Stability Analysis of Non-overflow Section of Concrete Gravity Dams

A Longtan Dam case study

Dan Valtersson
Lukas Johansson

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Luleå University of Technology
Department of Civil, Environmental and Natural Resources Engineering
Dan Valtersson
Lukas Johansson

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PREFACE

This master thesis has been carried out as the fulfilling part of our M.Sc. degree in Civil Engineering with specialization towards Mining and Geotechnical Engineering at Luleå University of Technology, Luleå, Sweden. The thesis covers 30 ECTS and was conducted at Hohai University, Nanjing, China during the period April – July 2018. We want to thank Energiforsk and Vattenfall for financing the project and particularly Professor James Yang for creating this opportunity for us.

We want to express our gratitude to Professor Wenhong Dai at the college of Water Conservancy and Hydropower Engineering, Hohai University for supervising this project. We would also like to thank Mengjiao Ding and Tao Hu for helping us on the project and making our stay in China pleasant.

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Nanjing, July 2018
Dan Valtersson
Lukas Johansson
ABSTRACT

The rapid growth of China's economy results in drastic increase of energy demand amongst its 1.4 billion inhabitants. To meet the demands, the government of China has over the years invested in hydropower structures. The climate of China is suitable for hydropower dams where the stations can store and produce electricity from the high-water reservoirs.

Roller-compacted concrete (RCC) dams is a type of concrete gravity dams where the stability of the dam relies on the self-weight of the structure. Compared to other types of gravity dams, RCC dams are more time and cost efficient. Such a dam is Longtan Dam, located in Tian’ê County on the Hongshui river, China. It is one of the tallest of its kind with the height 216.5 m and length 849 m. During this study, Longtan Dam will be used as reference.

The objective of this thesis is to investigate the safety of the non-overflow section of a concrete gravity dam. As consequences of dam failure are devastating, the failure modes and seepage of the dam needs to be analyzed. This is conducted by determining the limit states and seepage of the dam. Three scenarios are considered – normal water condition, critical water condition and an earthquake hitting the normal water scenario.

The results show marginal difference between the water cases when considering the limit states and seepage. In fact, the numerical simulation show that the stress distribution and values are almost identical in the normal and critical case. However, the increase of water level does result in a higher displacement, where the dam tilts 0.5 cm more in the critical case compared to the normal case with 7.0 cm tilt.

The numerical results indicate that the dam is going to fail due to tensile stress at the dam heel, with the stress ranging from -6.0 MPa to -2.0 MPa. The induced compressive stress shows stable results, being below the compressive strength of concrete. The seepage analysis shows no significant risk for the stability of the dam, where high concentrations of seepage velocity concentrates near the dam heel and grout curtains. The low seepage velocity implies no risk of erosion or inducement of high stresses.

As a conclusion, the dam is deemed stable against compressive stress and anti-sliding. However, the risk for tensile failure is high and therefore reinforcements in the dam heel should be applied. The study implies that the dam is safe in some regards, but this thesis is heavily based on several simplifications and assumptions. Further study such as using the analysis type QUAKE/W should be conducted to compare the normal earthquake scenario with the hand calculations.

Keywords: RCC, Longtan Dam, limit states, seepage, FEM
Kinas ekonomi har utvecklats starkt under de senaste åren, vilket har resulterat i en drastisk ökning av energikonsumtion bland landets 1,4 miljarder invånare. För att bemöta efterfrågan har Kinas regering över åren investerat i vattenkraft. Kinas klimat visar sig vara passande för vattenkraft där energi produceras från höga vattenreservoarer.

Roller-compacted Concrete (RCC) dammar är en klassifikation av betongdammar där konstruktionens stabilitet förlitar sig på dammens egen vikt. Jämfört med andra typer av betongdammar så är RCC dammar mer tids- och kostnadseffektiva. Ett exempel på en sådan damm är Longtan dammen som är belägen i Tian’e County på Hongshui floden, Kina. Dammen är en av de högsta i dess klass med höjden 216,5 m och längden 849 m. Longtan dammen kommer under denna studie användas som referens.

