated. Similarly, the nodes 423 and 391 are always above the water table and hence, partially saturated. A large changed observed in the node 471 because; the water table raised above the node at a time and the soil become fully saturated.

### Y gradient (Hydraulic gradient) Vs Distance

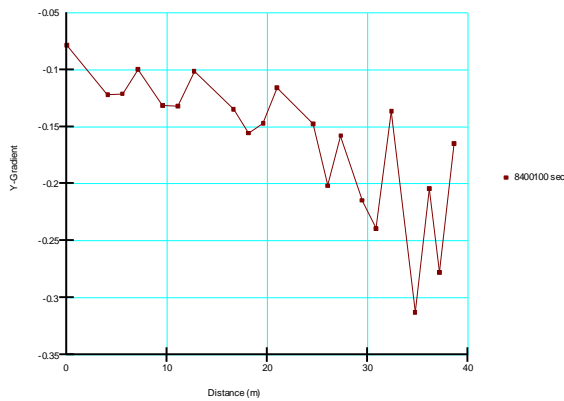


Figure 3.14: Change Y gradient with distance at the end of the flood

The figure 3.13 shows the changes in Y gradient (hydraulic gradient) with distance at the end of the flooding. In the upstream slope of the deposit (distance 0), the changes in the Y gradient is low and increases with the increasing distance. When it approaches to the downstream slope, the variation in the Y gradient is very high. The reason for the increases could be the widening of the flow path towards the downstream side. This indicates the increased erosion and the the initiation of piping in the downstream slope.

During the transient seepage analysis, we have seen that the pressures and the material properties have different values for different time periods. Therefore the stability of the embankment should be analyzed for some different time periods in order to find possible slides during the flood. The stability analysis has been done by *SLOPE/W* together with the *SEEP/W* (transient analysis) in a same project. The material properties and the pore-water pressure conditions are assigned from the transient analysis for the particular time period.

The stability has been analyzed for four different time periods after the flood. They are 30 minutes, 4 days, 33 days and 97 days (last time period). All the analysis carried out based on *Morgenstern-price method*. The factor of safety was obtained for each analysis with keeping the same slip surface for all analysis. Stability conditions for each time period are shown in figures 3.14(a)-3.14(f)

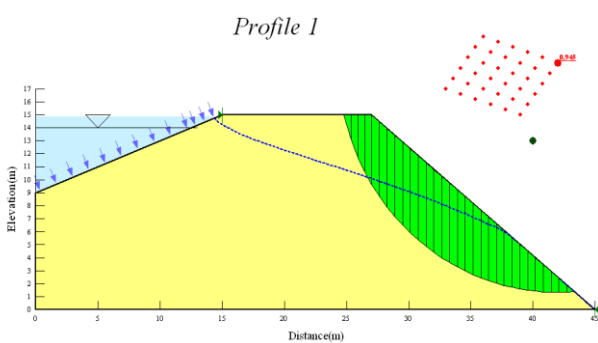


Figure 3.15(a): Stability analysis after 3 days and 21 hours

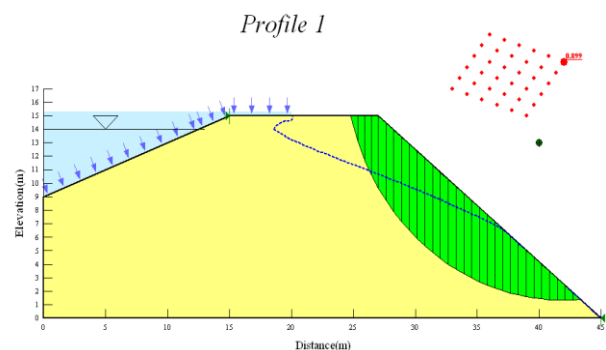


Figure 3.15(b): Stability analysis after 11 days and 16 hours

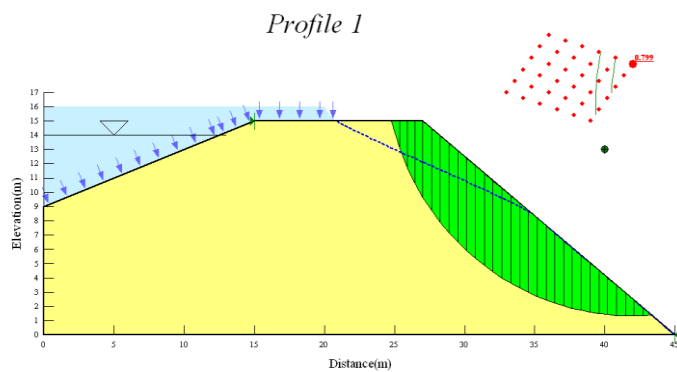


Figure 3.15(c): Stability analysis after 23 days and 8 hours  
Profile 1

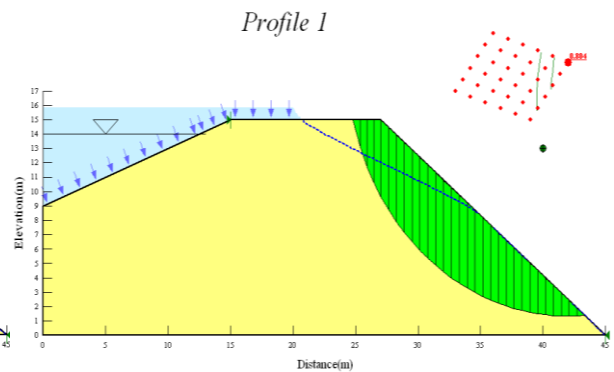


Figure 3.15(d): Stability analysis after 27 days and 5 hours  
Profile 1

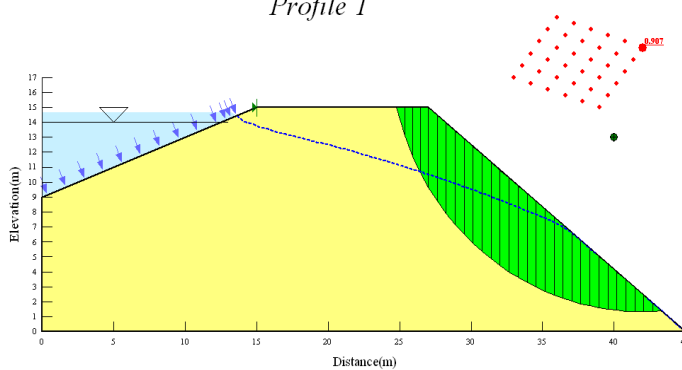


Figure 3.15(e): Stability analysis after 38 days and 21 hours  
Profile 1

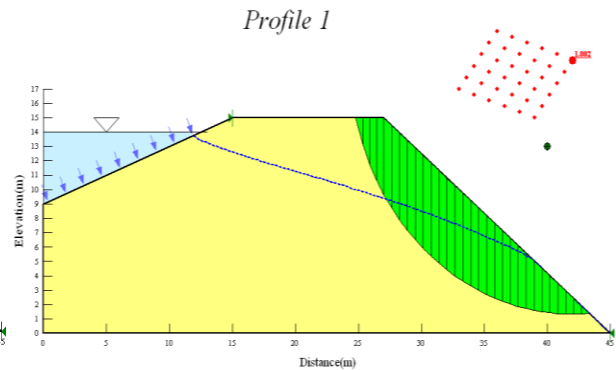


Figure 3.15(f): Stability analysis after 97 days and 5 hours  
Profile 1

Table 3.3: Factor of safety for all stability analysis

Time	Factor of safety	Method
3 days and 21 hours	0.945	Morgenstern-price method
11 days and 16 hours	0.899	Morgenstern-price method
23 days and 8 hours	0.799	Morgenstern-price method
27 days and 5 hours	0.804	Morgenstern-price method
38 days and 21 hours	0.907	Morgenstern-price method
97 days and 5 hours	1.002	Morgenstern-price method

Table 3.3 shows the factor of safety for each analysis. The factor of safety is decreases until the increase of water table and then increasing until the end of the analysis. The results show that the



slope is potentially unstable throughout the flooding. It is because the increasing pore-water pressure reduces the shear strength of the soil and the saturation of water reduces the frictional strength.

The results proved that the deposit is potentially unstable in theoretically. But some other situations at the field may increase the possibility of failure. The suspected factors may be

1. Complexity of the slope geometry
2. Uneven characteristics of the embankment soil
3. Presence of small cracks along the slope
4. Piping through the deposit

The factor mentioned at last may play an important role in the slope stability. Because there are some number of such springs were observed during the field visit to the dam site. The figure (3.15)

shows a water spring at the field.



*Figure 3.15: A water spring observed around the dam site*

### 3.2.2 Profile 2

Profile 2 has a quiet gentle slope on the downstream side; selected little straight down the dam (See Figure 3.5) and located apart from the major slides that happened during the flood. The upstream water level in the dam is 3 m and the top of the embankment is 2m above the water level. So, the head difference between the upstream and downstream is small, resulting in less seepage throughout the profile and the downstream slope seems to be more stable even in most critical situations.

As same as the profile 1, profile 2 also analyzed for a transient analysis of seepage and some stability analysis of different time periods. Selection of material properties has been done in the same way as in the profile 1.

3.2.2.1 Seepage analysis

Seepage analysis has been done only for flooding conditions. For the flooding condition, the water table of the upstream side is selected 2m above the normal water table of the dam. The water table corresponds to the flooding condition meets the upstream slope at 1m above the normal water table. Functions for the hydraulic conductivity and the volumetric water content are imported from the *GeoSlope* resource files.

The analysis consists of total time period of 28 hours with 25 time steps and the time increment is selected in exponential manner. Pressure head along the upstream slope and the potential seepage face are selected as boundary conditions. Seepage and pressure conditions are analyzed in 5 different time periods. They are; initial stage, after 3hours and 30minutes, 7hours and 20minutes, after 14hours and 25minutes and after 28hours (final stage).

