Safety Evaluation of the Zhaoli Tailings Dam
A seepage, deformation and stability analysis with GeoStudio

Johan Bäckström
Malin Ljungblad
Abstract

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The mining industry produce large amount of mine waste, also called tailings, which must be kept in tailings dams. In this thesis the safety and stability of a tailings dam have been studied, where some of the tailing material is being used as filling material. The dam has been modelled and simulated using the software Geostudio. To evaluate the safety and stability of the dam seepage, stress and strain as well as slope stability have been simulated with SEEP/W, SIGMA/W and SLOPE/W, which are different modules in the Geostudio software.

The results show that the dam is stable for all tested scenarios. However, in this thesis many simplifications and assumptions have been made so it is recommended to do a more detailed study to confirm the safety of the dam. The dam should also be simulated for earthquakes before a definite evaluation can be made.

This master thesis has been conducted in cooperation with Vattenfall AB, Energiforsk AB, Uppsala University and Tsinghua University in Beijing, China. The project has been carried out on the planned Zhaoli ditch tailing dam in the Shanxi Province, China.
Sammanfattning

Detta examensarbete har utförts i samarbete med Energiforsk, Vattenfall, Tsinghua universitet och Uppsala universitet. Arbetet har bedrivits under månaderna januari till juni 2016, med den största delen av arbetet utfört vid Tsinghua universitet under mars till juni. Syftet har varit att utvärdera säkerheten av en planerad gruvdamm som konstruerats med gruvmaterial som fyllnadsmaterial utifrån analys av genomflöde, spännings, deformering och stabiliteten av en planerad gruvdamm i Kina.

Den föreslagna gruvdammen Zhaoli är planerad att byggas i anslutning till järnmalmsgruvan Daxigou i provinsen Shanxi i norra Kina. Dammens nuvarande design är enligt uppströmsmetoden och konstruktionen är planerad i ett flertal steg med en slutlig höjd på ungefär 190 meter.

Simuleringarna har genomförts i GeoStudio för två olika scenariom som utgör två planerade höjder i dammkonstruktionen; vid höjden 110 meter och vid höjden 190 meter. Båda scenariorna simulerades för normala och översvämmede driftförhållanden, vilket motsvarade olika maximalt tillåtna vattennivåer i dammarna.

Resultaten visar att den studerade gruvdammen är säker i de två simulerade scenariom, både vid normala och översvämmede driftförhållanden. Hastigheten på genomströmning är låg och deformationen är överkomlig, vilket inte bör orsaka stabilitetsrisker i dammen. Den mest kritiska ytan i dammens slutning har en säkerhetsfaktor (factor of safety) på 1.5 eller högre, vilket är högre än de värden som krävs enligt kinesiska standarder för säker dammkonstruktion.
Acknowledgement

This master degree thesis project has been carried out at the department of Hydraulic Engineering at Tsinghua University in Beijing, China from March to May 2016.

First of all, we would like to thank our supervising Professor Liming Hu for inviting us to Tsinghua University and his group of Ph.D. and M.Eng. students in the Department of Hydraulic Engineering. Even though the Professor had a full schedule, he always took time to answer our questions and to give us advice on how to proceed in our project. We would also like to thank Dr. Wu and our co-working student Ms. Dantong Lin for all the help throughout the project.

We are very grateful to the group of Ph.D. and M.Eng. students that we had the pleasure to getting to know during our stay at Tsinghua University. They did not only give us help and support on our work on the thesis, but also helped us with arrangements of social activities and trips. Furthermore, we would like to state our greatest gratitude and many thanks to Mr. Yang Yang and Mr. Luo Xiaoyu for their kind treatment, their advice and friendship during our time in China.

We would also like to thank and state our gratitude to Professor James Yang from Vattenfall R & D and Kungliga Tekniska Högskolan for arranging this master thesis project and making this invaluable experience possible. We are also very thankful for the help, feedback and guidance that have been given by Doctor Per Norrlund, our supervisor at Uppsala University.

The project is within the frame of dam safety and is a diploma work program funded by Energiforsk AB and managed by Professor James Yang. The director of this diploma work program is Ms. Sara Sandberg. Uppsala University have also contributed with funding to this master degree thesis, which has enabled the accomplishment of the project.

Beijing, May 27th 2016
Johan Bäckström and Malin Ljungblad
## Nomenclature

<table>
<thead>
<tr>
<th>Denomination</th>
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<tr>
<td><strong>Greek</strong></td>
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<tr>
<td>Angle of shearing resistance</td>
<td>$\phi$</td>
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<tr>
<td>Angle of shearing resistance effective</td>
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<tr>
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<td>$\gamma'$</td>
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<td>Volume of water flowing per unit time</td>
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<tr>
<td>Young’s modulus</td>
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Glossary

**Consolidation**
Consolidation is the process of a long term gradual decrease in volume of a soil due to a decrease in pore water in the soil. The reduction of pore water occurs when soil are subjected to stress that pack the soil particles together, causing the water to squeeze out of the soil.

**Datum**
Datum is a baseline or a reference that can be chosen at any elevation. When calculating values of different points and parameters these can be defined relative to the datum.

**Hydraulic head**
Hydraulic head, or the piezometric head, is the measurement of the pressure in a liquids above a set datum. It is constituted by the pressure head and the elevation head.

**Liquefaction**
When a saturated or partially saturated soil behaves like a liquid due to extensive losses of shear strength due to sudden changes in stress condition. Changes in stress conditions could be caused by earthquakes etc.

**Mine tailings**
Mohr’s circles are used when determining stress distributions in a rotated coordinate system graphically.

**Mohr-Coulomb theory**
Describes the response of different materials when they are subjected to shear stress and normal stress. Materials often obey this theory, even if only for a small part, which can be seen in their linear shear failure envelopes.

**Permeability**
In earth science, permeability is the measure of the ability of a material to transmit a fluid. For soil and water the permeability is called hydraulic conductivity.

**Poisson’s ratio**
Poisson’s ratio is used for materials displacement in elasticity theory. The ratio express the relative contraction strain applied normal to the load and the relative extension strain applied in the direction of the load. The Poisson’s ratio is required in order to solve certain stress equations.

**Phreatic surface**
The term phreatic is used in hydrology and the earth sciences to refer to matters relating to ground water (an aquifer) below the water table. The
phreatic surface indicates the location of the pore water pressure under atmospheric conditions, i.e. when the pressure head is zero. This surface is the same as the water table, or ground water level as it is also called. The pore water pressure is positive below the phreatic surface and negative above the phreatic surface.

**Seepage**

Seepage can be defined as the flow of liquid through porous media, e.g. soil. Seepage characteristics are very important in many soil structures, such as dams and embankments.

**Shear failure envelope**

The shear failure envelope is a function of stresses and material parameters which can be used when solving material failure problems. The envelope is expressed in terms of normal and shear stresses which acts on a plane that is inclined in the same direction as the principal stress. The envelope can show the operating boundaries for the given system.

**Specimen**

A sample taken for the purpose of analysis. In this thesis the specimen is defined as an amount of tailing material that is regarded as typical for its kind.

**Strain**

Strain is defined by proportional deformation in a material.

**Stress**

Stress is defined as the force per unit area in a material.

**Young’s modulus**

Young’s modulus, also called elastic modulus, defines the relation between stress and strain in a material. It is used to describe properties of solid materials that are linearly elastic.
Contents

Sammanfattning i

1 Introduction
1.1 Purpose .................................. 1
1.2 Objectives ................................ 1
1.3 Limitations ................................ 1
1.4 Method ................................ 1
1.5 Work breakdown .......................... 2

2 Background
2.1 Tailing materials and disposals ............ 3
2.2 Tailing dam construction methods ............ 3
  2.2.1 General ................................ 3
  2.2.2 Raised embankments ..................... 4
2.3 Tailing dam failures ........................ 9
  2.3.1 Causes of tailing dam failures .......... 9
  2.3.2 Environmental impacts of tailing dam failures .. 11
2.4 The Daxigou Mine Tailing Dam Project ...... 11
  2.4.1 The Zhaoli Ditch Tailing Dam ............ 11
  2.4.2 Design and geometry .................... 12
  2.4.3 Soil parameters ......................... 13

3 Theory
3.1 Seepage Theory ............................ 14
  3.1.1 Seepage in embankment dams .......... 14
  3.1.2 Darcy’s empirical law .................. 14
3.2 Stress-Strain Theory ........................ 15
  3.2.1 Load and deformation .................... 15
  3.2.2 Material models and properties ......... 18
3.3 Slope Stability Theory ...................... 20
  3.3.1 Types of slip surfaces ................. 20
  3.3.2 Limit equilibrium fundamentals ......... 22
  3.3.3 Factor of safety ....................... 24
  3.3.4 Sensitivity Analysis .................... 25

4 Simulation and modeling in GeoStudio™ 26
4.1 General ................................ 26
  4.1.1 About GeoStudio™ ..................... 26
  4.1.2 Dam model and material parameters ..... 26
  4.1.3 Simulated scenarios .................... 26
4.2 SEEP/W ................................ 28
  4.2.1 Purpose ................................ 28
  4.2.2 Input data and boundary conditions ..... 28
  4.2.3 Steady-state and transient analysis ..... 28
4.3 SIGMA/W .................................................. 29
  4.3.1 Purpose ............................................. 29
  4.3.2 Input data and boundary conditions ............... 29
4.4 SLOPE/W .................................................. 29
  4.4.1 Purpose ............................................. 29
  4.4.2 Input data .......................................... 29
  4.4.3 Slip surface ........................................ 30
  4.4.4 Sensitivity Analysis Simulations .................... 30

5 Results .................................................................. 31
  5.1 Seepage analysis ........................................... 31
    5.1.1 110 m-dam during normal conditions .......... 31
    5.1.2 110 m-dam during flooded condition ......... 33
    5.1.3 190 m-dam during normal conditions .......... 35
    5.1.4 190 m-dam during flooded conditions ......... 37
    5.1.5 Summary ............................................. 40
  5.2 Load and deformation analysis .......................... 41
    5.2.1 110 m-dam during normal conditions .......... 42
    5.2.2 110 m-dam during flooded conditions ......... 44
    5.2.3 190 m-dam during normal conditions .......... 46
    5.2.4 190 m-dam during flooded conditions ......... 48
    5.2.5 Summary ............................................. 50
  5.3 Slope stability analysis ................................... 50
    5.3.1 110 m-dam during normal conditions .......... 51
    5.3.2 110 m-dam during flooded conditions ......... 51
    5.3.3 190 m-dam during normal conditions .......... 52
    5.3.4 190 m-dam during flooded conditions ......... 52
    5.3.5 Sensitivity Analysis ................................ 53
    5.3.6 Summary ............................................. 53

6 Discussion .......................................................... 56
  6.1 Further studies ........................................... 57

7 Conclusion .......................................................... 58

Appendix I - Tailing dam failures between 2000-2015 62

Appendix II - Soil parameters 64

Appendix III - Settings and data for simulations in GeoStudio 66
1 Introduction

1.1 Purpose

The purpose of this thesis is to evaluate the stability and seepage of a dam in Shanxi Province in China. The tailing material from the mine is being planned to be used as a filling material for the tailing dam connected to the mine. Since mine tailings and tailing dams are associated with environmental impact, the properties for the tailing material must be studied in order to evaluate the qualities of the tailing material as a filling material regarding structural stability and seepage of water and contaminants.