Målet med detta examensarbete är att undersöka stabiliteten hos en betongdamm. Konsekvenserna vid ett dammbrott är förödande, därmed måste dammens stabilitet och läckage analyseras. Denna studie tar hänsyn till tre olika fall, nämligen normalt vattenstånd, kritiskt vattenstånd och även det normala vattenståndet samtidigt som en jordbävning tar plats.

Resultaten visar att det är marginell skillnad mellan vattenståndsfallen med hänsyn till brottkriterier och läckage. Faktum är att den numeriska simuleringen visar att spänningsfordelningen och dess värden är nära identiska i det normala och kritiska vattenståendet. Däremot leder en ökning av vattennivåer till ökade deformationer, där dammen lutar 0,5 cm mer i det kritiska tillståndet jämfört med 7,0 cm i det normala.

Resultaten från de numeriska analyserna antyder att dammen kommer att gå till brott på grund av dragspänningar på dammhälen, spänningskretser kring -6,0 MPa till -2,0 MPa. Den inducerade tryckspänningen visar stabila resultat med marginal under tryckhållfastheten för dammens betong. Läckageanalysen visar ingen signifikant risk för dammens stabilitet, högre flöden kretser kring hälen och vid injekteringsskärmarna. Det låga läckaget antyder att ingen risk för erosion eller inducerade spänningar förkommer.

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# SYMBOLS

## Roman uppercase

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<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tr>
<td>$E$</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>$F$</td>
<td>Force</td>
</tr>
<tr>
<td>$FEM$</td>
<td>Finite element method</td>
</tr>
<tr>
<td>$G$</td>
<td>Material shear modulus</td>
</tr>
<tr>
<td>$G_K$</td>
<td>Standard value of permanent effect</td>
</tr>
<tr>
<td>$H$</td>
<td>Section height</td>
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<tr>
<td>$L$</td>
<td>Arm of force</td>
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<td>Wave length</td>
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<td>Standard value of variable effect</td>
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<tr>
<td>$RCC$</td>
<td>Roller-compacted concrete</td>
</tr>
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<td>$S$</td>
<td>Action effect function</td>
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<tr>
<td>$SF$</td>
<td>Factor of safety</td>
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<tr>
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<td>Height of steel penstock</td>
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<tr>
<td>$U$</td>
<td>Uplift pressure</td>
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<tr>
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<tr>
<td>$t$</td>
<td>Time</td>
</tr>
<tr>
<td>$w$</td>
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Greek uppercase

Γ  Shear material strain
∇  Elevation
Σ  Resultant
ψ  Design condition coefficient

Greek lowercase

α  Dynamic distribution coefficient
γ  Unit weight
δ  Limit states coefficient
ε  Strain
η  Reduction factor
θ  Volumetric water content
ν  Poisson’s ratio
ζ  Comprehensive influencing coefficient
ρ  Density
σ  Stress
τ  Shear
φ  Friction angle
1 INTRODUCTION

China’s economy has grown rapidly the last years and with a population of 1.4 billion people, the demand for energy is drastically increasing. As a result, China has become the largest energy consumer and producer in the world, where in 2016 63% of China’s energy was produced through coal (International Energy Agency [IEA], 2017), see Figure 1.1. Nearly three-quarter of the global growth in carbon emission between 2010-2012 occurred in China. With the current growth rate, it is predicted that Chinese emissions could rise by more than 50% in the next 15 years (Liu, 2016).

Large emission of CO$_2$ and pollution has become a big environmental issue for China. The Chinese government has stated a goal that by 2020, 15% of all energy consumed should have renewable energy source background. In 2013, China’s carbon emission was 9.2 Giga ton CO$_2$ which corresponds to 25% of the global carbon emissions that year (Energy Information Administration [EIA], 2015).

![Figure 1.1. China’s installed energy capacity in 2016 (IEA, 2017).](image)

An environmental friendly way to produce energy is through hydropower. Luckily, China’s landscape is suitable for hydropower and is the second largest energy source (Figure 1.1). In 2016 the installed capacity was 330 GW, which equals around 20% of China’s total energy capacity. Chinese hydropower capacity represents more than 25% of the global capacity, and with China’s government 2020 goal, it is aimed to increase the capacity to 480 GW (International Hydropower Association [IHA], 2016). To achieve this goal, more dams and hydropower dams is planned to be constructed.
1.1 Background

Hydropower dams regulate the water flow in rivers and use it to extract energy, this is done by building large constructions. The safety is a critical part in these constructions, as the consequences of a failed dam is catastrophic. The growing energy demand creates a need to build more and higher hydropower dams which results in higher stresses and larger deformations. This creates a need to understand the behavior of the structures, one way to analyze this is to conduct a case study on an already constructed dam. This thesis covers the design procedure of a concrete gravity dam using existing Longtan Dam in China as case study.