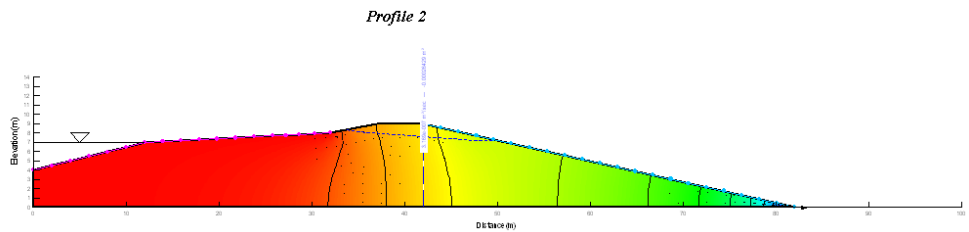


Figure 3.16(a): Seepage conditions at initial stage

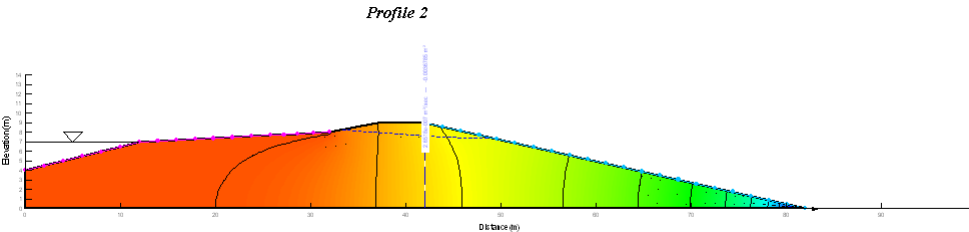


Figure 3.16(b): Seepage conditions after 3 hours and 30 minutes

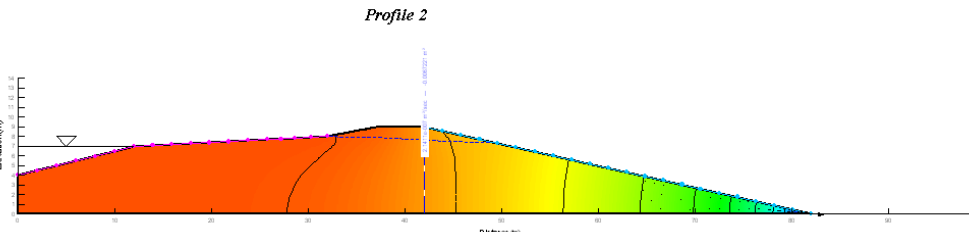


Figure 3.16(c): Seepage conditions after 7 hours and 20 minutes

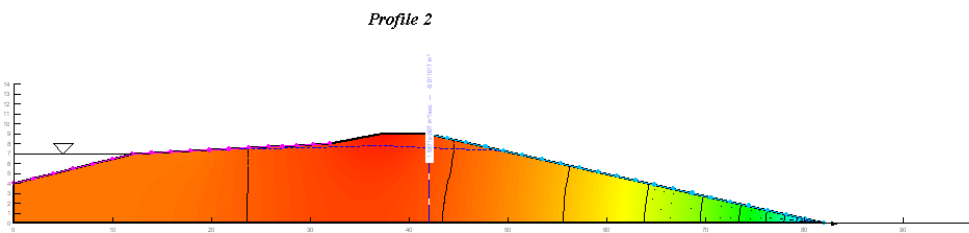


Figure 3.16(d): Seepage conditions after 14 hours and 25 minutes

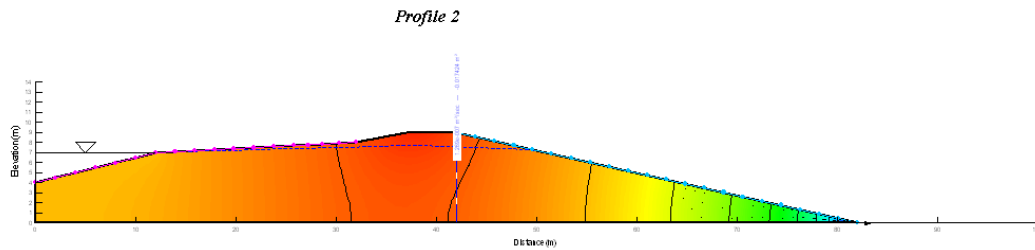


Figure 3.16(e): Seepage conditions after 28 hours (Last)

Table 3.4: Total flux and the total discharged volume with increasing time

Time period	Total flux ( $\times 10^{-7} \text{ m}^3/\text{sec}$ )
Initial	3.168
3 hours and 30 minutes	2.652
7 hours and 20 minutes	2.141
14 hours and 25 minutes	1.580
28 hours	1.289

The figures 3.16(a) – 3.16(e) shows varies stages of transient seepage analysis. From the initial stage, the water table is lowering and the total flux through the embankment is decreasing with increasing time. These results are summarized in the table 3.4

Table 3.4 shows the changes of total flux and the total volume of water discharged by seepage for different time periods. As mentioned in the analysis of profile 1, the decreasing water table at the upstream side of the dam causes the continuous reduction in the seepage. But, the changes in the profile 1 are very small in numbers when compared with the changes in the profile 2. The possible reasons are

1. Total head in the profile 1 is higher than the profile 2
2. As a narrow profile, the length of the seepage path is smaller comparing with profile 2
3. Profile 1 is located close to the spillway and the dam is quiet deep near to the spillway when compared with other area. Therefore the movement of flood water will be more towards profile 1, than profile 2

The changes of total head, pressure head and the pore water pressure with the time is represented by the figures 3.17, 3.18 and 3.19 respectively. There are 6 nodes (nodes 570, 548, 511,

470, 426 and 356) on the potential seepage face have been selected for the analysis of pressure changes. But, we can observe that there are no huge changes in the pressure conditions throughout the time as we observed during the analysis of profile 1.

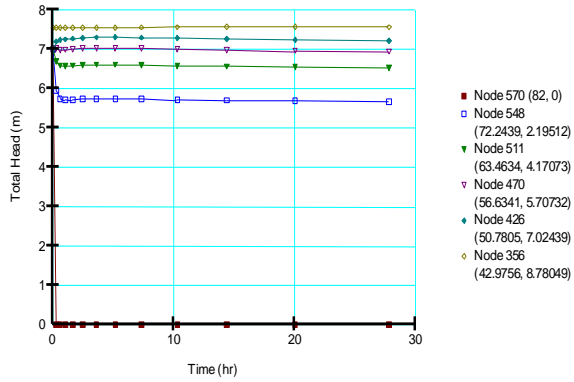


Figure 3.17: Total head Vs Time

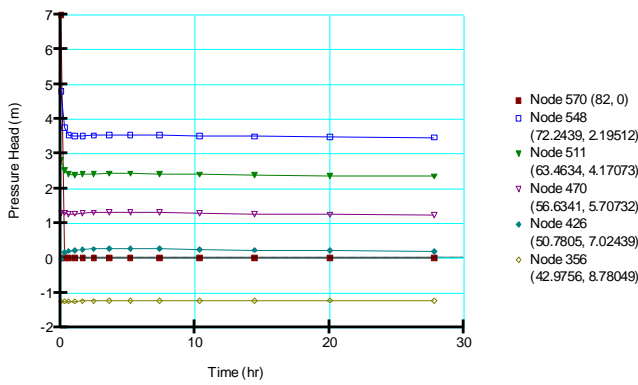


Figure 3.18: Pressure head Vs Time

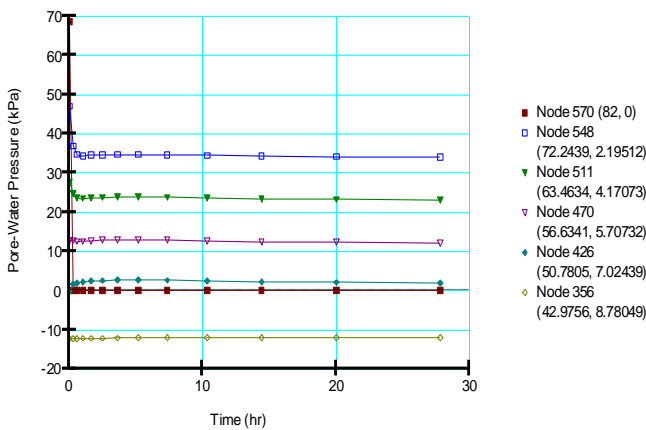


Figure 3.19: Pore-water pressure Vs Time

3.2.2.2 Stability analysis

The results of the seepage analysis show that the seepage and pressure conditions are not changing so much with the time. Changes have observed within a short period after the flooding. Therefore the stability analysis is not really required to do often. So, stability has been analyzed in both initial and final stage of the seepage analysis.

Stability analysis has been done with Mohr-Coulomb method and the strength parameters are defined as follows; Unit Weight = 18kN/m<sup>3</sup>, Cohesion = 5 kPa and Phi = 36°. Factor of safety is calculated using *Morgenstern-Price* method. Figures 3.20 (a) and 3.20 (b) show the results of stability analysis of initial and final stage of the flooding condition respectively.

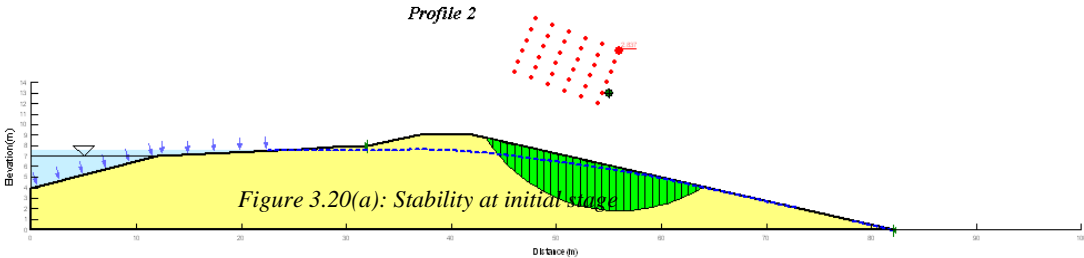


Figure 3.20(b): Stability at initial stage

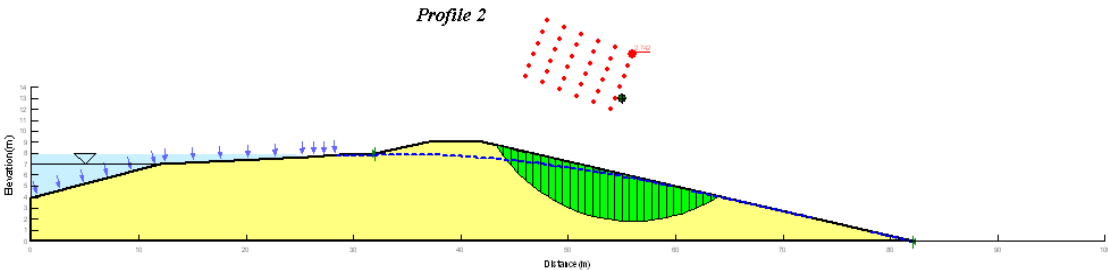


Figure 3.20(b): Stability at final stage

Table 3.4: Factor of Safety for stability analysis of profile 2

Time	Factor of Safety	Method
Initial	2.742	<i>Morgenstern-Price</i>
Final	2.837	<i>Morgenstern-Price</i>

The factor of safety values obtained from the stability analysis of profile 2 shows that the slope is extremely stable at all.

3.2.3 Profile 3

Profile 3 has the longest and very gently downstream slope when compared with other 2 profiles (42m length and 7m height). The profile has the maximum height of 7m and the water table corresponds to the normal dam condition lies 1m below the top surface of the profile. Figure 3.5 shows the exact location of the profile 3.

As usual a transient seepage analysis and stability analysis have been done for the profile. Material properties selected as same as the previous profiles.

3.2.3.1 Seepage analysis

Seepage analysis has been done only for flooding conditions. The water table for the flooding condition is selected 2m above the normal water table. The new water table meets the upstream slope at the top surface of the profile. Again the functions for the hydraulic conductivity and the volumetric water content are directly imported from the *GeoSlope* resource files.