1.2 Objectives

1. Carry out literature study in the beginning of the project.
2. Model and conduct numerical simulations on the dam using the software GeoStudio.
3. Analyze and evaluate the overall stability of the given tailing dam embankment design to see if, and under what conditions, the tailing material is suitable as a filling material in the purposed dam design.

The scenarios and operation conditions used in this thesis were set in accordance to the requested stability analysis and the tailing dams project pre-feasibility study.

1.3 Limitations

The slope stability analysis in this thesis is not considering seismic loading conditions, even though the dam is planned to be located in an area with known seismic activity and risk for earthquakes. Limitation has also been done in terms of simulated dam raises, where only the first and the last of the planned five dam raises are simulated.

1.4 Method

This master thesis project constitutes of a literature study and numerical simulations in GeoStudio\textsuperscript{TM} 2007 regarding the Zhaoli tailing dam in the Shanxi province, China.

The literature study was conducted to understand tailing dams and their construction methods in order to learn how both the dam design and the tailings material will affect seepage, stress-strain and slope stability in the given dam. Causes of dam failures were studied to get an understanding of
the phenomena that might affect the dam stability. The literature study was also performed to get familiar with numerical analysis of tailing dams.

GeoStudio\textsuperscript{TM} is a simulation program based on both limit equilibrium and finite element methods in order to analyze and compute complex geotechnical problems. SEEP/W, SIGMA/W and SLOPE/W are tools within GeoStudio\textsuperscript{TM} which has been used when simulating seepage, stress-strain and slope stability in the given dam embankment.

Simulations were conducted in three parts. Firstly, a stationary and transient seepage analysis in SEEP/W was carried out in a saturated and unsaturated soil dam embankment. Secondly, the seepage analysis was coupled with SIGMA/W to perform a stress and strain analysis in a saturated and unsaturated stationary analysis. In the third part, the seepage and stress and strain analysis were coupled in SLOPE/W to enable a slope stability analysis of the unsaturated dam embankment.

All three parts of the simulations and analysis in GeoStudio\textsuperscript{TM} were performed for two different scenarios; the 110 m-dam (the first raising of the dam) and the 190 m-dam (the final raising of the dam). Every scenario was also simulated at normal and flooded operation conditions, where the water level in the dam was assumed to change.

1.5 Work breakdown

This master thesis project and report is conducted by two authors, Johan Bäckström and Malin Ljungblad. In order to simplify both the project execution and the report writing process the different parts of the project have been divided between the two authors as follows:

**Johan Bäckström** has been responsible for the research and writing of parts of section 3. Theory (the subsections 3.1 Seepage Theory and 3.2.1 Load and deformation). Johan is also responsible for writing the nomenclature, doing the sensitivity analysis and putting the discussion and conclusion into words.

**Malin Ljungblad** has been responsible for the research and writing of section 2. Background, section 4. Simulation and modeling in GeoStudio\textsuperscript{TM} and parts of section 3. Theory (the subsections 3.2.2 Material Models and properties and 3.3 Slope Stability Theory). Malin is also responsible for writing the abstract, sammanfattning (the Swedish conclusion), acknowledgement, glossary, putting together the appendix and assemble the reference list.

Both authors has contributed to the section 1. Introduction, 5. Results, discussion of both results, further studies and conclusions as well as performing the simulations in this master thesis project. Although, it should be noted that Johan has done the final changes in the simulations of this project because Malin was in a bike accident in mid-May 2016.
2 Background

2.1 Tailing materials and disposals

Large quantities of rock needs to be mined, crushed, pulverized, and processed in order to obtain metals and minerals that is needed in industrial processes and consumer products in today’s society. To release these metals and minerals needed, these rocks are grained into fine particles in order to be separated, leaving large amounts of uneconomical fine rock particles as residue on the mining site. These residues are known as tailings [1].

Tailings, or tailing materials, often contains hazardous substances and needs to be separated from the surrounding environment for a long time. Stationary impoundments and embankments are common disposal methods in order to store the tailing material. When disposed, the tailing material is traditionally and most commonly, in a slurry form consisting of solid tailings diluted in water [1]. There are a range of tailing material compositions that can be used in order to create better and more stable storage systems. For example, there are dewatering methods which thickens or dries tailings prior to disposal, creating advantages in terms of lower seepage volumes, less need of water, and less need of land area. Though, the disadvantages with dewatering methods are that they may be less cost effective than traditional tailing disposal methods [1].

From an economic point of view, local tailing materials are generally the most cost effective choice of dam construction material in order to decrease the embankment cost. This is mainly due to the reduced amount of tailing material that needs to be disposed [1]. However, tailing material composition and properties can vary, which can have an influence on both the dam impoundment and embankment in different ways. Therefore, it is of great importance to investigate the properties of the tailing material so that they meet the requirements for design, stability and drainage of the dam construction. The most important properties for investigation are in-place and relative density, permeability, plasticity, compressibility, consolidation, and stress-shear strength characteristics, which are going to be evaluated in this report [1].

2.2 Tailing dam construction methods

2.2.1 General

There are two basic dam construction types used in order to retain the tailing slurry in the impoundment; the raised embankment and the retention dam. The construction type chosen mainly depends on site conditions and economical factors [1].
The main difference between these two construction methods is that retention dams are constructed at its full height from the beginning whereas raised embankments are, as the name suggests, constructed in phases as the need of disposal capacity increases [1]. The later, raised embankments, are the most common construction method and are therefore the main focus of this report.

### 2.2.2 Raised embankments

When building a raised embankment the construction often begins with a starter dyke at the base of the planned embankment wall. The embankments height is then added on in phases which depends on the volume of tailings that is added to the impoundment. These construction phases enable changes in the embankments during its lifespan, making it possible to solve unplanned design problems or making enlargements of the dam construction. Also, by spreading out the cost of fill materials and placements on the different phases of the impoundments lifespan the initial cost of constructing a raised embankment is lower than other dam construction types [1].

![Figure 1: The three main construction methods for raised embankments: (a) Upstream, (b) Centerline, (c) Downstream method. [2]](image)

There are three construction methods for raised embankment dams; the upstream method, the downstream method, and the centerline method. These are shown in figure 1 above and explained in more detail in the following sections.
2.2.2.1 Upstream

The upstream method is the most commonly used dam design for raised embankments in low risk seismic areas, mainly because of its low initial cost. The reasons for these economical benefits are the minimal amount of tailing material that is required for initial construction and the subsequent adding of embankment height which often consists of the coarse fraction of the tailing material [3].

An upstream embankment construction begins with a starter dike foundation. On the top of the dyke, on the crest, the tailings are usually discharged into the impoundment using spigots or cyclones. This disposition methods creates a dike and a wide beach which mainly consist of coarse material. As more tailings are produced and disposed in the dam impoundment there will be a need for another dike, which will then be built on top of the beach. This construction of dikes will take place in phases, creating a sequential raising of the upstream embankment as shown in figure 2 [1].

![Figure 2: The sequential rising of embankment construction using the upstream method. [2]](image)

There are several limiting factors for the upstream constructing method. The most important ones are tailing gradation, seismic liquefaction susceptibility, the embankments raisin-rate, and phreatic surface control [1]. The later, the phreatic surface control, is the level of saturation in the impoundment and embankment, and has great impact on the stability of the tailing dam. In
order to construct a stable dam the phreatic surface has to be low and should not exit on the embankment slope. This low phreatic surface is maintained by effects caused by important factors such as pond water location, beach-size segregation, and permeability variation [2], shown in figure 3 below.

Figure 3: The phreatic surface for upstreams embankments are affected by the following factors: (a) Impact of pond water location; (b) Impact of beach grain-size segregation and lateral permeability variations; (c) Impact of the permeability of the impoundment foundation [2].

The distance between the location of the ponded water and the embankment crest, also called beach width, is highly affecting the phreatic surface location. The beach width of the embankment in combination with its permeability can help create a low phreatic surface. For example, if the difference between the permeability at the crest ($k_0$) and at the edge of the ponded water ($k_L$) is great ($\frac{k_0}{k_L} = 100$), then a short beach width is enough in order to produce an acceptable phreatic surface [4]. The beach width is an operational factor which is difficult to control by the designer, partly due to different operational conditions (normal water levels, flooded water levels, etc). Generally, the permeability of different zones increases in the direction of the seepage flow. This means that zones with lower permeability are located upstream the embankment whereas zones with higher permeability are located downstream the embankment [4].

There are many factors and variables that cannot be controlled when designing an upstream embankment, which makes seepage and stability characteristics difficult to predict and control. Therefore, uncontrollable factors and variables contributes to risks of dam instability, and even dam failures,
throughout the years (see 2.3 Tailing dam failures) which in recent years has led to an increased numbers of downstream and centerline dam constructions [5].

2.2.2.2 Downstream

Just as the upstream method, the downstream construction method begins with a starter dike and tailing that are disposed into the impoundment through the use of spigots. When there is need for increased storage capacity in the dam, the embankment rise is created by putting additional material on the outside of the dam. Just as the upstream method, this sequential rising of the downstream embankment takes place in phases, which is shown in figure 4.

![Figure 4: The sequential rising of embankment construction using the downstream method [2].](image)

Incorporations of drains and impervious cores are allowed by the downstream method, creating a good phreatic surface control. This along with the ability of compaction and a stronger foundation makes the downstream construction method more stable compared to the upstream construction method. Although the downstream methods stability advantages, the large amounts of tailing materials that is needed for this type of construction can be expensive. Especially if the volume of tailing from the mill is not meeting the volume of fill that is required. Another disadvantage is the large area that
is needed for this construction type, which can be a problem if the space of the dam construction site is limited [1].

2.2.2.3 Centerline
The the centerline method is somewhat a combination of the upstream and the downstream methods. The embankment construction begins with a starter dike and tailing are spread from the crest using spigots, in order to form a beach. When rising the embankment the centerline is kept as a fill meanwhile progressive raises are made on both the upstream and the downstream slopes of the embankment, as shown in figure 5.

![Centerline construction method](image)  
*Figure 5: Centerline construction method [2].*

The centerline method has both advantages and disadvantages when comparing with the upstream and the downstream method. It needs less material than the downstream method, it has acceptable stability qualities in seismic areas, and does not carry too many restrictions for the raising rate of the embankment. The disadvantage is that it is still expensive due to the large volume of tailing needed compared to the upstream method [1].

2.2.2.4 Summary of construction methods
As discussed in above paragraphs, each dam construction methods has its advantages and disadvantages. Therefore, a summary of the characteristics of the described methods is presented in table 1.
Table 1: Short summary of the upstream, downstream and centerline construction methods [2].