1.1.1 Longtan Dam

The Longtan Dam is a roller-compacted concrete (RCC) gravity dam located in Tian’e County on the Hongshui River, China, see Figure 1.2. The purpose of the dam is hydroelectric power production, flood control and navigation. With the dimensions of the height 216.5 m and body length 849 m, it is one of the tallest of its kind. Formal construction began in 2001 and in 2009 all generators became operational, making the dam reach full capacity with the annual generation of 18.7 TWh and capacity 6426 MW (Electric Light & Power [ELP], 2013).

![Figure 1.2. Location of Longtan Dam, China (Google, n.d.).](image-url)
1.2 Objective and Purpose

The aim of this thesis is to investigate the safety of the non-overflow section of a concrete gravity dam. The consequences of dam failure are devastating and therefore the behavior and stability of the dam must be analyzed with time.

This thesis covers a small part of a wider investigation where the focus in this report is the non-overflow section of the dam. The task is to design the layout of the non-overflow section and determine if the design is stable enough against considered failure modes.

The following questions are aimed to be answered in this thesis:

- Is the chosen design layout stable against the limit states of anti-sliding, compressive stress and tensile stress?
- What are the critical stability areas in the dam concerning material failure?
- Is there any risk of failure due to seepage in the dam?
- What influences has natural events such as earthquake and floods on the stability and deformations of the dam?

1.3 Limitations

The design methodology followed in this thesis only covers the non-overflow section in a gravity dam, with the Longtan Dam as reference. In addition, the following limitations are present in this design:

- The shape, slope inclinations and bottom elevation were based on reference data due to time limitations.
- Occurring loads such as ice, wind and sub-atmospheric are not considered in this thesis.
- The numerical analysis was conducted on a 2D cross-section of the non-overflow section. To analyze the effects of widespread stresses, a 3D model should be established and analyzed.
- Analysis in GeoStudio® is conducted using a linear elastic material model, thus Assuming the material never deforms plastically.
- No earthquake analysis is computed in GeoStudio® due to time limitations.
- Numerical computations were conducted on a private laptop, limiting the coarseness of the mesh in the numerical analysis.
2 THEORETICAL BACKGROUND

2.1 Concrete Dams

Dams are generally divided into two groups: embankment dams and concrete dams, where Longtan Dam is classified as a concrete dam. Concrete dams are commonly built using concrete, but older constructions are made of masonry and classify as concrete dams. Compared to embankment dams, concrete dams require less material due to concrete’s strength which results in a slenderer construction. However, concrete dams require to be built upon a hard and resistant foundation compared to embankment dams. Concrete dams are categorized in three types: gravity, arch and buttress dams (Association of State Dam Safety Officials [ASDSO], n.d.), see Figure 2.1.

Gravity dam is the most common concrete dam, where the design utilizes the weight of the dam to transfer the induced reservoir water load downward. The self-weight resists the horizontal water pressure against the dam. Each section of a gravity dam is independently stable, being divided into overflow and non-overflow sections (Tata & Howard, 2016).

Arch dams are designed and built curved in a planar view with the convexity facing the upstream reservoir. This form makes it possible to transfer the induced loads to the adjacent rock formations, compared to gravity dams where the force is relocated downwards. Therefore, the design is only optimal in canyons with solid rock that can resist the pressure and bear the induced bulking. When compared to other concrete dams, arch dams require less materials (ASDSO, n.d.).