The analysis extends to a maximum time period of 11 days and 12 hours with 25 time steps and the time increment is selected in exponential manner. Pressure head along the upstream slope and the potential seepage face are selected as boundary conditions. Lowering water table with time is defined as a function and added to the boundary conditions. Seepage is analyzed for 6 different selected time period. They are; initial stage, after 9hours, after 21hours, after 2days and 2hours, after 4 days and 21hours and after 11 days and 12hours (final stage). Figures 3.21 (a) - 3.21(f) shows seepage conditions visually.

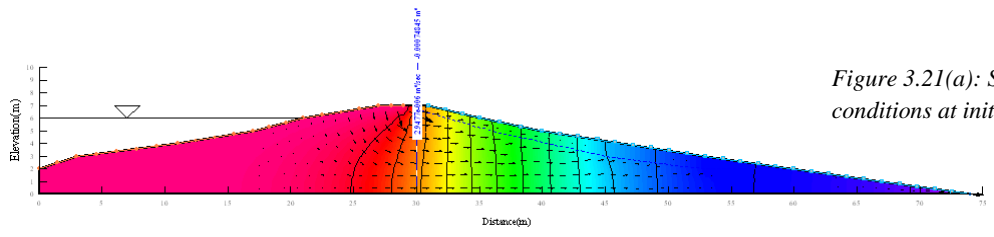


Figure 3.21(a): Seepage conditions at initial stage

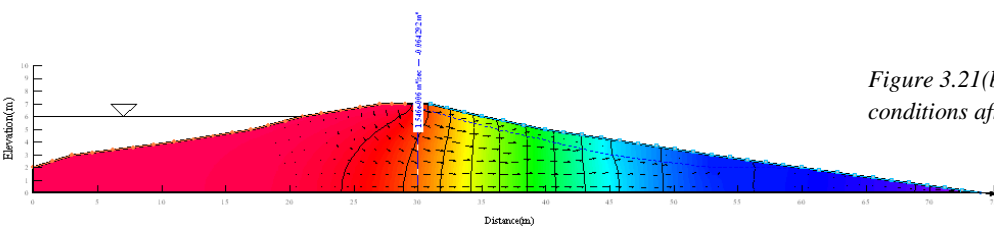


Figure 3.21(b): Seepage conditions after 9 hours

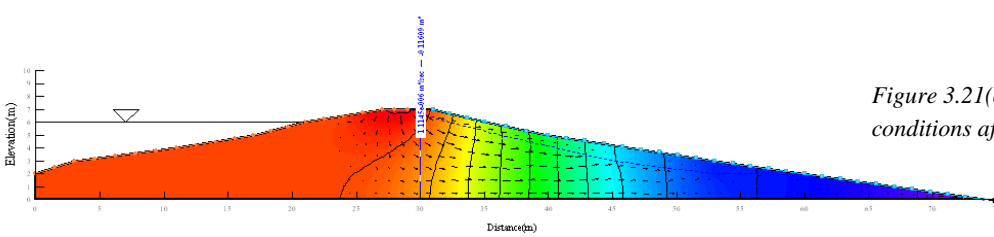


Figure 3.21(c): Seepage conditions after 21 hours

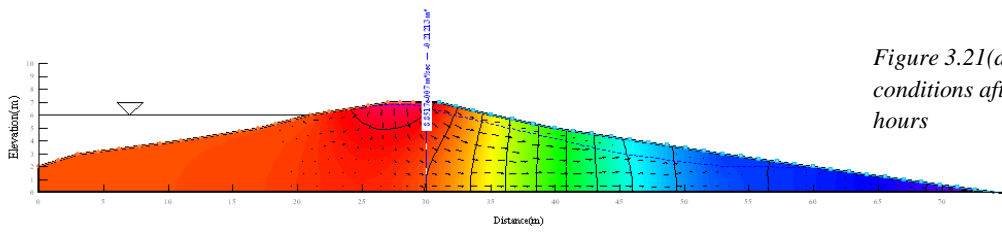


Figure 3.21(d): Seepage conditions after 2 days and 2 hours

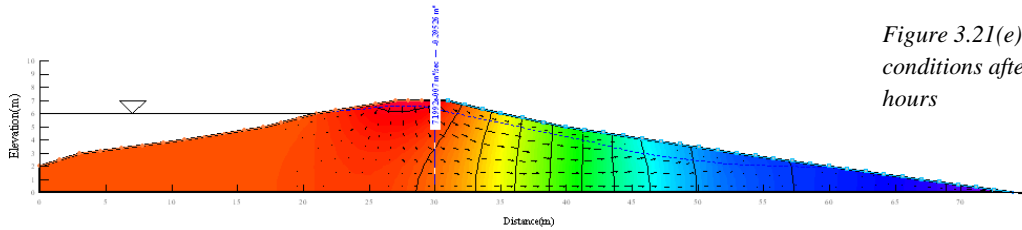


Figure 3.21(e): Seepage conditions after 4 days and 21 hours

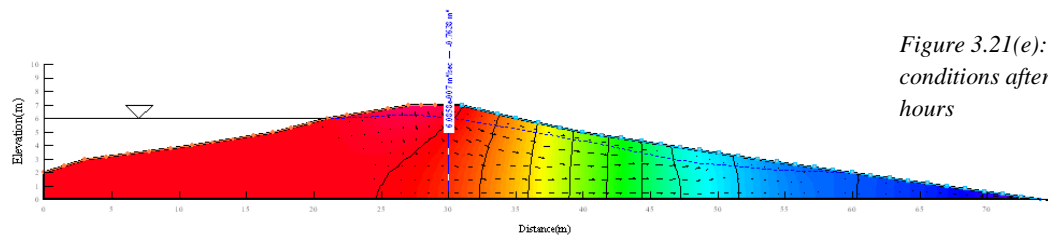


Figure 3.21(e): Seepage conditions after 11 days and 12 hours

From the visual interpretation of the results, we can clearly see that the water table lowering with increasing time. Changes of total flux and the total volume of water discharged with the time is listed in the table 3.5

Table 3.5: Total flux through the profile31

Time period	Total flux( $\times 10^{-7} \text{ m}^3/\text{sec}$ )
Initial	29.477
After 9 hours	15.46
After 21 hours	11.14
After 2 days and 2 hours	8.852
After 4 days and 21 hours	7.199
After 11 days and 12 hours	6.086

The total flux through the embankment continuously reducing with increasing time and is very low when compared with the flux through the profile 1. Total volume of water discharged also very low even after 11 days of seepage. It shows that the flooding doesn't affect the seepage condition so much. The analysis of pressure and flow changes may give more information about the seepage condition throughout the flood.

Pressure and flow properties have been analyzed along the potential seepage face. There are 8 nodes (node 365, 348, 332, 307, 280, 248, 223 and 187) on the potential seepage face have been selected and the properties change with time plotted graphically. Figure 3.22 shows the selected nodes on the potential seepage face and the figures 3.23(a) – 3.23(c) illustrate the pressure changes.

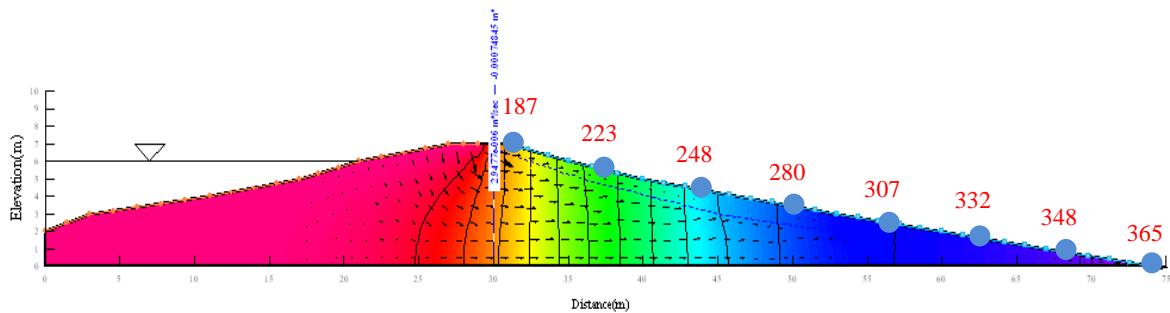


Figure 3.22: Selected nodes on the potential seepage face

### Pressure Head Vs Time

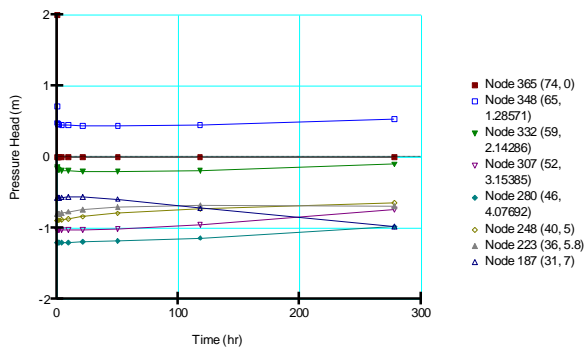


Figure 3.23(a): Change of pressure head with time

### Total Head Vs Time

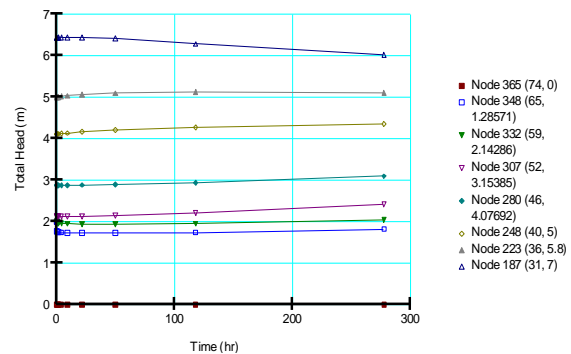


Figure 3.23(b): Change of total head with time

### Pore-water pressure Vs time

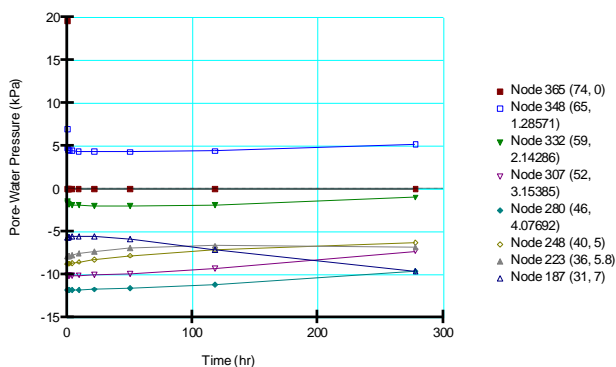


Figure 3.23(c): Change of pore-water pressure with time



Total head, at the node 365 is always zero because it defined as a boundary condition and increasing with the elevation of the nodes (Figure 3.23(a)). But the total pressure at all the nodes does not change so much with the time. It is because that the portion of the water table which representing the downstream side doesn't show big variation. Only change observed at the upstream side.

The same results have obtained in case of pore - water pressure and the pressure head changes (figures 3.23(c) and 3.23(b)). The node 348 only has the positive pressure head as well as the pore-water pressure because; it is the one and only node which lies above the water table at all. But, considerable changes observed at the node 187 (on the top). Because, at the very early stage, water table was very close to the node 187 and with time it has moved apart from the node. Therefore, further reduction in pore-water pressure and pressure head is observed at the node 187.