<table>
<thead>
<tr>
<th>Embankment Type</th>
<th>Upstream</th>
<th>Downstream</th>
<th>Centerline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mill Tailings Requirements</td>
<td>&gt;40-60% sand; Low pulp density for grain-size segregation</td>
<td>Suitable for any type of tailings</td>
<td>Sands or low-plasticity slimes</td>
</tr>
<tr>
<td>Discharge Requirements</td>
<td>Peripheral, well controlled beach</td>
<td>Varies according to design details</td>
<td>Peripheral, at least nominal beach needed</td>
</tr>
<tr>
<td>Water Storage Suitability</td>
<td>Not suitable for water storage</td>
<td>Good</td>
<td>Permanent storage not recommended</td>
</tr>
<tr>
<td>Seismic Resistance</td>
<td>Poor in high seismic areas</td>
<td>Good</td>
<td>Acceptable</td>
</tr>
<tr>
<td>Raising Rate Restrictions</td>
<td>&lt;4.6-9.2 m/year desirable; &gt;15.2 m/yr can be hazardous</td>
<td>None</td>
<td>Height restrictions may apply</td>
</tr>
<tr>
<td>Embankment Fill Requirements</td>
<td>Natural soil, sand tailings, mine waste</td>
<td>Sand tailings or mine waste if sufficient production rates, or natural soil</td>
<td></td>
</tr>
<tr>
<td>Relative Embankment Cost</td>
<td>Low</td>
<td>High</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

2.3 Tailing dam failures

2.3.1 Causes of tailing dam failures

There was an estimation of 3500 active, inactive, and abandoned tailing dams in the world in 2014 [6]. Among these dams there has been at least 223 recorded cases of dam failures [7][8][9]. However, up to date there is no complete worldwide database for all tailing dam failures due to a large number of unreported accidents and incomplete basic information about the known dams that has failed [6]. Although, in recent reports a number of larger databases has been compiled in order to analyze available tailing dam
failure data. Figure 6 below presents recorded dam embankment failure events over time. Records of dam failures of the decades 2000 and 2010 has been compiled in Appendix I.

![Number of dam failures per decade](image)

Figure 6: Number of recorded dam failures per decade [8][7][9].

There are many reasons for tailing dam failures, such as: (i) use of residual materials from mining when constructing dykes; (ii) successive dam raise in combination of an increase in waste water; (iii) inadequate dam design regulations; (iv) high costs of maintenance when the mine has closed [10]. Furthermore, the main causes of dam failure are extreme weather events (such as heavy rain and snow melting), poor dam management and failing dam constructional design mechanisms such as seepage control [10] [8], as can be seen in figure 7.

There are several dam failures that has taken place since 1990, statistics which has raised warnings about an increased risk potential and higher dam failure frequency in recent years [9]. Some reports points at estimations of an increased number of extreme weather events caused by climate change as a potential future cause of dam failures, as about 28 percent of all dam failures world-wide are caused by these particular events. This could lead to an increase in dam failure rate in the future [10] [8]. Nevertheless, up to date there is no proven link between climate change and dam failures. In addition, the claim of an increased frequency of dam failures in has been criticized in terms of selection of time frame, meaning that a 70 year time span might ”mask decennial spikes” [11].
2.3.2 Environmental impacts of tailing dam failures

Extraction of ore creates large amounts of tailing material. According to mining operational data, the volume of tailing generated range between 97-99 percent of the total amount of ore that is produced [6]. Therefore, as tailings often contain potential hazardous contaminants, it is crucial to prioritize reasonable and responsible safety management of these large volumes of tailing material in order to forestall them to enter groundwaters, rivers, lakes, and winds. For example, seepage control design in tailing dams has been out of great importance in order to retain potential contaminants in the impoundments [2]. Despite, when mining organizations fail in dam design and tailing management the results are often accidents with severe and irreversible environmental, social, and economical consequences [6].

2.4 The Daxigou Mine Tailing Dam Project

2.4.1 The Zhaoli Ditch Tailing Dam

The analysis in this thesis has been performed on the proposed Zhaoli ditch tailing dam. The tailing dam belongs to the Shaanxi Daxigou mine which is located the Shaanxi province in northern China. The iron ore reserves in the mine is estimated to 302 million tonnes, and 97 percent of these reserves can be classified as siderite [12]. Siderite is a mineral composed by iron(II)
carbonate with the chemical formula $FeCO_3$ [13].

The mine is owned by the Chinese company Shaanxi Daxigou Mining Co which collaborates with Beijing Hong ye Aetna Center for Environmental Technology of Rock and Soil and Tsinghua University in order to perform laboratory testings on the tailing material, evaluate the design and to simulate different working conditions for the dam. The planning and design of the dam started in 2012 and yet in 2016 there is no set date when the construction will start.

![Location of the Zhaoli Ditch Tailing Dam](image)

**Figure 8**: Location of the Zhaoli Ditch Tailing Dam. Source: Google Maps.

### 2.4.2 Design and geometry

The Zhaoli ditch tailing dam is located about 5 km west of the Shaanxi Daxigou mine. The dam is designed to be constructed according to the upstream method with a final accumulated height of approximately 1130 meters above sea level, a capacity of 135 million cubic meter of tailing slurry and a total service life of 18.5 years [12]. A cross section of the dam is shown in figure ??.
Figure 9: The Zhaoli Ditch Tailing Dam in cross section: 1) Dams ground line; 2) Starter dam; 3) Starter dams crest; 4) Final height of dam; 5) Coarse Tailings; 6) Coarse tailings; 7) Fine tailings; 8) Tailings slurry. The red line is the intersection between foundation layer 1 and foundation layer 2 [14].

The ground line of the dam is located about 940 meter above sea level and the starter dams crest is designed to raise 1030 meter above sea level. The dam is planned to be raised in five different steps in order to reach its final total height of 190 meters, which is 1130 meter above sea level.

In this thesis the focus will be on two different construction raises; one at a dam height of 110 meters (1050 meters above sea level) and the other at the final dam height of 190 meter (1130 meters above sea level).

2.4.3 Soil parameters

The Zhaoli ditch tailing dam can be divided into layers with different tailing material properties. The soil parameters for the different layers was partly provided by the mining company and partly obtained throughout laboratory testings at Tsinghua University. Parameters that could not be obtained from the mining company or laboratory testings were assumed to be in accordance to the soil parameters from the Yuhe Village iron ore tailing dam, a project similar to the Zhaoli ditch tailing dam [15]. The soil parameters are presented in Appendix II - Soil parameters.
3 Theory

3.1 Seepage Theory

3.1.1 Seepage in embankment dams

Since the materials used in the tailing dam are permeable there will be seepage, which is water flowing through the pores between the soil particles. Seepage through embankment dams is a required feature of the dam in order to maintain the slope stability [16]. It is also desired to keep all the seepage within the dam body. If there would be no seepage water would start to accumulate in the dam and eventually risk to seep out onto the downstream slope. This can result in a gradual erosion of the slope, which can lead to a dam failure [16]. To avoid this a filter can be used through which the water can seep and thus drain the dam before problems occur. A filter is a layer of material with a higher coefficient of permeability so that water can seep through it at a higher flow rate than surrounding materials [16]. Seepage can also cause erosion within an embankment in places where a high hydraulic gradient is present. In the case erosion do occur within a dam voids can be created. These voids take the form of channels, also pipes, which impairs the dam stability [16].

3.1.2 Darcy’s empirical law

The software used to simulate seepage in the tailing dam, SEEP/W, is only formulated for water flow, in both saturated and unsaturated soils, that follows Darcy’s law [17]. When water is seeping through a porous medium, such as the materials used in embankment dams, Darcy’s law is used to describe the flow. Darcy’s law is given by

\[ q = A k i \] (1)

or

\[ v = \frac{q}{A} = ki \] (2)

where

\( q \) = rate of water flow in volume per unit time
\( A \) = cross-sectional area through which the water flows
\( k \) = coefficient of permeability
\( i \) = the hydraulic gradient, and
\( v \) = the discharge velocity.

The permeability coefficient \( k \) varies mainly with the size of pores in the soil and the pore size depends on the particle size, particle shapes and the structure of the soil [16]. In other words, if the soil particles are small the
value of \( k \) also tends to be small. In coarse-grained soils the permeability coefficient [18] decreases significantly if a small fraction of fines are present. Since the coefficient of permeability depends on the viscosity of the water it will also vary with the temperature of the water, though this has not been taken into consideration in this project due to lack of temperature data for the area where the dam will be situated. The coefficient of permeability can be calculated as

\[
k = \frac{\gamma_w}{\eta} K
\]

where
- \( \gamma_w \) = specific weight of the water
- \( \eta \) = viscosity of water, and
- \( K \) = absolute coefficient depending on the soil skeleton \([m^2]\).

For two-dimensional seepage SEEP/W is using the governing differential equation

\[
\frac{\partial}{\partial x}(k_x \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y}(k_y \frac{\partial H}{\partial y}) + Q = \frac{\partial \theta}{\partial t}
\]

[17] where
- \( H \) = the total head,
- \( k_x \) = the hydraulic conductivity in the x-direction,
- \( k_y \) = the hydraulic conductivity in the y-direction,
- \( Q \) = the applied boundary flux,
- \( \theta \) = the volumetric water content, and
- \( t \) = time.

When a steady state is assumed the change in volumetric water content in any given elemental volume is zero since the amount of water entering and exiting is the same [17].

### 3.2 Stress-Strain Theory

#### 3.2.1 Load and deformation

A plane in soil can be subjected to normal stresses and shear stresses. Normal stress \( \sigma \) is acting perpendicular to the plane while a shear stress \( \tau \) is acting along the plane. When a body is subjected to a stress it will deform and this deformation is called strain. There is a relationship between the stress a body is subjected to and the deformation it causes, which can be described using Hooke’s law which is shown in paragraph **3.3.1.6 Hooke’s Law** below.

#### 3.2.1.1 Stress

There are several sources of stress in an embankment dam. The self-weight
of the materials in the dam, pore water pressure as well as the weight of the water that the materials are containing. If there is seepage in the dam it will cause some additional stress in the direction of the seepage due to frictional forces between the water and the soil particles.

### 3.2.1.2 Effective stress

Effective stress is the result of inter particle contact and is the stress transmitted through the skeletons of the soil particles. The principle of effective stress is only valid for fully saturated soils and is a function of pore water pressure \( u \), total normal stress \( \sigma \) with the relationship

\[
\sigma = \sigma' + u
\]

, where \( \sigma = h_1\gamma + h_2\gamma_{sat} \) and the pore water pressure \( u = h_2\gamma_w \), \( \gamma_w \) is the unit weight of water, \( \gamma \) is the dry unit weight, \( \gamma_{sat} \) is the saturated unit weight, and \( h_1 \) and \( h_2 \) are the height of the dry and saturated material respectively [19][16].

### 3.2.1.3 Influence of seepage on effective stress

As water is seeping through a soil a force called the seepage force is acting on the soil particles. The seepage force arises due to the frictional drag the soil particles are subjected to due to the viscosity of the water [16]. Effective stress is governed by the resultant body force which is the sum of the forces acting in the plane of interest. The resultant body force is the combination of the gravitational forces and the seepage force [16]. If a load is suddenly applied so that the pore water does not have time to drain out, there will be an increase in excess hydrostatic pressure. This occurs if the coefficient of permeability is low, hence there is not enough time for the water to drain [19]. Excess hydrostatic pressures can occur when an embankment dam is flooded and the water level rises above its normal level.

### 3.2.1.4 Principal Stresses

A plane on which there is no applied shear stress is called a principal plane. The normal stress acting on such a plane is called the principal stress. When the shear stress is equal to zero we have the following expression

\[
\tau_{xy}\cos2\theta = \left(\frac{\sigma_x - \sigma_y}{2}\right) \sin2\theta
\]

Thus

\[
tan2\theta = \frac{2\tau_{xy}}{\sigma_x - \sigma_y}
\]

where \( \theta \) is the theoretical angle between the major principal plane and the plane of failure [16]. One can obtain the expressions for the two principal
stresses \( \sigma_1 \) and \( \sigma_3 \) if equation 6 is substituted into equation 7, and obtain the equations

\[
\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x + \sigma_y}{2}\right)^2 + \tau_{xy}^2} \quad (8)
\]

and

\[
\sigma_3 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x + \sigma_y}{2}\right)^2 + \tau_{xy}^2} \quad (9)
\]

where \( \sigma_1 \) is the major principal stress and \( \sigma_3 \) is the minor principal stress, and [19]

\[
\tau_{xy} = \frac{E}{2(1 + \nu)}(\epsilon_x - \epsilon_y) \quad (10)
\]

where \( \tau_{xy} \) is shear stress.