Buttress dams have similar design as a gravity dam but has a reduced mass of concrete where vertical and sloping buttresses resist the induced loads and transfer it downwards. The buttresses act like support and are constructed along the dam at certain intervals on the downstream side (Tata & Howard, 2016).
2.1.1 Roller-compacted Concrete Dams

RCC is a blend of concrete that has the same composition as conventional concrete, but with other ratios such as occasional fly ash. The use of RCC for gravity dam construction has rapidly increased as it proves to be more time and cost efficient compared to conventional construction methods. As the mix has lower cement content and use fly ash, the generated heat declines during curing and therefore a higher rate of concrete placement is achieved (Abdo, 2008).

During dam construction, RCC sections are built layer-by-layer in successive horizontal manner, resulting in a slope facing the downstream side. After a layer is built, it immediately supports the continuation of the next layer. The method is like paving, where the material is delivered by conveyors or dump trucks, spread by small dozers and finally compacted by vibratory rollers (Abdo, 2008).

2.2 Layout Design

The purpose when designing a gravity dam section is to select a section design that satisfies the required stability and strength, minimizes the volume to reduce the environmental impact, costs and be constructed and operated easily. To start with, the main load of the dam is considered – which defines the basic section and dimensions according to safety and economic requirements. The basic design is analyzed depending on stress, strain and stability, where it’s modified to a practical design until the most reasonable profile regarding stability and costs is achieved (United States Bureau of Reclamation [USBR], 1976).

2.2.1 Slope Turning Point and Top Elevation

Two essential design conditions of a concrete gravity dam are the slope turning point and top elevation. If using a design with an upstream slope, the slope turning point determines the height of the slope which stabilizes the dam. According to Changsong (2007), the elevation of slope turning point is defined after an assumption based on two criteria;

\[ \frac{1}{3} h < H_s < \frac{2}{3} h \]  \hspace{1cm} (2.1)

\[ \nabla H_s < \nabla S \]  \hspace{1cm} (2.2)

where \( h \) = reservoir height [m]; \( H_s \) = height from bottom elevation to slope turning point [m]; \( \nabla H_s \) = slope turning point elevation [m] and \( \nabla S \) = bottom elevation of the steel penstock [m].
The top elevation is the peak of the dam, which is designed after the water elevation and any additional reservoir height due to accidental situations (Sheng-Hong, 2015). This is done by

\[ \nabla H = \nabla h + \Delta h \]  

(2.3)

where \( \nabla h \) = water elevation [m] and \( \Delta h \) = additional reservoir height [m], which is defined by

\[ \Delta h = h_{1\%} + h_z + h_c \]  

(2.4)

where \( h_{1\%} \) = wave height with a frequency of 1% [m], \( h_z \) = height difference between the central line of the wave and the flood storage level [m] and the height \( h_c \) = freeboard height [m].

2.3 Induced Forces

It’s essential to determine the induced forces acting on the dam structure. To start with, the structure must be stable to withstand certain loads that vary with time and events such as earthquakes. Therefore, the loads are classified according to their time dependence and spatial variation, effect property to the structure, and occurring frequency (US Army Corps of Engineers [USACE], 1995). The loads are classified into:

- Permanent loads - Loads that under the design reference period do not change value with time, or such small change is overlooked. Permanent loads include self-weight, earth pressure, etc.
- Variable loads - Loads that under the design reference period change value with time. Variable loads include water pressure, uplift, and wave pressure.
- Occasional loads - Loads that under the design reference period has a very low probability of occurring, e.g. loads with high impact but short duration. Occasional loads include water pressure under the critical water level and seismic inertia.

2.3.1 Gravitational Force

The gravitational force is the self-weight of the dam and acts as the major resisting force. In a 2D analysis, the force is calculated by the heaviness of the concrete and the area (USACE, 1995)

\[ W = \gamma_c \times H \times w \]  

(2.5)
where $W = \text{cross-section self-weight of the dam [kN]}$; $\gamma = \text{unit weight of concrete [kN/m}^3]\); $H = \text{height of the section [m]}$ and $w = \text{width of force [m]}$.