How ever, this pressure changes cannot create any critical situations on the downstream slope of the profile 3. So, we can conclude that the slope should be more stable for any flooding situations. Let see the stability analysis.

3.2.3.2 Stability analysis

Stability analysis of the profile 2 has been done for initial and final stages of the flooding. Same material properties and same methods have been chosen as in the previous profiles. Figures 3.24(a) and 3.24(b) represent the stability of the slope at initial and final stages respectively.

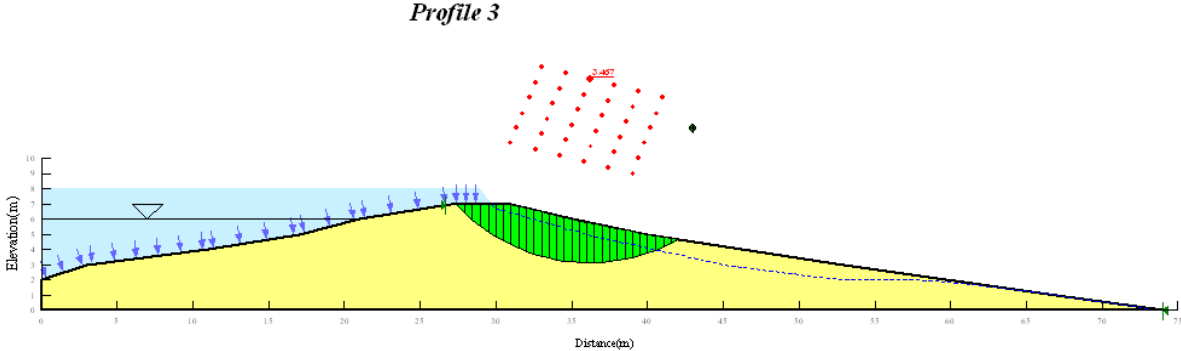


Figure 3.24(a): Stability at initial stage

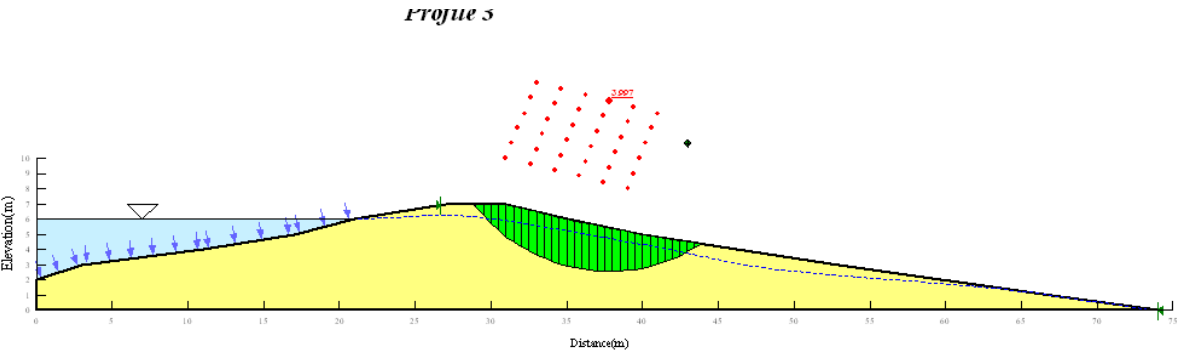


Figure 3.24(b): Stability at final stage

The factors of safety for the initial (3.457) and final (3.997) stages are extremely high. So, the slope has enough stability even in critical flood conditions. We can clearly know the reasons for the extreme stability are

1. The slope is very gently steep on downstream side
2. No huge changes in pressure conditions
3. Very low amount of seepage through the dam even in the extreme flood situations

Now, the above results satisfy our conclusion which mentioned before the stability analysis. As the slope is stable for any condition, it is unnecessary that the analysis of slice and interslice data corresponds to the slope stability analysis of profile 3.

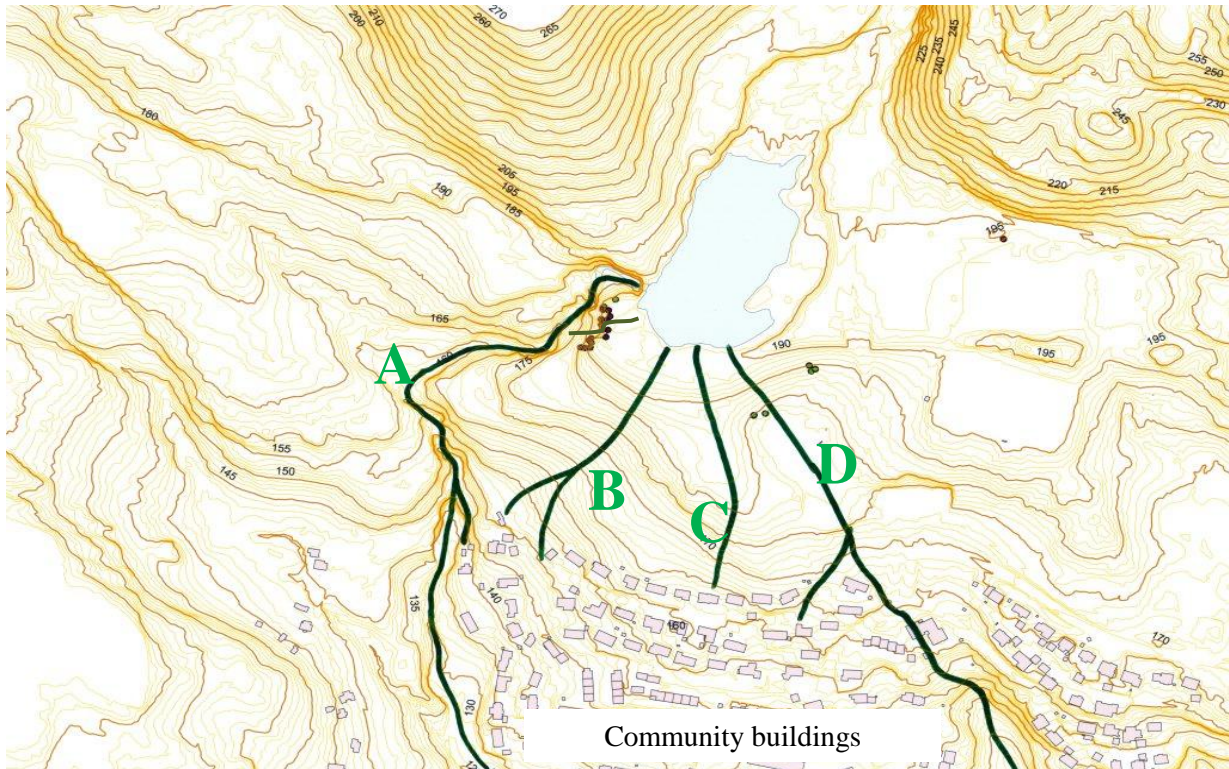
### 3.3 Consequence of Dam breaks

A dam failure may release huge quantity of water suddenly and produce large flood waves. Depending on the quantity of water and the higher pressure, these flood waves have enough capacity to damage nearest inhabitants as well as the infrastructure in the downstream side(Singh 1996a). Some extreme flood situations may destroy the strong structure like power lines, industrial plants and causes several environmental impacts.

Another minor threat of an embankment dam failure is sedimentation, because of the finer particles which carried out and accumulated in the upstream side of the dam(Evans et al. 2000). The threat significantly reduces the depth of the upstream side and effect the integrity of the downstream side of the dam.

The dam Årbogen situated between 180-190 m above the mean sea level. This elevation is low at the downstream side increases towards the upstream side within that range. In the downstream side, community residences established almost 20m below the dam with a distance of ~300m. Therefore, the flood waves produced due to the dam failure have enough potential to reach and impact the community structures.

The figure 3.25 shows the possible flood paths which reach the nearest community when the dam failed or overflowed.



*Figure 3.25: Possible flood paths down to the community residences*

The elevation contours in the figure 3.25 clearly shows that the upstream side of the dam is highly elevated area. Therefore, in the flooding situations flood waves never travel in a direction other than the direction of the flood path those indicated in the map. The A, B, C and D are some possible flood paths which reach the community buildings in different areas. However, each flood paths should have distinct characteristics.

The flood A is generated by either dam failure or direct overflow. Because, the stability analysis of profile 1 shows that the possibility of slope failure in the area (corresponds to the flood path A) is quiet high. The flood A travels through a valley until it passes the residences. Also there are very few buildings near to the flood path. Therefore impact from the flood wave A is considerably low.

The other flood waves can be generated most probably by the direct overflow. Because, the stability analysis show that the slopes corresponds to the other flood waves are extremely stable. Still, these waves are potentially dangerous. Because;

1. Less runout distance to the community
2. The upstream side which corresponds to the flood generating area is very shallow (overflow may occur easily)

# CHAPTER 4

## 4.0 Conclusion and recommendations

This thesis work is based a recently happened small - natural dam failure in Norway. The dam Årbogen was encountered by an intense rainfall in August 2010 which caused flood and slides on downstream side of the dam. The primary objective of the study is analyzing the reliability of the dam during the flooding situation. For the task, an attempt of numerical modeling has been performed using the professional version of the popular geotechnical software *GeoSlope*. The results and conclusions of the analysis are summarized below.

### 4.1 Conclusions

During the flood, there are three major slides occurred on downstream slope which close to the concrete part of the dam. The avalanche pits have maximum of 10m wide and 4m depth at the back scarp. Upper part of the pits are mostly composed with gravels and getting fine sands through the deep. The bottom of the pit 1 has fine sand layer which appears in dark gray color shows a long time out flow of water through the layer. From the observations at the field it could be concluded that the direct overflow and the exceeded seepage water are the main causes of the slides. The Norwegian Geotechnical Institute has been stabilized the slides temporarily immediately after the event with large stones called 'Drammen granite'.

The grain size analysis has been done for the samples collected from all the avalanche pits and two boreholes. At the end of the analysis most of the samples described as gravelly sand and some sandy gravel. But, the values for the *coefficient of uniformity* ( $C_u$ ) show that the all samples collected from the field are *well graded soils*.

The seepage and stability analysis have been done for 3 selected profiles across the embankment. The profile 1 is a narrow profile and quiet steep at the downstream side of the embankment. Seepage analysis results for the profile 1 shows that the seepage through the embankment at normal conditions was  $6.22 \times 10^{-5} \text{m}^3/\text{s}$  and it is increased up to  $9.89 \times 10^{-5} \text{m}^3/\text{s}$  during the flood. This shows that the deposit is not well compacted. The factor of safety for the normal dam condition was 1.076 and during the flood, it was reduced up to 0.799, which shows, the deposits are getting more critical stability conditions during the flood. The major slides happened during the flood are located around the profile 1. This is a solid evidence for the results from the stability analysis.

The analysis clearly shows that there are no possible slides along the profile 2 and profile 3. Both of the profiles are sloping very gently at the downstream side and have relatively long profiles.