### 3.2.1.5 Young’s Modulus

Young’s modulus is the ratio between the stress acting along an axis and the strain along that same axis. Young’s modulus, also called E-modulus, is the gradient of the curve obtained when plotting the strain as a function of stress. Since SIGMA/W sees the effective stress as the total stress it also uses the effective E-modulus. The effective E-modulus is simply the gradient of the curve obtained when strain is plotted as a function of effective stress instead of total stress.

### 3.2.1.6 Hooke’s Law

Hooke’s law is valid for elastic, linear, ideal, isothrophic materials and describes the axial strains in terms of stress components. In a 2-dimensional problem Hooke’s law is described in the x and y direction as

\[
\varepsilon_x = \frac{\partial u}{\partial x} = \frac{1}{E}[\sigma_x - \nu(\sigma_y + \sigma_z)] \quad (11)
\]

and

\[
\varepsilon_y = \frac{\partial u}{\partial y} = \frac{1}{E}[\sigma_y - \nu(\sigma_x + \sigma_z)] \quad (12)
\]

respectively, where \( \nu \) is the Poisson’s ratio and \( E \) is Young’s modulus for the material [16]. Poisson’s ratio is the negative ratio between transverse strain \( \epsilon_{\text{trans}} \) and \( \epsilon_{\text{axial}} \).

Since the tailing materials, as well as the starter dam material used in the dam construction have been shown not to have a linear Young’s modulus, but a non-linear elastic, Hooke’s law is not valid. For the other materials, i.e. the starter dam; foundation layer 1; foundation layer 2 and the slurry, linear elasticity has been assumed, thus a linear-elastic model and Hooke’s law are used by SIGMA/W [20]. More information about material models and properties can be found in section 3.2.2 Material models and properties.
3.2.1.7 Consolidation theory
Consolidation is a process where the volume of a saturated soil is reducing because of drainage of the pore water, without it being replaced by air [19]. The drainage occurs because of an excessive pore water pressure which arise when the soil is subjected to a compressive load. The reverse process is called swelling, and occurs when water is entering the soil due to a negative excessive pore water pressure [16]. The consequence of consolidation and swelling is that the surface of the soil displaces. Consolidation and swelling are a usually slow processes because of the low permeability of the soil which leads to a slow drainage of the pore water [19]. Since consolidation is a process that is going on over relatively long periods of time the consequences can be that a structure deforms after it has been built.

3.2.1.8 Compressibility and settlement
The laboratory testing of the tailing materials gave data for the confined Young’s modulus, \( E_s \). To calculate the effective Young’s modulus, \( E \), the following equation was used

\[
E_s = \frac{E(1 - \nu)}{(1 + \nu)(1 - 2\nu)}
\]  

The initial Young’s modulus, \( E_i \), and tangential Young’s modulus, \( E_t \), were also obtained by laboratory testings. These different values of the E-modulus are used in two different material models in the Sigma/W simulations, which are presented in section 3.2.2 Material models and properties below.

Laboratory results and calculations of the different E-modulus are presented in Appendix II.

3.2.2 Material models and properties

Soils have different stress-strain behaviour which depends on factors such as density, water content, structure, drainage condition, loading, confining pressure, and shear stress. Therefore, it is convenient to describe these stress-strain behaviours of soils in different material models. The linear-elastic and non-linear elastic hyperbolic models that are used in the Sigma/W simulations are presented below.

3.2.2.1 Linear elastic model
In the linear elastic material model of a soil the stresses, \( \sigma \), are directly proportional to the strains, \( \epsilon \), as shown in figure 10.
The proportionally constants in the linear-elastic model are Young’s modulus, $E$, and Poisson’s Ratio, $\nu$. The relationship between these can mathematically be expressed by the equation

$$
\begin{bmatrix}
\sigma_x \\
\sigma_y \\
\sigma_z \\
\tau_{xy}
\end{bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix}
a_{1-\nu} & \nu & \nu & 0 \\
\nu & a_{1-\nu} & \nu & 0 \\
\nu & \nu & 1-\nu & 0 \\
0 & 0 & 0 & \frac{1-2\nu}{2}
\end{bmatrix} \begin{bmatrix}
\tau_x \\
\tau_y \\
\tau_z \\
\gamma_{xy}
\end{bmatrix}
$$

(14)

In an embankment construction some extent of deformation and yielding can be acceptable. Since a simple linear-elastic model only use the linear relationship of the soils, which often only represents the initial portion of the stress-strain curve, the model could underestimate the displacements in the construction. Therefore, a non-linear elastic hyperbolic model is required to obtain more realistic results of a probable displacement [20].

### 3.2.2.2 Non-linear elastic hyperbolic model

It is commonly found that, as failure conditions are approached the stress-strain behavior of a soil becomes non-linear. This non-linear relationship between stress and strain are shown in figure 11.

Figure 11: Stress and strain relationship in the non-linear elastic hyperbolic model [20].
In order to represent this non-linear soil behaviour a hyperbolic soil model was formulated by Duncan and Chang (1970) [21]. In this formulation the non-linear stress curve can be expressed by the equation

$$E_i = K_L P_a \left( \frac{\sigma_3}{P_a} \right)^n$$  \hspace{1cm} (15)

where

- $E_i$ = initial tangent modulus as a function of confining stress, $\sigma_3$
- $K_L$ = loading modulus number
- $P_a$ = atmospheric pressure (normalizing parameter)
- $\sigma_3$ = confining stress
- $n$ = exponent which is defining the influence of the confining pressure on $E_i$

When a soil is influenced by an increasing shear stress its behaviour is governed by the tangent modulus, $E_t$. $E_t$ are related to both stress difference and strain, but as stresses can be calculated more accurately with Duncan and Chang expressed $E_t$ in terms of stress only [21]. The tangent E-modulus was defined as a function of soil properties, triaxial stress, $\sigma_1 - \sigma_3$ and confining stress, $\sigma_3$ as in the following equation [21][20]

$$E_t = \left( 1 - \frac{R_f \sigma_1 - \sigma_3 1 - \sin \phi}{2(c \cos \phi + \sigma_3 \sin \phi)} \right)^2 E_i$$ \hspace{1cm} (16)

where:

- $E_i$ = initial tangent modulus
- $E_t$ = tangent modulus
- $\phi$ = friction angle of soil
- $c$ = cohesive strength of soil
- $R_f$ = ratio between the hyperbolic curves asymptote and maximum shear strength (usually between 0.75-1.0)
- $\sigma_1$ = major principal stress $\sigma_3$ = minor principal stress

### 3.3 Slope Stability Theory

Finding potential failure surfaces and their locations are necessary in order to determine the stability of a tailing dam embankment. The analysis of slope stability also requires knowledge about what computational methods are available when computing the factor of safety, which is the end product when analyzing a given potential failure surface. Therefore, in the following sections types of slip surfaces and computational procedures that are commonly used for slope stability assessments are presented.

#### 3.3.1 Types of slip surfaces

**Rotational slips**

Rotational slip failure surfaces, also called circular slip surfaces, can often be
approximated by a circular arc [16]. The resisting shear strength ($\tau_f$) in the embankment has to be greater than the shear stress ($\tau_{mob}$) created by the self-weight of the soil in the slope of an embankment. If not, the slip occurs when the shear stress on the failure circle is greater, or equal to, the shear strength of the soil [22]. This occurs when the factor of safety is less than one, which is explained in more detail in section 3.4.2 Factor of safety.

The failing mass will rotate in a shape of an arc around an instantaneous center of rotation [16]. There are an infinite number of possible slip circles in an embankment, two of the possible ones can be seen in figure 12. However, it is at the slip circle with the lowest factor of safety that the failure most likely will occur[22].

Figure 12: Potential rotational slip surfaces: both circular and non-circular [16]

Translational and compound slips
Translational and compound slips may have a straight and/or curved failure surface, as shown in figure 13. These slips often occurs when the soil in the failure surface layer is weaker than the underlying, adjacent soil layer, causing the weaker soil layer to slip. For translational slips the slipping soil layer often can be found on a relatively shallow depth, whereas for compound slips the slip occurs on greater depths [16].

Figure 13: Translational (left) and compound (right) slip surfaces [16]
3.3.2 Limit equilibrium fundamentals

Limit equilibrium methods are commonly used in computational analysis of tailing dam slope stability. The basics are that forces, moments, or stresses that are resisting movement of a rigid mass are compared to those that can cause unstable motion.

The most commonly used limit equilibrium computational method is the method of slices. This limit equilibrium technique divides the rigid mass above a trial failure surface into vertical slices, as illustrated in figure 14.

![Illustration of the method of slices at the rotation center of a slip surface](image)

Figure 14: Illustration of the method of slices at the rotation center of a slip surface

where:
- \( W \) = self-weight of an individual slice
- \( N \) = normal force
- \( \tau_f \) = shear strength of resisting slide
- \( E_i \) = normal forces on the sides
- \( X_i \) = shear forces on the sides
- \( b \) = distance between centre of slip circle and centre of the slice radius
- \( r \) = radius of slip circle
- \( \alpha \) = angle between horizontal and base of the slice
- \( L \) = length of the base of the slice

Limit equilibrium methods calculates the factor of safety (described in section 3.3.3 Factor of safety below) for a specific slip surface, which can be used when analysing the stability of the tailings dam construction in this thesis[16].
The limit equilibrium methods used in SLOPE/W simulations are presented below:

**Bishop’s simplified method**
Bishop’s simplified method assumes there is no shear force between the different interslices but that the normal forces between the slices are acting horizontally on each slice [16].

Initially the factor of safety has to be estimated in the Bishop’s simplified method in order to make the first iteration. Iterations will then continue until the obtained factor of safety is within the specified value of tolerance for the calculation. The value of the factor of safety that is obtained by using Bishop’s simplified method is similar to calculations made by using other limit equilibrium methods (differs by 0-6 percent) [23].

**Janbu generalized**
The Janbu Generalized method, or Janbu, assumes there is a hydrostatic stress distribution on each of the interslices. This results in lines of thrust on each side of the slices which combined with the moment about the base of each slice gives the magnitude of the force acting on the interslice. In SLOPE/W this method only computes the factor of safety using the force equilibrium [24].

**Morgenstern-Price**
The Morgenstern-Price method calculates the limit equilibrium of both force and moment for each interslice on the slip surface. The method can be used for both circular and non-circular slip surfaces and compounds [16].

**Fellenius (or Swedish Arc) method**
This simple method considers both force and momentum equilibrium for each slice in the slope mass. There is assumed to be no interaction between interslices in this method and for each slice the resultant force is zero [24].

Compared to other methods of analysis the Fellenius method underestimates the factor of safety, usually within a range of 5-20 percent. Today this method is not recommended in practice [16].

**Limitations of limit equilibrium methods**
Most limit equilibrium methods make the assumption that the factor of safety is the same for all slices. Although, this could only be true in the moment of failure when the factor of safety for every slice is equal to one [23]. Errors in slope stability analysis do not have to come from the calculating methods of the factor of safety, they often have their origins in the input data. Therefore, proper input data is of higher importance than the computational
methods used. When performing any analysis, it is suggested that a hierarchy of growing complexity of the processes should be employed in order to detect possible errors on the way [2].