Large flood waves created by a dam failure are the major threat to the human life as well as the property. Sedimentation along the upstream slopes reduces the depth of the dam and causes the overtopping when an intense rainfall occurred.

The dam Årbogen is situated ~20m above the community residences with the distance of ~300m. Therefore the flood waves produced by the dam break have enough potential to reach the residence area and highly impact the human life around the dam site.

## 4.2 Recommendations

The dam Årbogen is constructed for the purpose of an industry and no longer use for the purpose now. But, very less maintenance works are carried out during the past. Continuous monitoring and maintaining are strongly recommended to protect the nearest people from the flood hazard. Increasing the depth of the upstream water may help to prevent the overtopping situations. Also the simple construction of safety measures could be an option to reduce the impact from the flood waves.

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## ***Appendices:***

### ***Appendix A – Borehole Logs(Årbogen Dam)***

- A.1 Borehole 1*
- A.2 Borehole 2A*
- A.3 Borehole 2B*
- A.4 Borehole 3*
- A.5 Borehole 4*
- A.6 Borehole 5*
- A.7 Borehole 6*
- A.8 Borehole 7*
- A.9 Borehole 8*

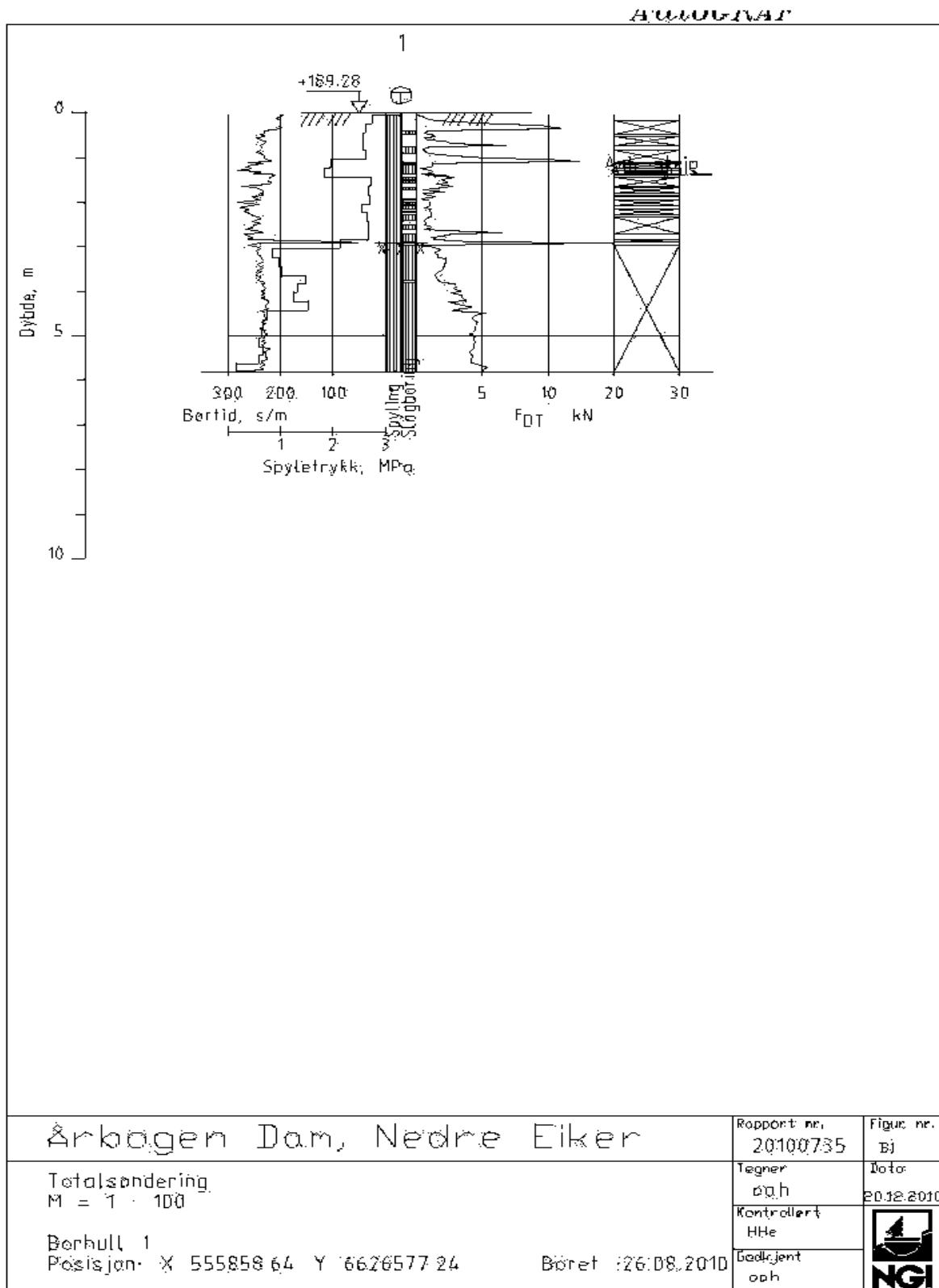
### ***Appendix B – GeoSlope Reports***

- B.1 Profile 1*
- B.2 Profile 2*



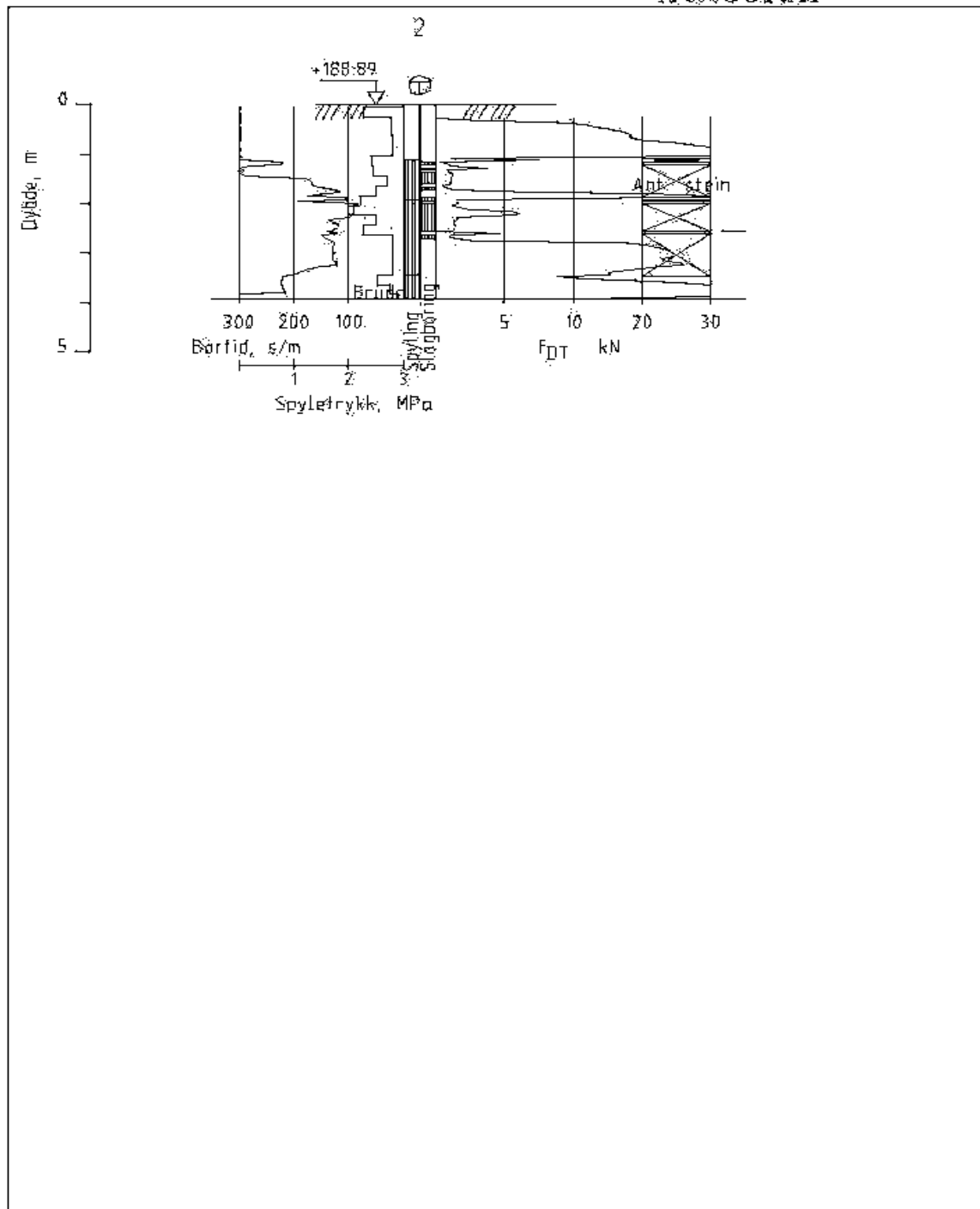
## Appendix A: Borehole Logs (Årbøgen Dam)