3.3.3 Factor of safety

Definition

The factor of safety, FoS, is defined as the ratio of the shear strength ($\tau_f$) divided by an mobilized shear strength ($\tau_{mob}$), as can be seen in the following equation [22]

$$ FoS = \frac{\tau_f}{\tau_{mob}} \quad (17) $$

When factor of safety is greater than unity, the stresses resisting failure are greater than the stresses that are creating the failure. Hence, when the factor of safety is larger than the value of one, the embankment is considered stable. The factor of safety is instantaneous and depends on the properties of the soil that is used when constructing an embankment. This means that the factor of safety will change with time due to factors affecting the embankment, such as consolidation, change of loads due to rise of the embankment height, etc [2][16].

When the value of the factor of safety is equal to one, the resisting forces are equal to the forces that are creating the failure, causing a critical failure surface on the embankment slope. When the forces that are creating failure become larger than the resistance forces, the critical surface on the embankment slope will fail, resulting in a movement of the soil mass [16].

Recommended factor of safety

The factor of safety is dependent on the consequence of the failure, material properties, and subsurface conditions. There is no set standard value of factor of safety, although there are some published recommendations for minimum factors of safety under different operation conditions. According to the Chinese national standard “Code for design of tailings facilities” (GB 50863-2013) [25] and ”Technical code for geotechnical engineering of tailings embankments” (GB50547-2010) [26] the value of the factor of safety should not be less than the specified values in table 2.

Table 2: Tailing dam factor of safety specification values

<table>
<thead>
<tr>
<th>Computational Method</th>
<th>Operation condition</th>
<th>Minimum Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fellenius</td>
<td>Normal conditions</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>Flooded conditions</td>
<td>1.15</td>
</tr>
<tr>
<td>Bishop’s Simplified</td>
<td>Normal conditions</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>Flooded conditions</td>
<td>1.25</td>
</tr>
</tbody>
</table>
The given specification values of the minimum factor of safety in table 2 will be used as a guideline when analyzing the slope stability with SLOPE/W in this thesis.

### 3.3.4 Sensitivity Analysis

If the parameters used in a simulation are not very precise but somewhat estimated as in this thesis, a probabilistic analysis or sensitivity analysis can be conducted in order to find out how the system responds to changes in the parameters. Both methods are based on varying the values of certain parameters so that one can observe what the outcome of the simulation would have been using other values for the parameters. In the probabilistic analysis a parameter is varied according to a known distribution such as a normal distribution. If, however the probability distribution over which the given parameter is varying is not known, a sensitivity analysis can be made [24]. In a sensitivity analysis a certain parameter is varied with a uniform distribution between a set minimum and a maximum value. In this thesis only a sensitivity analysis have been conducted regarding the angle of shearing resistance $\phi$. This is because the shearing resistance angle is the parameter that is most likely to vary, and it will also have the greatest impact on the stability of the dam [27]. When the simulation is being carried out the model is tested for all possible values of $\phi$ according to the sensitivity analysis [24].
4 Simulation and modeling in GeoStudio\textsuperscript{TM}

4.1 General

4.1.1 About GeoStudio\textsuperscript{TM}

The numerical analysis in this thesis has been made based on models and simulations carried out in the computer software GeoStudio\textsuperscript{TM}. GeoStudio\textsuperscript{TM} is a widely used computer software for geotechnical and earth science problem modeling.

SEEP/W, SIGMA/W and SLOPE/W, which are tools within GeoStudio\textsuperscript{TM}, were used in order to analyze the given construction design of the dam embankment in this thesis.

4.1.2 Dam model and material parameters

The CAD drawing of the Zhaoli ditch tailing dam was translated into GeoStudio\textsuperscript{TM}. Regions were created and material layers were assigned to each region according to the embankment design that was given by the mining company [14].

In the beginning of the modelling process simple or estimated material parameters were used as input data. This was according to directions from GeoStudio\textsuperscript{TM} instructions [24] and the purpose was to verify that the models worked properly. When the models were verified to similar studies, the accurate material parameters were substituted into the model in order to refine the analysis. Material parameters used in the simulations will be presented in the SEEP/W, SIGMA/W and SLOPE/W sections.

4.1.3 Simulated scenarios

Two dam raise scenarios were simulated in GeoStudio\textsuperscript{TM}. In the first scenario the dam embankments total height was set to be 110 meter, which is 1050 meters above sea level. This dam raise includes the height of the starter dam and approximately 3 additional perimeter dikes. This scenario will be referred to as the 110 m-dam scenario in this thesis. The GeoStudio-model of the 110 m-dam is shown in figure 15.

In the second scenario the dam was assumed to be at its final construction height of 190 meter, which is 1130 meter above sea level. This scenario will be referred to as the 190 m-dam scenario in this thesis. The GeoStudio-model of the 190 m-dam is shown in figure 16.
Figure 15: Model of the 110m-dam. The enlargement shows a more detailed view of the first foundation layer in dark grey.

Figure 16: Model of the 190m-dam

In both figure 15 and figure 16 the beach slope was 0.7 percent. Furthermore, every scenarios were simulated and analyzed for different water level operation conditions, as can be seen in table 3.

Table 3 shows the two scenarios at different operating conditions where the location (maximum water level) of the ponded water changes. The estimated locations of the ponded water during normal and flooded conditions are expressed in height, in meters, measured from the dam toe.

Table 3: Simulated scenarios at different operating conditions. The Normal and Flooded condition below are representing the highest estimated water level, in meters, in the two dam raises

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Total height of dam [m]</th>
<th>Normal condition [m]</th>
<th>Flooded condition [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>110 m-dam</td>
<td>110</td>
<td>102</td>
<td>107.7</td>
</tr>
<tr>
<td>190 m-dam</td>
<td>190</td>
<td>184</td>
<td>185.2</td>
</tr>
</tbody>
</table>
4.2 SEEP/W

4.2.1 Purpose

Simulations in SEEP/W was conducted in order to locate the phreatic surface, determine the pore water distribution and penetration, as well as obtaining the velocity and the direction of the seepage in the dam scenarios during normal and flooded conditions. As seepage control is crucial for a stable and safe embankment, the seepage analysis can provide information and data for further stability analysis in SLOPE/W [4][24].

4.2.2 Input data and boundary conditions

Seepage calculations were based on saturated/unsaturated and steady/unsteady soil seepage equations with the fixed grid finite element method. The mesh properties were set to 5071 nodes and 5153 elements with an approximate global element size of 5 meters. The boundary conditions for SEEP/W were set according to the table and figures in Appendix II.

The input conductivity for the different dam layers are shown in table 4. For the starter dam, coarse and fine tailings conductivity graphs have been created from obtained laboratory test material parameters. These graphs can be found in Appendix II.

Table 4: Tailings material conductivity parameters used in SEEP/W.

<table>
<thead>
<tr>
<th>Dam Material Layer</th>
<th>Conductivity, $k$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Starter dam</td>
<td>see Appendix II</td>
</tr>
<tr>
<td>Coarse</td>
<td>see Appendix II</td>
</tr>
<tr>
<td>Fine</td>
<td>see Appendix II</td>
</tr>
<tr>
<td>Foundation layer 1</td>
<td>$1.0 \times 10^{-6}$</td>
</tr>
<tr>
<td>Foundation layer 2</td>
<td>$1.0 \times 10^{-9}$</td>
</tr>
<tr>
<td>Slurry</td>
<td>$1.0 \times 10^{-9}$</td>
</tr>
</tbody>
</table>

4.2.3 Steady-state and transient analysis

Steady-state and transient seepage analysis were conducted for the 110 m-dam and the 190 m-dam scenarios. Simulation began with a steady-state analysis at normal conditions in order to obtain a defined steady-state seepage process through the embankment and continued with a transient analysis at flooded conditions to obtain knowledge about how the seepage process migrates with time. Water levels at normal and flooded conditions used in the simulations can be found in 3 in section 4.1.3 Simulated scenarios.

The transient analysis simulates the seepage at flooded conditions over a
three days period (259,200 seconds). Using a three days time period in simulations is based on directions according to embankment design which states that pumps would be used to get rid of excess water in the dam if the seepage has not gone back to normal conditions after three days. In the transient analysis, an instant raise of the water level is assumed and therefore the highest water levels will be reached at the first seconds of the simulations.

4.3 SIGMA/W

4.3.1 Purpose

Simulations in SIGMA/W are performed in order to analyze the resulting stresses and deformation in the dam scenarios under normal and flooded conditions.

4.3.2 Input data and boundary conditions

The load and deformation in the dam embankment were simulated using the linear elastic and nonlinear elastic hyperbolic material models. The linear elastic material model was used for the starter dam layer, the foundation layer 1, the foundation layer 2, and the slurry. The nonlinear hyperbolic material model was used for the coarse and the fine layers. The boundary conditions were specified with a zero value x-displacement along both the left and right boundary edges and a zero value x/y-displacement along the base boundary edge [20]. A table and figures of boundary conditions used in SIGMA/W can be found in Appendix II.

4.4 SLOPE/W

4.4.1 Purpose

Simulations in SLOPE/W were carried out to find potential slip surfaces and the minimum factor of safety for the dam scenarios during normal and flooded conditions.

4.4.2 Input data

The Mohr-Coulomb model was applied to all the material layers in the dam. The Mohr-Coulomb model make use of five main material parameters. These parameters has been obtained from the mining company and laboratory tests at Tsingua and can be found in Appendix II. Furthermore, the poisson’s ratio for the different materials has been assumed to be the same values as in the Yuhe Village tailing dam project [15]. This is true for all materials except
the tailing slurry where the poisson’s ratio was set to 0.45, as was obtained from a laboratory testings report [28]. In order to get results from the models Morgenstern-Price, Bishop, Janbu and Fellenius data from the simulation in SEEP/W is used as a parent analysis.

The used material parameters in the simulated dam model are presented in table 5.

<table>
<thead>
<tr>
<th>Dam Material Layer</th>
<th>Unit weight, $\gamma_{sat}$ [kN/m$^3$]</th>
<th>Poisson’s Ratio, $\nu$</th>
<th>$c'$ [kPa]</th>
<th>$\phi'$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Starter dam</td>
<td>23.4</td>
<td>0.3</td>
<td>0</td>
<td>40</td>
</tr>
<tr>
<td>Coarse</td>
<td>21.4</td>
<td>0.25</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>Fine</td>
<td>21.5</td>
<td>0.35</td>
<td>0</td>
<td>34</td>
</tr>
<tr>
<td>Foundation layer 1</td>
<td>22.0</td>
<td>0.2</td>
<td>1000</td>
<td>30</td>
</tr>
<tr>
<td>Foundation layer 2</td>
<td>26.0</td>
<td>0.18</td>
<td>1000</td>
<td>50</td>
</tr>
<tr>
<td>Slurry</td>
<td>22.0</td>
<td>0.45</td>
<td>0</td>
<td>15</td>
</tr>
</tbody>
</table>

4.4.3 Slip surface

The Entry-and-Exit method is used in SLOPE/W, where a potential sliding mass is divided into sections, starting where the slip surface enters the ground surface on the dam crest and ends where the slip surface exits the ground surface on the dam toe. The number of desired slices were specified to the default value of 30, as recommended from the SLOPE/W manual since a greater number of slices than 30 seldom contribute to a more specified calculation of the factor of safety[24]. All assumed slip surfaces must enter and exit on and along the ground surface of the dam otherwise they will be considered inadmissible by SLOPE/W[24].

4.4.4 Sensitivity Analysis Simulations

Geostudio has a built in function to perform both probabilistic and sensitivity analyses. As the shearing resistance angle $\phi$ was assumed to have the greatest impact on the factor of safety for potential slip surfaces [27] it was the only parameter being varied. For the coarse tailing material it was varied between 20 and 58 where 39 was assumed to be the most likely value. The fine tailing material was varied between 15 and up to 53 where 34 was assumed to be the most likely value. As the critical slip surfaces were found to occur in the starter dam, a sensitivity analysis for this has also been done, however the starter dam is made out of rock and is not very likely to have a shearing resistance angle diverging too much from it’s assumed value [27].
5 Results

The results from the simulations are presented divided into the different simulations that has been conducted. The results are presented in the same order as they were conducted starting with the seepage analysis in SEEP/W. The seepage analysis was conducted to provide data to the load and deformation analysis done in SIGMA/W and also the analysis done in SLOPE/W since it is using data from SIGMA/W. The results that SIGMA/W and SLOPE/W are using from SEEP/W are pore water pressure, seepage velocity, volumetric water content, etc.

5.1 Seepage analysis

This section includes the result from the seepage analysis that has been conducted for the 110-m dam scenario and 190-m dam scenarios. Both scenarios has been simulated with normal water level conditions as well as a flooded water level condition. Note that the results from the flooded conditions are from a transient analysis, and the normal conditions are simulated using a steady state analysis. In this section figures showing pore-water pressure, total head, pressure head and seepage velocity are presented.

5.1.1 110 m-dam during normal conditions

The dark blue line in figure 17, 18, 19, and 20 below represents the phreatic surface in the 110-dam during normal conditions. The phreatic surface is low throughout the dam and exits at the dam toe, which is desirable for stability.

Figure 17: Pore-water pressure in the dam, in kPa. The phreatic surface is shown as the thick blue line, which is also the iso-line where the pore-water pressure is zero. The pore-water pressure is negative above the phreatic surface and positive below the phreatic surface.
Figure 18: Pressure head in the dam, in kPa. This shows how flowing water within the dam builds up a negative pressure head above the phreatic surface and a positive pressure head below the phreatic surface.

Figure 19: Total hydraulic head in the dam, in metres. This shows that the total head on the right is larger than the total head on the left. The total head difference creates a water flow from the right to the left.

Figure 20: Velocity vectors (the black arrows) for the seeping water and the total flux through the dam. The highest velocity was $1.922 \times 10^{-7} m/s$. The velocity vectors are hard to distinguish in this figure, but are concentrated to foundation layer 1, see figure 25 for magnification. The water flow was measured across the blue vertical lines, also called flow lines. The thick blue line represents the phreatic surface.
5.1.2 110 m-dam during flooded condition

The dark blue line in figure 21, 22, 23, and 24 below represents the phreatic surface in the 110-dam during flooded conditions. The phreatic surface is now beginning at a higher water level where it decreases rapidly and then decreases slowly at a low level throughout the dam before exiting at the dam toe, which is desirable for stability.

Figure 21: Pore-water pressure in the dam, in kPa, 10 seconds after the flooding occurred. The phreatic surface is shown as the thick blue line, which is also the iso-line where the pore-water pressure is zero. The pore-water pressure is negative above the phreatic surface and positive below the phreatic surface.

Figure 22: Pressure head in the dam, in kPa, 10 seconds after the flooding occurred. This shows how flowing water within the dam builds up a negative pressure head above the phreatic surface and a positive pressure head below the phreatic surface.
Figure 23: Total hydraulic head in the dam, in metres, 10 seconds after the flooding occurred. This shows that the total head on the right is larger than the total head on the left. The total head difference creates a water flow from the right to the left.

Figure 24: Velocity vectors (the black arrows) for the seeping water and the total flux through the dam, 10 seconds after the flooding occurred. The highest velocity was $1.922 \times 10^{-7} \text{m/s}$. The velocity vectors are hard to distinguish in this figure, but are concentrated to foundation layer 1, see figure 25 below for magnification. The water flow was measured across the blue vertical lines, also called flow lines. The thick blue line represents the phreatic surface.

Figure 25: Magnification of the seepage velocity vectors magnitude and direction. Note the clear difference between the magnitude of the seepage velocity in the coarse tailing material and in foundation layer 1.
5.1.3 190 m-dam during normal conditions

The dark blue line in figure 26, 27, 28, and 29 below represents the phreatic surface in the 190-dam during normal condition. The phreatic surface decreases slowly throughout the fine tailing layer in the dam. The phreatic surface is still on a considerable low level throughout the dam before exiting at the dam toe, which is desirable for stability.

Figure 26: Pore-water pressure in the dam, in kPa. The phreatic surface is shown as the thick blue line, which is also the iso-line where the pore-water pressure is zero. The pore-water pressure is negative above the phreatic surface and positive below the phreatic surface.

Figure 27: Pressure head in the dam, in kPa. This shows how flowing water within the dam builds up a negative pressure head above the phreatic surface and a positive pressure head below the phreatic surface.
Figure 28: Total hydraulic head in the dam, in metres. This shows that the total head on the right is larger than the total head on the left. The total head difference creates a water flow from right to left.

Figure 29: Velocity vectors (the black arrows) for the seeping water and the total flux through the dam. The highest velocity was $2.09 \times 10^{-6} \text{ m/s}$. The velocity vectors show that the seepage is still concentrated in foundation layer 1, but there is also considerable seepage occurring in the fine tailing material layer, between the phreatic surface and the slurry.

Figure 30: Magnification of the seepage velocity vectors magnitude and direction (the black arrows). Note the clear difference between the magnitude of the seepage velocity in the coarse and fine tailing material, and in foundation layer 1.
5.1.4 190 m-dam during flooded conditions

The dark blue line in figure 32, 33, and 34 below represents the phreatic surface in the 190-dam after the flooded condition occurred. The phreatic surface is now beginning at a higher water level where it decreases slowly throughout the fine tailing layer in the dam. The phreatic surface is still on a considerable low level throughout the dam before exiting at the dam toe, which is desirable for stability.

Figure 31: Pore-water pressure in the dam, in kPa, 10 seconds after the flood occurred. The phreatic surface does not show in this figure, but it should be located along the iso-line where the pore-water pressure is zero. The pore-water pressure is negative above the phreatic surface and positive below the phreatic surface.

Figure 32: Pressure head in the dam, in kPa, 10 seconds after the flood occurred. This shows how flowing water within the dam builds up a negative pressure head above the phreatic surface and a positive pressure head below the phreatic surface.
Figure 33: Total hydraulic head in the dam, in metres, 10 seconds after the flooding occurred. This shows that the total head on the right is larger than the total head on the left. The total head difference creates a water flow from right to left.

Figure 34: Velocity vectors (the black arrows) for the seeping water and the total flux through the dam, 10 seconds after the flood occurred. The highest velocity was $2.09 \times 10^{-6} \text{ m/s}$. Because of the flood, the phreatic surface has now moved further up to the left. There is also now a vertical seepage which is making the phreatic surface move to the left within the dam over time. See figure 37 and 38 for change over 3 days. The seepage is still concentrated in foundation layer 1, but there is also considerable seepage occurring in the fine tailing material layer, between the phreatic surface and the slurry.
Figure 35: Magnification of the seepage velocity vectors magnitude and direction through the dam, 10 seconds after the flood occurred. The black vertical arrows shows the vertical seepage.

Figure 36: Magnification of the seepage velocity vectors magnitude and direction. Note the clear difference between the magnitude of the seepage velocity in the coarse and fine tailing material, and in foundation layer 1.
Figure 37: Velocity vectors (the black arrows) for the seeping water and the total flux through the dam, 3 days after the flood occurred. The highest velocity was $2.09 \times 10^{-6}$ m/s. Because of the flood, the phreatic surface has now moved further up to the left. But, as mentioned in figure 34, the vertical seepage has made the phreatic surface move to the left during the simulated time span of 3 days. The seepage is still concentrated in foundation layer 1, but there is also considerable seepage occurring in the fine tailing material layer, between the phreatic surface and the slurry.

Figure 38: Magnification of the seepage velocity vectors magnitude and direction through the dam, 3 days after the flood occurred. The black vertical arrows show the vertical seepage, which has changed slightly to the left after 3 days.

5.1.5 Summary

The results from simulations in SEEP/W are summarized in table 6.
Table 6: Results of maximum seepage velocity and total flux at the dam toe in the 110 m-dam and 190 m-dam scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Operation condition</th>
<th>Max. seepage velocity, $v_{\text{max}}$ [m/s]</th>
<th>Tot. flux at toe, $Q$ [m$^2$/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>110 m-dam</td>
<td>Normal</td>
<td>$1.92 \times 10^{-7}$</td>
<td>$9.54 \times 10^{-8}$</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>$1.92 \times 10^{-7}$</td>
<td>$9.48 \times 10^{-8}$</td>
</tr>
<tr>
<td>190 m-dam</td>
<td>Normal</td>
<td>$2.09 \times 10^{-6}$</td>
<td>$5.60 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>$2.09 \times 10^{-6}$</td>
<td>$5.60 \times 10^{-7}$</td>
</tr>
</tbody>
</table>

Under normal conditions the seepage is concentrated to the first foundation layer which is expected since it is supposed to function as a filter. In the second stage of the dam, the seepage velocity increases by a factor of 10 compared to the first stage. Likewise the total flux at the dam toe also increases roughly by a factor of 10. The maximum seepage velocity reached, which is in the 190 metre dam, was $2.09 \times 10^{-6}$ m/s which is within the limits of what the first foundation layer can handle without being damaged. During the flooded conditions in the phreatic surface naturally has a very high gradient. The phreatic surface moves in the negative x-direction during the three days of flooding, which is also expected. In the 190 metre dam there is significant seepage between the phreatic surface and the slurry, which is explained by the much lower coefficient of permeability in the slurry compared to the fine tailing material in which the phreatic surface is located. During flooded conditions in the 190 metre dam there is significant seepage in the upper layer of the fine tailing. This seepage is being diverged in the negative x direction as the phreatic surface is also moving.

5.2 Load and deformation analysis

This section includes the result from the load and deformation analysis that has been conducted for the 110-m dam scenario and 190-m dam scenarios. Both scenarios has been simulated during normal water level conditions as well as during flooded water level conditions. In this section figures shows the total stress distribution, the shear stress, the displacement in x-direction, the displacement in y-direction, and the total displacement in the dam.
5.2.1 110 m-dam during normal conditions

Figure 39: Total stress in kPa, in the x-direction in the dam. The total stress is the lowest on top of the dam (zero), then it increases in the middle of the dam as well as when the dam elevation decreases.

Figure 40: Total stress in kPa, in the y-direction in the dam. The total stress is lowest in the top area of the dam construction and increases as the dam elevation decreases.

Figure 41: Shear stress, in kPa, in the xy-direction in the dam. The greatest shear stress is located in the left regions of the dam.
Figure 42: Displacement i.e. the deformation, in metres, in the x-direction in the dam. The displacement is the greatest on top of the dam structure.

Figure 43: Displacement i.e. the deformation, in metres, in the y-direction in the dam. Again, the displacement is the greatest on top of the dam.