### A.1 Borehole 1



A.2 Borehole 2A

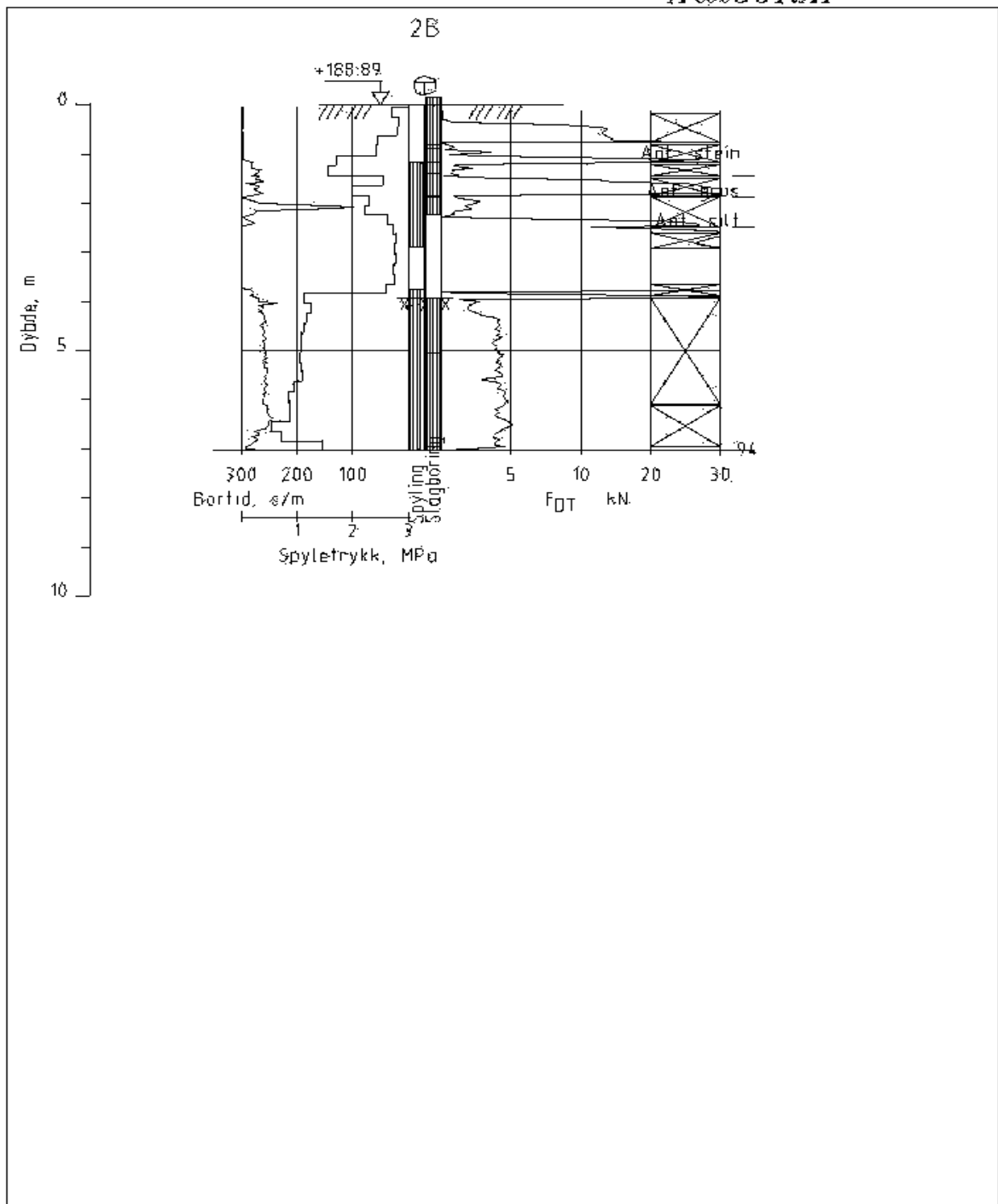
AUUGUAS




Årkøgen Dam, Nedre Eiker Totalsøndering M = 1 : 100 Børhull 2 Posisjon: X 555848.29 Y 6628575.10 Boret 30.08.2010	Rapport nr. 20100735	Figur nr. B2-A
	Tegnet ooh	Dato 20.12.2010
	Kontrollert HHe	
	Godkjent ooh	

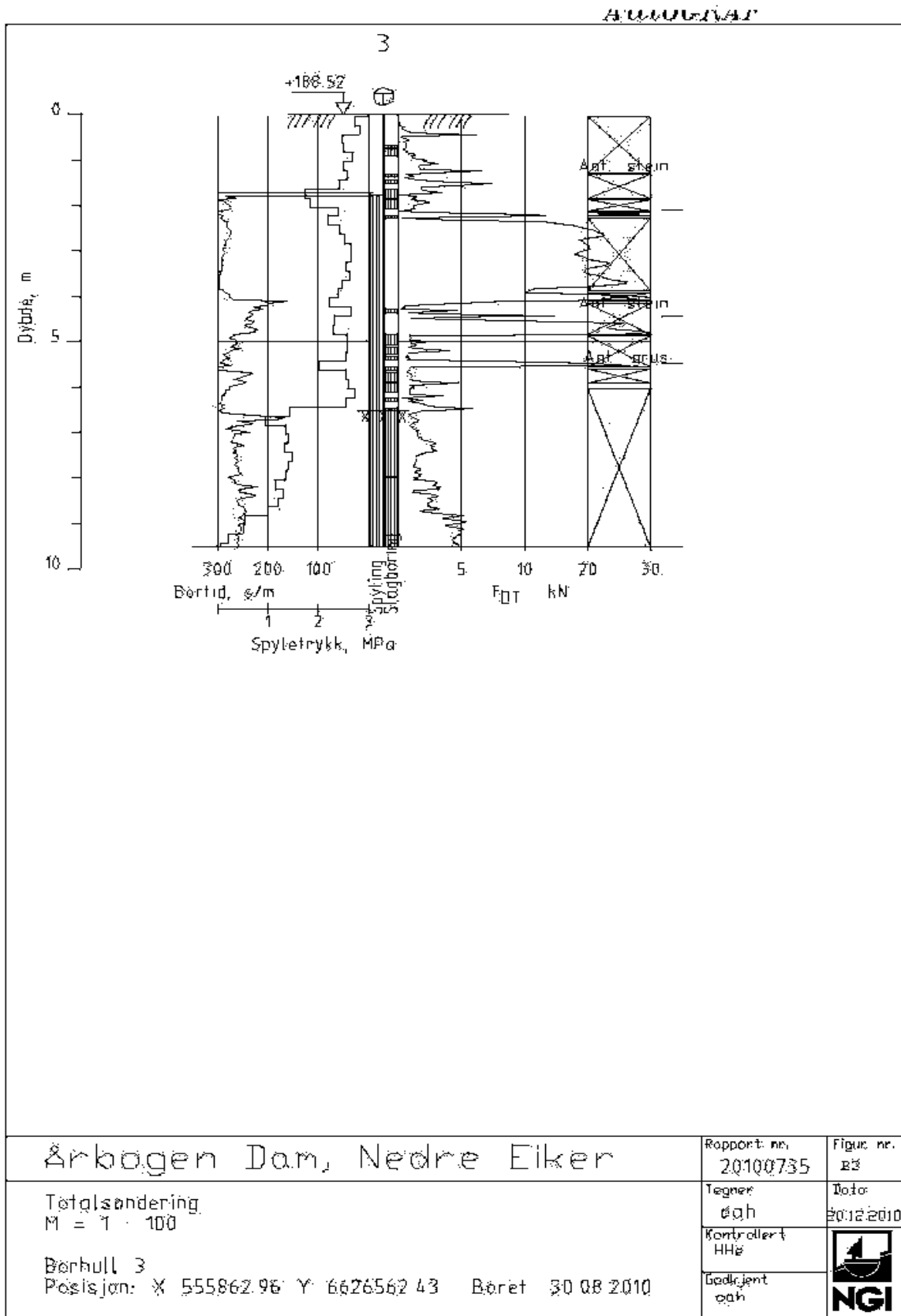
A.3 Borehole 2B:

AUUGU 2010

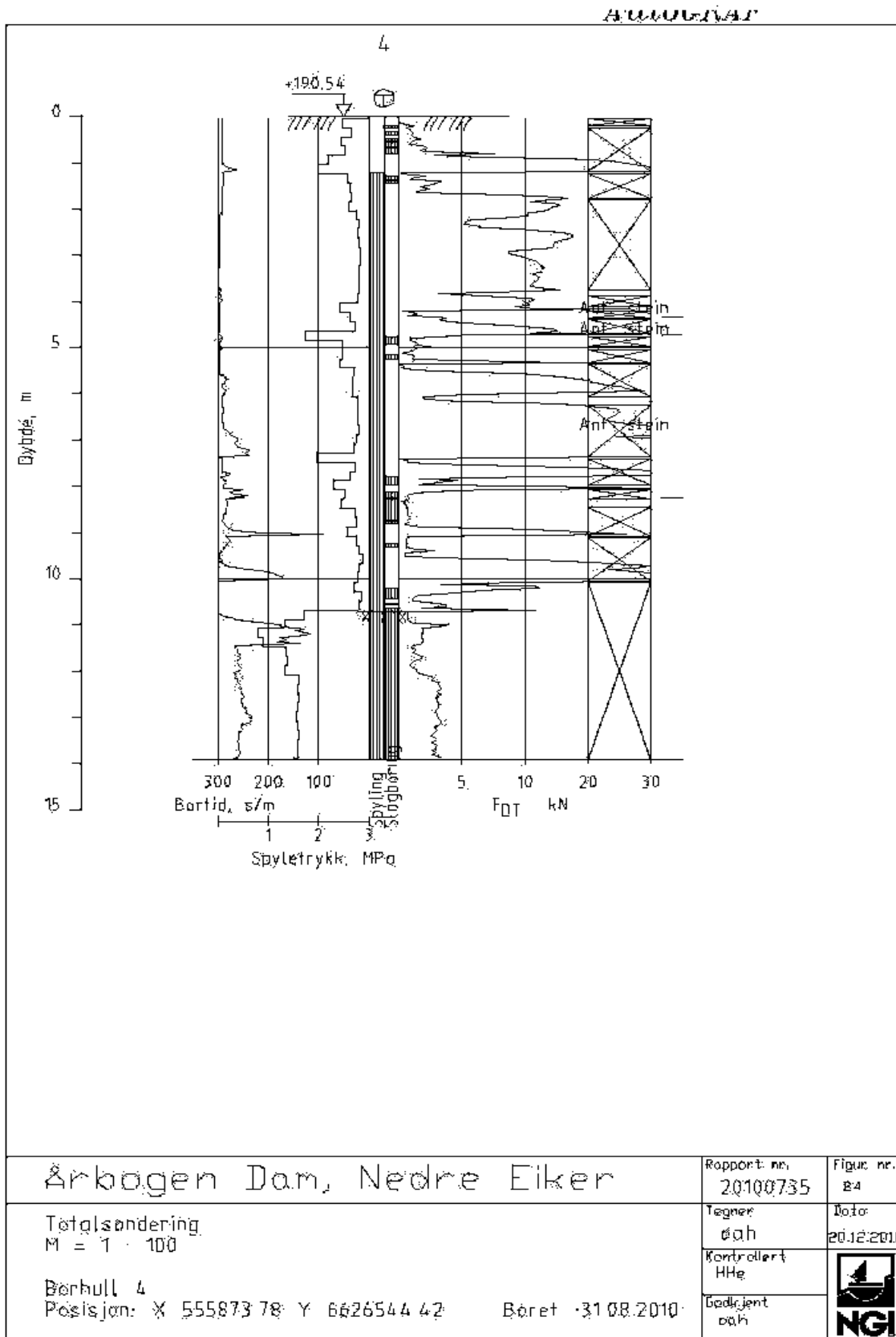


Årbogen Dam, Nedre Eiker Totalsøndering M = 1 : 100 Ber hull 2B Posisjon: X 555848 Z9 Y 6626575 10 Boret 30 08 2010	Rapport nr. 20100735	Figur nr. B2-B
	Tegner oah	Dato: 20.12.2010
	Kontrollert HHe	
	Godkjent oah	