Figure 44: Total displacement i.e. the deformation, in metres, in the dam. This is the combination of the displacement in both the x-and y-direction. The displacement in the y direction is much greater than in the x-direction, why it does not appear to differ much from the y-displacement. The displacement is positive since it is the absolute displacement.
5.2.2 110 m-dam during flooded conditions

Figure 45: Total stress, in kPa, in the x-direction in the dam. The total stress is the lowest on top of the dam (zero), then it increases in the middle of the dam as well as when the dam elevation decreases.

Figure 46: Total stress in kPa, in the y-direction in the dam. The total stress is lowest in the top area of the dam construction and increases as the dam elevation decreases.

Figure 47: Shear stress, in kPa, in the xy-direction in the dam. The greatest shear stress is located in the left regions of the dam.
Figure 48: Displacement i.e. the deformation, in metres, in the x-direction in the dam. The displacement is the greatest on top of the dam structure.

Figure 49: Displacement i.e. the deformation, in metres, in the y-direction in the dam. The displacement is the greatest on top of the dam.

Figure 50: Total displacement i.e. the deformation, in metres, in the dam. This is the combination of the displacement in both the x-and y-direction. The displacement in the y direction is much greater than in the x-direction, why it does not appear to differ much from the y-displacement. The displacement is positive since it is the absolute displacement.
5.2.3 190 m-dam during normal conditions

Figure 51: Total stress, in kPa, in the x-direction in the dam. The total stress is the lowest on top of the dam (zero), then it increases in the middle of the dam as well as when dam elevation decreases. The greatest total stress value in the simulation is found in the middle of the dam construction.

Figure 52: Total stress in kPa, in the y-direction in the dam. The total stress is lowest in the top area of the dam construction and increases as the dam elevation decreases.

Figure 53: Shear stress, in kPa, in the xy-direction in the dam. The greatest shear stress is located in the left regions of the dam.
Figure 54: Displacement i.e. the deformation, in metres, in the x-direction in the dam. The x-displacement is the greatest inside the dam as the figure shows.

Figure 55: Displacement i.e. the deformation, in metres, in the y-direction in the dam. Just like in the 110-metre dam, the top of the dam is displaced the most in the y-direction.

Figure 56: Total displacement i.e. the deformation, in meters, in the dam. This is the combination of the displacement in both the x- and y-direction. Note that the displacement in the y direction is much greater than in the x-direction, why it does not appear to differ much from the y-displacement. The displacement is positive since it is the absolute displacement. Also, the magnitude of the displacement is much greater in the 190-metre dam compared to the 110-metre dam.
5.2.4 190 m-dam during flooded conditions

Figure 57: Total stress, in kPa, in the x-direction in the dam. The total stress is the lowest on top of the dam (zero), then it increases in the middle of the dam as well as when dam elevation decreases. The greatest total stress value in the simulation is found in the middle of the dam construction.

Figure 58: Total stress in kPa, in the y-direction in the dam. The total stress is lowest in the top area of the dam construction and increases as the dam elevation decreases.

Figure 59: Shear stress, in kPa, in the xy-direction in the dam. The greatest shear stress is located in the in the left region of the dam.
Figure 60: Displacement i.e. the deformation, in meters, in the x-direction in the dam. The x-displacement is the greatest inside the dam as the figure shows.

Figure 61: Displacement i.e. the deformation, in metres, in the y-direction in the dam. Just like in the 110-metre dam, the top of the dam is displaced the most in the y-direction.

Figure 62: Total displacement i.e. the deformation, in meters, in the dam. This is the combination of the displacement in both the x-and y-direction. Note that the displacement in the y direction is much greater than in the x-direction, why it does not appear to differ much from the y-displacement. The displacement is positive since it is the absolute displacement. Also, the magnitude of the displacement is much greater in the 190-metre dam compared to the 110-metre dam.
5.2.5 Summary

The results from simulations in SIGMA/W are summarized in table 7.

Table 7: Results of maximum shear stress and maximum displacement (displ.) in the 110 m-dam and 190 m-dam scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Operation condition</th>
<th>Max. shear stress [kPa]</th>
<th>Max. x-displ. [m]</th>
<th>Max. y-displ. [m]</th>
<th>Max. total-displ. [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>110 m-dam</td>
<td>Normal</td>
<td>368.8</td>
<td>-0.8798</td>
<td>-2.68</td>
<td>2.716</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>390.3</td>
<td>-0.871</td>
<td>-2.682</td>
<td>2.716</td>
</tr>
<tr>
<td>190 m-dam</td>
<td>Normal</td>
<td>756.8</td>
<td>-2.354</td>
<td>-6.529</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>774.8</td>
<td>-2.322</td>
<td>-6.52</td>
<td>6.783</td>
</tr>
</tbody>
</table>

The maximum shear stress for the 110 m-dam was 368.8 kPa during normal conditions, which was lower than the resulting maximum shear stress of 390.3 kPa during flooded conditions. The maximum total displacement, i.e. deformation, in the 110 m-dam was 2.716 meters during normal conditions and 2.716 during flooded conditions, with the greatest displacement in the y-direction in both.

The maximum shear stress for the 190 m-dam was 756.8 kPa during normal conditions, which was lower than the resulting maximum shear stress of 774.8 kPa during flooded conditions. The maximum total displacement, i.e. deformation, in the 190 m-dam was 6.8 meters during normal conditions and 6.783 during flooded conditions, with the greatest displacement in the y-direction in both.

In the 110 m-dam scenario the maximum displacement in x- and y-direction is located in the top region of the dam during both normal and flooded conditions. In the 190 m-dam scenario the maximum displacement in x-direction is located in the center of the dam, close to the slope (see figure 54 and 60) meanwhile the maximum displacement in y-direction is located in the top region of the dam.

5.3 Slope stability analysis

The results of potential slip surfaces with corresponding minimum factor of safety for each scenarios at normal and flooded conditions are presented below.
5.3.1 110 m-dam during normal conditions

Figure 63: Slip surfaces during normal conditions. The green area represents the potential slip surface and the number its corresponding minimum factor of safety. Results are shown for a) Bishop’s simplified; b) Fellenius; c) Morgenstern & Price; and d) Janbu Generalized limit equilibrium methods.

5.3.2 110 m-dam during flooded conditions

Figure 64: Slip surfaces during flooded conditions. The green area represents the potential slip surface and the number its corresponding minimum factor of safety. Results are shown for a) Bishop’s simplified; b) Fellenius; c) Morgenstern & Price; and d) Janbu Generalized limit equilibrium methods.
5.3.3 190 m-dam during normal conditions

Figure 65: Slip surfaces during normal conditions. The green area represents the potential slip surface and the number its corresponding minimum factor of safety. Results are shown for a) Bishop’s simplified; b) Fellenius; c) Morgenstern & Price; and d) Janbu Generalized limit equilibrium methods.

5.3.4 190 m-dam during flooded conditions

Figure 66: Slip surfaces during flooded conditions. The green area represents the potential slip surface and the number its corresponding minimum factor of safety. Results are shown for a) Bishop’s simplified; b) Fellenius; c) Morgenstern-Price; and d) Janbu Generalized limit equilibrium methods.
5.3.5 Sensitivity Analysis

In this section the results from the sensitivity analysis are presented. As one can expect the factor of safety is decreasing when shearing resistance angle \( \phi \) is decreasing, however when tested for the tailing materials the effect on the factor of safety is very small. For the fine tailing material the shearing resistance angle has no significant impact on the factor of safety at all. The reason for this is that the critical slip surfaces are only occurring within the starter dam and in some cases also the coarse material.

![Figure 67: The figure shows how the factor of safety varies with a change of the shearing resistance angle in the fine and coarse tailing materials. The sensitivity range represents the range over which the parameter is varied. The lowest value of the shearing resistance angle (\( \phi = 20 \)) is represented as 0 and the highest value (\( \phi = 58 \)) is represented as 1. In this case the Fellenius model has been used.]

5.3.6 Summary

The results from simulations in SLOPE/W are summarized in table 8 and table 9.
Table 8: Results of minimum factor of safety for the 110-m dam scenario

<table>
<thead>
<tr>
<th>Operation condition</th>
<th>Limit Equilibrium Method</th>
<th>Minimum Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>Bishop’s Simplified</td>
<td>1.685</td>
</tr>
<tr>
<td></td>
<td>Fellenius</td>
<td>1.628</td>
</tr>
<tr>
<td></td>
<td>Morgenstern-Price</td>
<td>1.630</td>
</tr>
<tr>
<td></td>
<td>Janbu</td>
<td>1.622</td>
</tr>
<tr>
<td>Flooded</td>
<td>Bishop’s Simplified</td>
<td>1.685</td>
</tr>
<tr>
<td></td>
<td>Fellenius</td>
<td>1.628</td>
</tr>
<tr>
<td></td>
<td>Morgenstern-Price</td>
<td>1.630</td>
</tr>
<tr>
<td></td>
<td>Janbu</td>
<td>1.622</td>
</tr>
</tbody>
</table>

Table 9: Results of minimum factor of safety for the 190-m dam scenario

<table>
<thead>
<tr>
<th>Operation condition</th>
<th>Limit Equilibrium Method</th>
<th>Minimum Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>Bishop’s Simplified</td>
<td>1.552</td>
</tr>
<tr>
<td></td>
<td>Fellenius</td>
<td>1.526</td>
</tr>
<tr>
<td></td>
<td>Morgenstern-Price</td>
<td>1.526</td>
</tr>
<tr>
<td></td>
<td>Janbu</td>
<td>1.522</td>
</tr>
<tr>
<td>Flooded</td>
<td>Bishop’s Simplified</td>
<td>1.552</td>
</tr>
<tr>
<td></td>
<td>Fellenius</td>
<td>1.526</td>
</tr>
<tr>
<td></td>
<td>Morgenstern-Price</td>
<td>1.526</td>
</tr>
<tr>
<td></td>
<td>Janbu</td>
<td>1.522</td>
</tr>
</tbody>
</table>

Results from the simulation of the 110-m dam scenario at normal and flooded operating conditions gives the lowest value of the minimum factor of safety of 1.622 when using Janbu’s limit equilibrium method. Results from the simulation of the 190-m dam scenario at normal and flooded operating conditions gives the lowest value of the minimum factor of safety of 1.522 when using Janbu’s limit equilibrium method.

As the results show, the minimum factor of safety is lower in the 190-m dam scenario compared to the 110-m dam scenario. The results also shows that there is no difference in the calculated minimum factor of safety between the normal and flooded conditions in both scenarios.

According to the standards presented in table 2 in section 3.3.2 Factor of safety the factor of safety has to be larger than 1.25 (Fellenius method) and 1.35 (Bishop’s simplified method) at normal operating conditions for the dam to be classified as stable. At flooded operating conditions the factor of safety has to be larger than 1.15 (Fellenius method) and 1.25 (Bishop’s simplified method) in order to be classified as stable, according to the same
standards. All the obtained results of the minimum factor of safety from simulations of the 110-m and 190-m dam scenarios, at both normal and flooded conditions, are above the required minimum factor of safety in the Chinese standards.
6 Discussion

In this thesis only two raises of the dam have been studied (the 110 and the 190 meter dam), i.e. all other raises in between have not been studied in term of seepage, deformation and slope stability. It is plausible that the dam could be more unstable for other raises of the dam, which should be taken into consideration during the construction of the dam.

The soil parameters have been assumed to be isotropic. A better assumption would have been anisotropic parameters as it would probably give more realistic results. Along with this, the parameters for the slurry has not been verified by previous studies, but approximated according to PhD student Wu [29]. Since the soil parameters have great impact on the results it would be of interest to determine how they vary throughout the dam and not only between different materials.