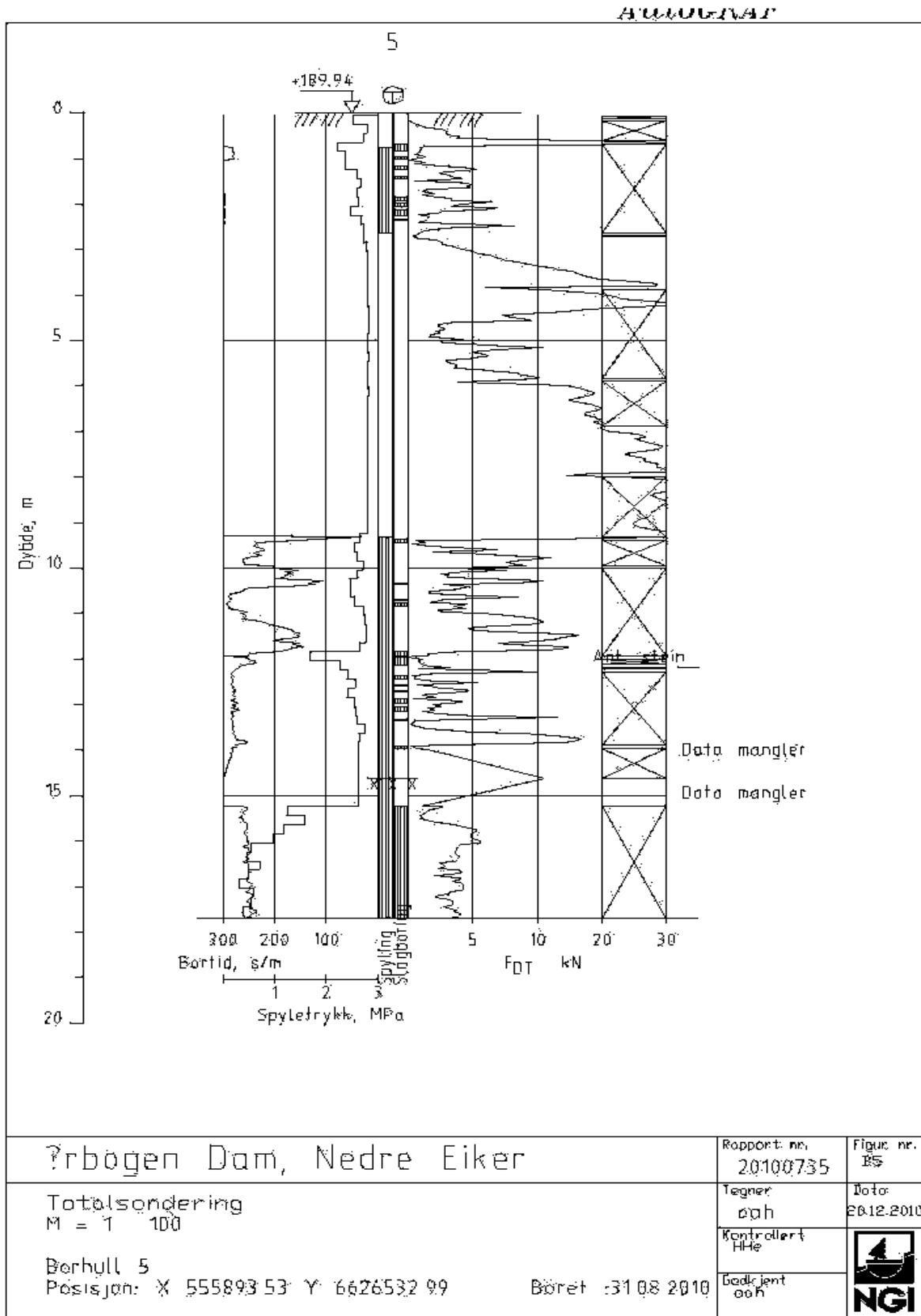
A.4 Borehole 3:



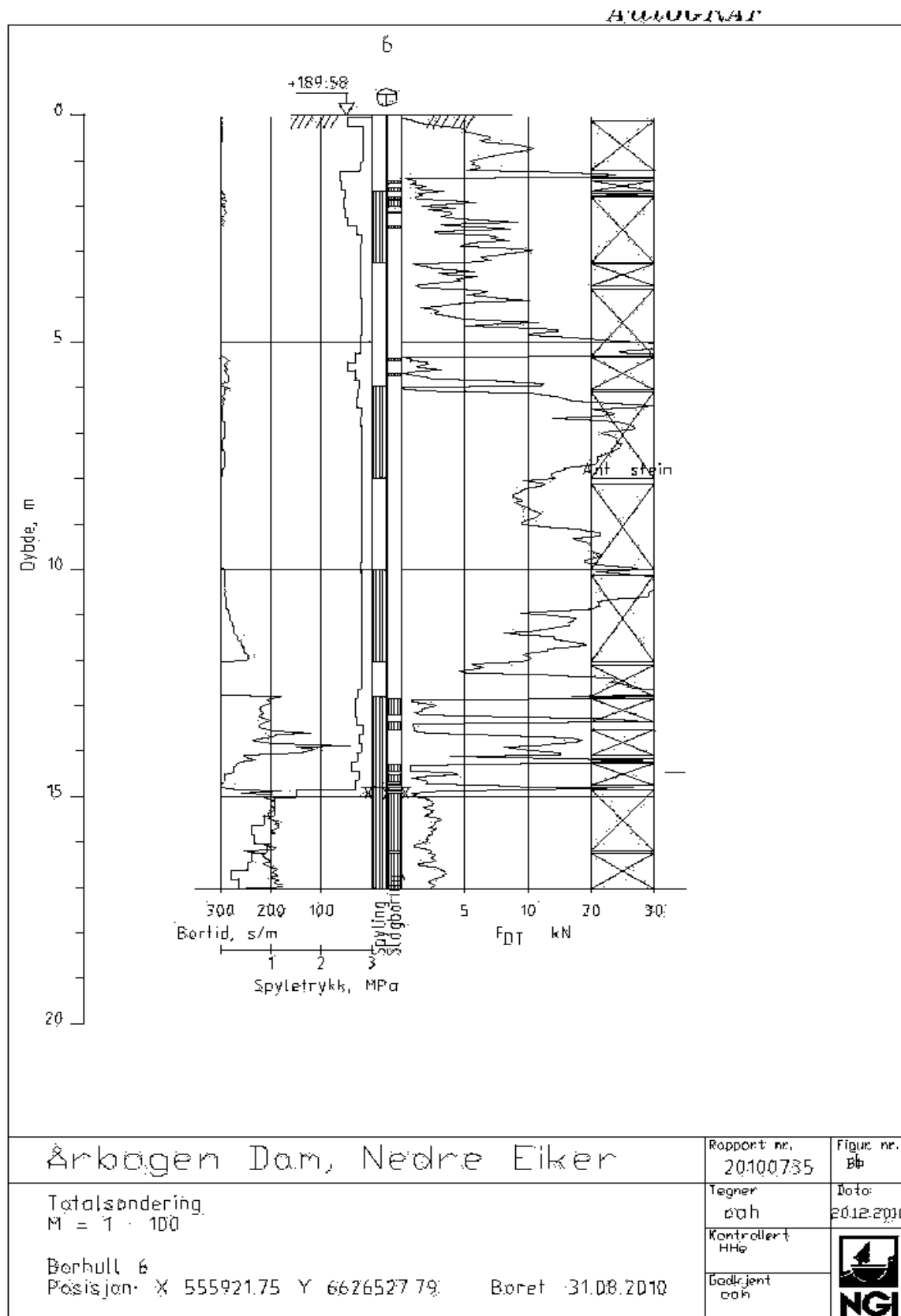
A.5 Borehole 4:



**A.6 Borehole 5:**

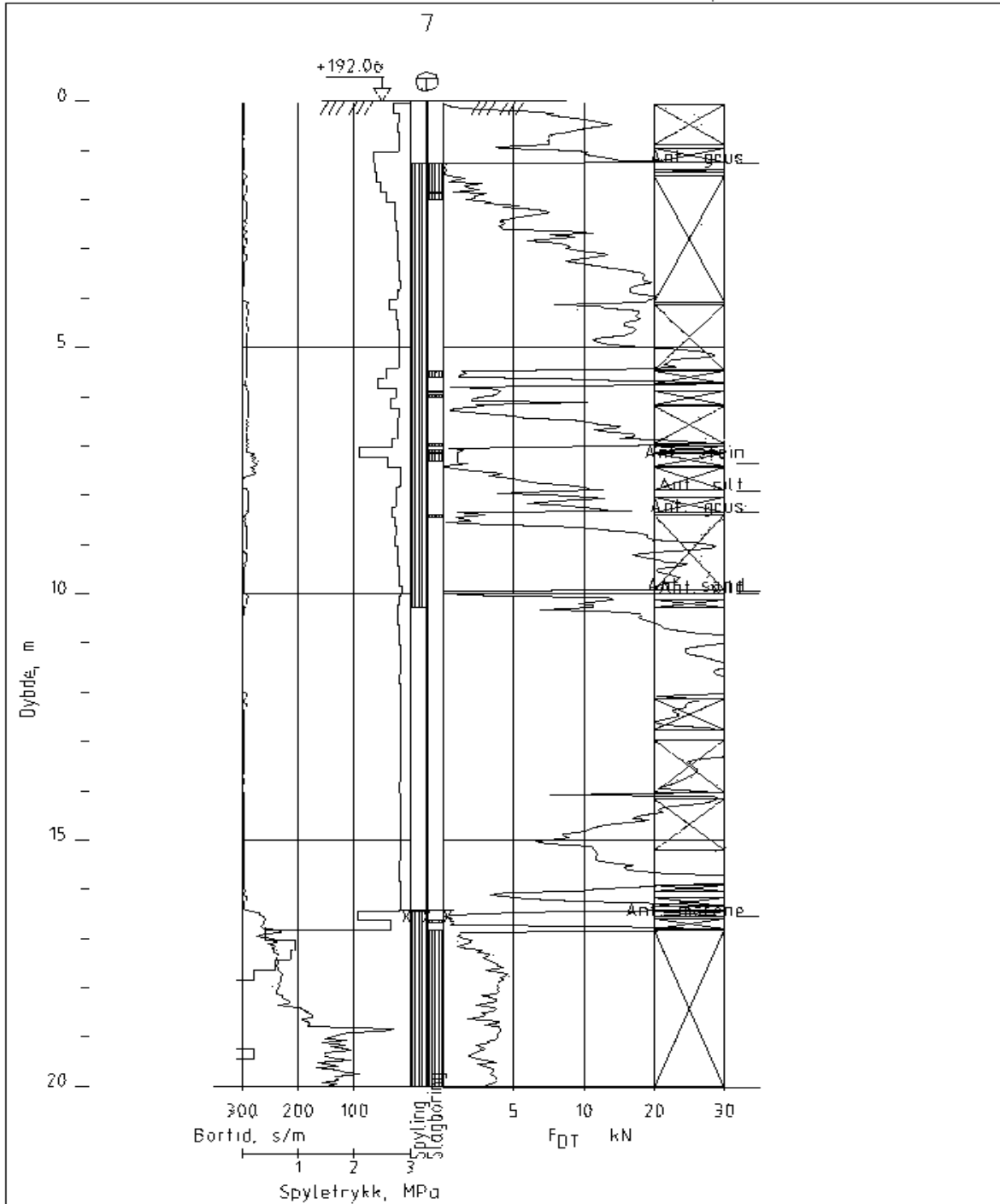



**A.7 Borehole 6:**



A.8 Borehole 7:

ARUBORG



Årkogen Dam, Nedre Eiker  Totalsøndering M = 1 : 100  Borchull 7 Posisjon: X 555948.64 Y 6626523.66 Boret 01.09.2010	Rapport nr. 20100735	Figur nr. B7
	Tegner oah	Dato: 20.12.2010
	Kontrollert HHe	
	Godkjent oah	



## ***Appendix B: GeoSlope Reports***

### ***B.1 Profile 1***

Transient analysis

# Transient analysis

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Report generated using GeoStudio 2007, version 7.13. Copyright © 1991-2008 GEO-SLOPE International Ltd.

## **File Information**

Created By: Rajeeth Ambikaipahan  
Revision Number: 78  
Last Edited By: Rajeeth Ambikaipahan  
Date: 2011-05-23  
Time: 13:39:42  
File Name: Årbogen\_Profile 1\_flooding.gsz  
Directory: E:\Thesis\Part 3 - Seepage -Stability analysis\SeepW\_Models\_original\Profile 1\  
Last Solved Date: 2011-05-23  
Last Solved Time: 13:40:00

## **Project Settings**

Length(L) Units: meters  
Time(t) Units: Seconds  
Force(F) Units: kN  
Pressure(p) Units: kPa  
Mass(M) Units: g  
Mass Flux Units: g/sec  
Unit Weight of Water: 9.807 kN/m<sup>3</sup>  
View: 2D

## **Analysis Settings**

### **Transient analysis**

Description: Transient analysis of the seepage  
Kind: SEEP/W  
Method: Transient  
Settings  
    Initial PWP: Water Table  
    Include Air Flow: No  
Control  
    Apply Runoff: Yes  
Convergence  
    Maximum Number of Iterations: 25  
    Tolerance: 0.001

Transient analysis

Coordinate: (11, 16) m

Coordinate: (20, 15) m

Coordinate: (26, 13) m

Coordinate: (34, 9) m

## Flux Sections

### Flux Section 1

Coordinates

Coordinate: (21, 0) m

Coordinate: (21, 16) m

## K Functions

### Uniform Fine Sand #1, Ksat = 2.15e-05 m/s

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 36 %

K-Saturation: 2.15e-005

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.05, 2.15e-005)

Data Point: (0.1, 2.15e-005)

Data Point: (0.31623, 2.15e-005)

Data Point: (1, 2.15e-005)

Data Point: (3.1623, 1.5655e-005)

Data Point: (10, 2.6663e-007)

Data Point: (31.623, 5.0635e-009)

Data Point: (100, 1.9311e-010)

Data Point: (316.23, 1.0936e-011)

Data Point: (714.29, 2.0178e-012)

Data Point: (1000, 1.0607e-012)

Data Point: (3162.3, 1.271e-013)

Data Point: (10000, 1.3876e-014)

Data Point: (31623, 1.2917e-015)

Data Point: (69541, 2.3667e-016)

Data Point: (100000, 1.065e-016)

Data Point: (138370, 5.1439e-017)

Data Point: (207190, 2.0282e-017)

Data Point: (276020, 1.0179e-017)

Data Point: (316230, 7.2478e-018)

Transient analysis

Data Point: (344850, 5.8021e-018)  
Data Point: (413670, 3.56e-018)  
Data Point: (482500, 2.2856e-018)  
Data Point: (551330, 1.5102e-018)  
Data Point: (620150, 1.0084e-018)  
Data Point: (688980, 6.6815e-019)  
Data Point: (757810, 4.3625e-019)  
Data Point: (826630, 2.8216e-019)  
Data Point: (895460, 2.1571e-019)  
Data Point: (964290, 2.1571e-019)  
Data Point: (1000000, 2.1571e-019)

#### Estimation Properties

Hydraulic K Sat: 0 m/sec  
Hyd. K-Function Estimation Method: Van Genuchten Function  
Maximum: 1000  
Minimum: 0.01  
Num. Points: 20  
Residual Water Content: 0 m<sup>3</sup>/m<sup>3</sup>

## Hydraulic Boundary Functions

### Lowering water table

Model: Spline Data Point Function  
Function: Total Head vs. Time  
Curve Fit to Data: 100 %  
Segment Curvature: 100 %  
Y-Intercept: 16  
Data Points: Time (sec), Total Head (m)  
Data Point: (0, 16)  
Data Point: (1036246.7, 15.31197)  
Data Point: (3688327.4, 14.316183)  
Data Point: (8000000, 14)

## Vol. Water Content Functions

### Uniform Fine Sand #1

Model: Data Point Function  
Function: Vol. Water Content vs. Pore-Water Pressure  
Curve Fit to Data: 100 %  
Segment Curvature: 100 %  
Mv: 0 /kPa

Porosity: 0.29986154

Data Points: Matric Suction (kPa), Vol. Water Content (m<sup>3</sup>/m<sup>3</sup>)

Data Point: (0.3, 0.3)

Data Point: (0.6, 0.3)

Data Point: (1.7384, 0.29109)

Data Point: (2.6544, 0.27465)

Data Point: (4.0437, 0.22834)

Data Point: (4.6935, 0.19763)

Data Point: (6.2689, 0.17053)

Data Point: (8.0649, 0.15418)

Data Point: (13.893, 0.12467)

Data Point: (33.333, 0.0983)

Data Point: (97.701, 0.079377)

Data Point: (375.15, 0.065204)

Data Point: (1810.5, 0.051968)

Estimation Properties

Vol. WC Estimation Method: Sample functions

Sample Material: Clay

Saturated Water Content: 0 m<sup>3</sup>/m<sup>3</sup>

Liquid Limit: 0 %

Diameter at 10% passing: 0

Diameter at 60% passing: 0

Maximum: 1000

Minimum: 0.01

Num. Points: 20

## Regions

	Material	Points	Area (m <sup>2</sup> )
Region 1	Embankment	1,2,6,3,7,4,5	495

## Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	1	2	
Line 2	4	5	Potential Seepage Face
Line 3	5	1	
Line 4	2	6	Pressure Head
Line 5	6	3	Pressure Head
Line 6	3	7	Pressure Head

Transient analysis

Line 7	7	4
--------	---	---

## Points

	X (m)	Y (m)	Hydraulic Boundary
Point 1	0	0	
Point 2	0	9	
Point 3	15	15	
Point 4	27	15	
Point 5	45	0	Zero Pressure
Point 6	12.5	14	
Point 7	20	15	

## B.2 Profile 2

Transient Seepage

# Transient Seepage

---

Report generated using GeoStudio 2007, version 7.13. Copyright © 1991-2008 GEO-SLOPE International Ltd.

## File Information

Revision Number: 44  
Last Edited By: Rajeeth Ambikaipahan  
Date: 2011-05-27  
Time: 09:25:24  
File Name: Årbogen\_Profile2.gsz  
Directory: E:\Thesis\Part 3 - Seepage -Stability analysis\SeepW\_Models\_original\Profile 2\  
Last Solved Date: 2011-05-27  
Last Solved Time: 09:25:32

## Project Settings

Length(L) Units: meters  
Time(t) Units: Seconds  
Force(F) Units: kN  
Pressure(p) Units: kPa  
Mass(M) Units: g  
Mass Flux Units: g/sec  
Unit Weight of Water: 9.807 kN/m<sup>3</sup>  
View: 2D

## Analysis Settings

### Transient Seepage

Description: Steady state seepage analysis of Årbogen dam, Profile 2  
Kind: SEEP/W  
Parent: Årbogen\_Profile2  
Method: Transient  
Settings  
    Initial PWP: Water Table  
    Include Air Flow: No  
Control  
    Apply Runoff: Yes  
Convergence  
    Maximum Number of Iterations: 500  
    Tolerance: 0.001

## Transient Seepage

Maximum Change in K: 0.1  
Rate of Change in K: 1.02  
Minimum Change in K: 1e-005  
Equation Solver: Direct  
Potential Seepage Max # of Reviews: 10

### Time

Starting Time: 100 sec  
Duration: 100000 sec  
# of Steps: 12  
Step Generation Method: Exponential  
Initial Increment Size: 834 sec  
Save Steps Every: 1  
Use Adaptive Time Stepping: No

## Materials

### Embankment

Model: Saturated / Unsaturated  
Hydraulic  
K-Function: Silty Sand, Ksat = 5.0e-7 m/s  
Vol. WC. Function: Silty Sand  
K-Ratio: 1  
K-Direction: 0 °

## Boundary Conditions

### Potential Seepage Face

Function: Total head vs time  
Type: H (Q=0)

### Head

Function: Lowering lake level  
Type: H (Q=0)

## Initial Water Tables

### Initial Water Table 1

Max. negative head: 5  
Coordinates

Transient Seepage

Coordinate: (0, 9) m  
Coordinate: (10, 9) m  
Coordinate: (37, 8) m  
Coordinate: (50, 7) m

## Flux Sections

### Flux Section 1

Coordinates

Coordinate: (42, 0) m  
Coordinate: (42, 13) m

## K Functions

### Silty Sand, $K_{sat} = 5.0e-7$ m/s

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 38 %

K-Saturation:  $5e-007$

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.05,  $5e-007$ )  
Data Point: (0.1,  $5e-007$ )  
Data Point: (0.59948,  $5e-007$ )  
Data Point: (3.5938,  $4.9508e-007$ )  
Data Point: (21.544,  $1.5004e-007$ )  
Data Point: (129.15,  $1.6829e-010$ )  
Data Point: (774.26,  $6.6189e-013$ )  
Data Point: (1111.1,  $2.5875e-013$ )  
Data Point: (4641.6,  $8.6713e-015$ )  
Data Point: (27826,  $1.4089e-016$ )  
Data Point: (105930,  $6.064e-018$ )  
Data Point: (166810,  $2.0175e-018$ )  
Data Point: (210740,  $1.1285e-018$ )  
Data Point: (315560,  $3.9826e-019$ )  
Data Point: (420370,  $1.7997e-019$ )  
Data Point: (525190,  $9.1455e-020$ )  
Data Point: (630000,  $4.8751e-020$ )  
Data Point: (734810,  $2.561e-020$ )  
Data Point: (839630,  $1.314e-020$ )  
Data Point: (944440,  $1.09e-020$ )



Transient Seepage

Data Point: (1000000, 1.09e-020)  
Estimation Properties  
Hydraulic K Sat: 0 m/sec  
Hyd. K-Function Estimation Method: Van Genuchten Function  
Maximum: 1000  
Minimum: 0.01  
Num. Points: 20  
Residual Water Content: 0 m<sup>3</sup>/m<sup>3</sup>

## Hydraulic Boundary Functions

### Lowering lake level

Model: Spline Data Point Function  
Function: Total Head vs. Time  
Curve Fit to Data: 100 %  
Segment Curvature: 100 %  
Y-Intercept: 9  
Data Points: Time (sec), Total Head (m)  
Data Point: (0, 9)  
Data Point: (15033.686, 8.4476922)  
Data Point: (44614.585, 7.5076807)  
Data Point: (100100, 7)

### Total head vs time

Model: Spline Data Point Function  
Function: Total Head vs. Time  
Curve Fit to Data: 100 %  
Segment Curvature: 100 %  
Y-Intercept: 0  
Data Points: Time (sec), Total Head (m)  
Data Point: (0, 0)  
Data Point: (100100, 0)

## Vol. Water Content Functions

### Silty Sand

Model: Fredlund-Xing Function  
Function: Vol. Water Content vs. Pore-Water Pressure  
A: 18.835 kPa  
N: 3.8995  
M: 0.8165