The seepage results obtained shows no sign of significant risk for the stability of the dam. The seepage is largely contained within the first foundation layer, which it is supposed to be [27]. The maximum seepage velocity of $2.09 \times 10^{-6}$ should not cause erosion as long as the first foundation layer is well designed as a filter. The seepage also does not cause any severe stresses, due to frictional forces between the seeping water and the soil particles, in the area of critical slip surfaces.

In the load and deformation simulations, the non-linear hyperbolic material model has only been used for the coarse and fine tailing materials, while a linear elastic model was assumed for the other materials. As stated in section 3.2.2 Material models and properties, using the non-linear model gives more accurate deformation results, which could also be confirmed from the simulation results. Because of this the deformation of the dam is probably not entirely realistic. However, the deformation is in the order of magnitude that could be expected according to Professor Hu and Dr Wu at the Department of Hydraulic Engineering at Tsinghua University[27, 29].

The results from the simulations in SLOPE/W shows that the stability of the dam in both stages is satisfied according to the standards. The stability decreases slightly in the 190 meters dam compared to the 110 meters dam, which is expected as the stresses have increased. The sensitivity analysis also shows that the dam would be safe even if the shearing resistance angle decreased by 50 percent in the tailing materials.
6.1 Further studies

Apart from the need to do a more detailed version of this study as discussed earlier, there are several areas of interest that this study has not taken into consideration. Therefore it would be advisable to conduct some further studies before construction the dam. Since the Shanxi province is subjected to frequent earthquakes of great magnitude, the slope stability during earthquakes should be investigated. Mine tailings are toxic by nature and it is therefore important to keep contaminants within the dam structure so surrounding environments are not harmed. Therefore contaminant transport due to seepage should be studied as well.
7 Conclusion

The simulations give results that are within reason compared to similar studies (e.g. Experimental Study on Iron Ore Tailings Yuhe Village [15]). Through the simulations conducted in this study both stages of the dam are found to be stable when using the given tailing material, and the dam design has potential to be recommended. However, in this study several simplifications and assumptions have been made in the simulations. This includes values of soil parameters, used material models etc. Also the simulations in SLOPE/W only takes data from SEEP/W into consideration, thus it leaves out the results from the deformation analysis made in SIGMA/W. In order to finally determine the stability of the dam, and make a definite recommendation, more extensive studies of the dam should be carried out. Also the slope stability has only been compared to Chinese standards, and there is reason to be conservative and aim for a higher factor of safety, since the consequences of a dam failure can have large impact on the surrounding environment.
References


Appendix I - Tailing dam failures between 2000-2015

Table 10: Code for each cause of dam failure in the table "Compiled tailing dam failures between 2000-2005" below

<table>
<thead>
<tr>
<th>Failure code</th>
<th>Cause of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Unusual weather (heavy rain, snow melt etc)</td>
</tr>
<tr>
<td>2</td>
<td>Management</td>
</tr>
<tr>
<td>3</td>
<td>Foundation subsidence</td>
</tr>
<tr>
<td>4</td>
<td>Piping/Seepage</td>
</tr>
<tr>
<td>5</td>
<td>Overtopping/Overflow</td>
</tr>
<tr>
<td>6</td>
<td>Structural defect</td>
</tr>
<tr>
<td>7</td>
<td>Slope instability</td>
</tr>
<tr>
<td>8</td>
<td>Erosion</td>
</tr>
<tr>
<td>9</td>
<td>Unknown</td>
</tr>
</tbody>
</table>
The table above was compiled and used to construct figure 6 in the section 2.3 Tailing dam failures. References for the dam failures between 2000-2015: [7, 10, 8, 6, 9, 11, 30, 31, 32, 33]
Appendix II - Soil parameters

The laboratory testing at Tsinghua University was performed during autumn/winter 2015 to determine physical and mechanical characteristics of the available soil for the construction of the Zhaoli ditch tailing dam, and the resulting parameters are presented below.

Table 11: Result of the calculation of Young’s modulus, E, for the fine tailing material.

<table>
<thead>
<tr>
<th>Stress [kPa]</th>
<th>1.625</th>
<th>4.75</th>
<th>9.375</th>
<th>18.75</th>
<th>37.5</th>
<th>75</th>
<th>150</th>
<th>300</th>
<th>600</th>
<th>1200</th>
<th>1800</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_s$ [MPa]</td>
<td>0.13</td>
<td>0.18</td>
<td>0.41</td>
<td>0.76</td>
<td>1.48</td>
<td>2.81</td>
<td>5.42</td>
<td>9.09</td>
<td>15.52</td>
<td>25.5</td>
<td>36.49</td>
</tr>
<tr>
<td>$E$ [MPa]</td>
<td>0.081</td>
<td>0.112</td>
<td>0.255</td>
<td>0.474</td>
<td>0.922</td>
<td>1.751</td>
<td>3.377</td>
<td>5.664</td>
<td>9.670</td>
<td>15.888</td>
<td>22.736</td>
</tr>
</tbody>
</table>

Table 12: Result of the calculation of Young’s modulus, E, for the fine tailing material.

<table>
<thead>
<tr>
<th>Stress [kPa]</th>
<th>1.625</th>
<th>4.75</th>
<th>9.375</th>
<th>18.75</th>
<th>37.5</th>
<th>75</th>
<th>150</th>
<th>300</th>
<th>600</th>
<th>1200</th>
<th>1800</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_s$ [MPa]</td>
<td>0.13</td>
<td>0.18</td>
<td>0.41</td>
<td>0.76</td>
<td>1.48</td>
<td>2.81</td>
<td>5.42</td>
<td>9.09</td>
<td>15.52</td>
<td>25.5</td>
<td>36.49</td>
</tr>
<tr>
<td>$E$ [MPa]</td>
<td>0.092</td>
<td>0.933</td>
<td>1.133</td>
<td>1.267</td>
<td>1.958</td>
<td>3.008</td>
<td>5.067</td>
<td>8.533</td>
<td>14.433</td>
<td>23.692</td>
<td>32.650</td>
</tr>
</tbody>
</table>

Table 13: Soil parameters used to obtain hydraulic water content function.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Starter dam</td>
<td>$10^{-10}$</td>
<td>0.05</td>
<td>Van Genuchten</td>
</tr>
<tr>
<td>Coarse</td>
<td>$1.98 \times 10^{-7}$</td>
<td>0.05</td>
<td>Van Genuchten</td>
</tr>
<tr>
<td>Fine</td>
<td>$1.76 \times 10^{-7}$</td>
<td>0.08</td>
<td>Van Genuchten</td>
</tr>
</tbody>
</table>

Figure 68 and figure 69 are the Mohr diagrams that were constructed from laboratory testings of the soil specimen and shows the total and effective stress for undrained coarse and fine tailings. The resulting material parameters were used as input parameters in SIGMA/W and are summarized in table 14.
Figure 68: Mohr diagrams for coarse tailings. The left graph is the total stress test strength envelope and the right graph is the effective stress test strength envelope [28].

Figure 69: Mohr diagrams for fine tailings. The left graph is the total stress test strength envelope and the right graph is the effective stress test strength envelope [28].

Table 14: Unit weight, cohesion and friction angle for total and effective stress for undrained coarse and fine tailings material

<table>
<thead>
<tr>
<th>Dam Material Layer</th>
<th>Unit Weight, $\gamma_{sat}$ [kN/m$^3$]</th>
<th>Total Cohesion, $c$ [kPa]</th>
<th>Total Friction Angle, $\phi$ [$^\circ$]</th>
<th>Effective Cohesion, $c'$ [kPa]</th>
<th>Effective Friction Angle, $\phi'$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>21.4</td>
<td>0</td>
<td>37</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>Fine</td>
<td>21.5</td>
<td>3</td>
<td>29</td>
<td>0</td>
<td>34</td>
</tr>
</tbody>
</table>
Table 15: Parameters calculated from lab results. These parameters were used as input data for the non-linear hyperbolic material models in the load and deformation simulation.

<table>
<thead>
<tr>
<th>Stress [kPa]</th>
<th>phi [-]</th>
<th>c [Pa]</th>
<th>k</th>
<th>n [-]</th>
<th>Rf [-]</th>
<th>Kb</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>0.7164</td>
<td>0</td>
<td>107.0290</td>
<td>0.8634</td>
<td>0.472771</td>
<td>43.86865</td>
<td>1.1102</td>
</tr>
<tr>
<td>Fine</td>
<td>0.6642</td>
<td>0</td>
<td>104.2404</td>
<td>0.87</td>
<td>0.579457</td>
<td>4.958718</td>
<td>0.74</td>
</tr>
</tbody>
</table>

Appendix III - Settings and data for simulations in GeoStudio™

Seepage boundary conditions

The boundary conditions that were used in the SEEP/W simulations are presented in table 16 and in figure 70, 71, 72, and 73.

Table 16: Boundary conditions used in SEEP/W simulations.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Operation Condition</th>
<th>Left active head, H [m] (red)</th>
<th>Right active head, H [m] (yellow/dark blue)</th>
<th>Flux, Q \left[ \text{m}^2/\text{s} \right] (turquoise)</th>
</tr>
</thead>
<tbody>
<tr>
<td>110 m-dam</td>
<td>Normal -1</td>
<td>102 (yellow)</td>
<td>107.7 (dark blue)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Flooded -1</td>
<td>184 (yellow)</td>
<td>185.2 (dark blue)</td>
<td>0</td>
</tr>
<tr>
<td>190 m-dam</td>
<td>Normal -1</td>
<td>102 (yellow)</td>
<td>107.7 (dark blue)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Flooded -1</td>
<td>184 (yellow)</td>
<td>185.2 (dark blue)</td>
<td>0</td>
</tr>
</tbody>
</table>
Figure 70: Boundary condition for the steady-state seepage simulation of the 110 m-dam during normal conditions.

Figure 71: Boundary condition for the transient seepage simulation of the 110 m-dam during flooded condition.

Figure 72: Boundary condition for the steady-state seepage simulation of the 190 m-dam during normal conditions.
Hydraulic Conductivity Functions

Hydraulic conductivity functions were set for the material layers starter dam, course and fine. The hydraulic functions created by SEEP/W are shown in figure 74, 75 and 76.

Figure 74: Hydraulic conductivity function for the starter dam material layer.
Figure 75: Hydraulic conductivity function for the coarse material layer.

Figure 76: Hydraulic conductivity function for the fine material layer.
Volumetric water content functions

Figure 77: Volumetric water content function for the starter dam material layer.

Figure 78: Volumetric water content function for the coarse material layer.
Figure 79: Volumetric water content function for the fine material layer.

**Stress-Strain Boundary Conditions**

The boundary conditions that were used in the SIGMA/W simulations are presented in table 17 and in figure 80 and 81.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Condition</th>
<th>Left and right x-displ. [m] (green)</th>
<th>Bottom x/y-displ. [m] (turquoise)</th>
</tr>
</thead>
<tbody>
<tr>
<td>110 m-dam</td>
<td>Normal</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>190 m-dam</td>
<td>Normal</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Flooded</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 80: Boundary condition for the stress-strain simulation of the 110 m-dam.
Figure 81: Boundary condition for the stress-strain simulation of the 190 m-dam.

Hyperbolic Young's modulus

Figure 82: Effective Young's modulus for the coarse material when a non-linear hyperbolic material model is used.
Figure 83: Effective Young's modulus for the fine material when a non-linear hyperbolic material model is used.