### 2.3.2 Hydrostatic & Uplift Pressure

Hydrostatic pressure has the largest impact on the dam. The pressure is induced due to water acting against the dam on both the upstream and downstream side (USACE, 1995). The pressures are calculated by

$$P_w = \gamma_w \times h \times w$$

(2.6)

where $P_w = \text{cross-section hydrostatic pressure [kPa]}$ and $\gamma_w = \text{is the unit weight of water [kN/m}^3]\)$. Uplift pressure is a result of water penetrating the foundation, creating a pressure that acts upwards on the dam base. The uplift is evaluated through a pressure diagram from the hydraulic head on the upstream and downstream sides, see Figure 2.2. The uplift can be reduced by installing drains in the bedrock, if no drains is installed the pressure are assumed to be linear between the down and upstream side (Shen-Hong, 2015).

![Figure 2.2. Uplift distribution with and without drain.](image)

Drains create seepage pressure which is calculated using

$$U = a \times \gamma_w \times h$$

(2.7)

where $U = \text{cross-section uplift pressure [kPa]}$ and $a = \text{pressure attenuation coefficient from drains}$.
2.3.3 Wave & Sediment Pressure

Wave pressure is induced due to wind blowing over the reservoir, where the top surface is pulled along the wind’s direction and form waves (Sheng-Hong, 2015). This is defined as

\[ P_{wk} = \frac{1}{4} \times \gamma_w \times L_m \times (h_{1\%} + h_z) \] (2.8)

where \( P_{wk} \) = cross-section wave pressure [kPa] and \( L_m \) = wave length [m].

The incoming stream brings sediment such as silt which deposits on the reservoir bed. The sediment will be carried to the dam face and the elevation of the deposit ascends with time. This creates horizontal pressure defined as active earth pressure (USACE, 1995)

\[ P_{sk} = \frac{1}{2} \times \gamma_s \times \left( \frac{1 - \sin \varphi_s}{1 + \sin \varphi_s} \right) \times h_s^2 \] (2.9)

where \( P_{sk} \) = cross-section sediment pressure [kPa]; \( \gamma_s \) [kN/m³] is the unit weight of the sediment; \( \varphi_s \) [°] the friction angle and \( h_s \) [m] the height of the sediment.

2.3.4 Earthquake Inertia Force

If a dam is situated in a region with moderate risk of earthquake shakes, it is appropriate to take the acceleration into account. The acceleration is either considered horizontally (upstream or/and downstream) or vertically (upward or/and downward), based on the impact of damage (Sheng-Hong, 2015).

The force act like self-weight, where the earthquake inertia applies to the centroid of mass element \( i \). The inertia force in horizontal direction is determined using

\[ F_i = \frac{a_h \times \xi \times W_i \times \alpha_i}{g} \] (2.10)

where \( F_i \) = horizontal earthquake inertia force of the mass element \( i \) [kN]; \( W_i \) = respective self-weight for mass element \( i \) [kN]; \( a_h \) = horizontal earthquake acceleration [m/s²]; \( \xi \) = comprehensive influencing coefficient and \( \alpha_i \) = dynamic distribution coefficient for mass element \( i \). For gravity dams, \( \alpha_i \) is defined by

\[ \alpha_i = 1.4 \times \frac{1 + 4 \left( \frac{H_i}{H} \right)^4}{1 + 4 \sum_{j=1}^{n} \frac{W_j}{W} \times \left( \frac{H_j}{H} \right)^4} \] (2.11)
where $H$ = total height of the dam [m]; $H_i$ = height for mass element $i$ [m]; $W$ = total self-weight of the dam [kN]; $W_j$ = accumulated self-weight for the mass elements [kN] and $H_j$ = accumulated height for the mass elements [m].

When a dam undergoes earthquake motion, the body of the water facing its front tends to remain in place and induce seismic dynamic water pressure on the dam body. Neglecting the vertical motion of water, the horizontal pressure can be simplified and calculated by (Ministry of Water Resources [MWR], 1997)

$$F_0 = 0.65 \times a_h \times \xi \times \rho_w \times h^2 \times \eta_c$$

(2.12)

where $F_0$ = resultant hydrodynamic force [kN]; $\rho_w$ = density of water [kg/m$^3$] and $\eta_c$ = reduction factor determined by the inclination of the upstream dam face.

### 2.4 Partial Safety Factor Method

The general design requirement for partial safety factor method for hydraulic structures according to Shen-Hong (2015) considers three situations during the structure's lifespan:

- Permanent situations: Long-term duration, usually corresponds to the design reference period. The normal operation situation is an example of a permanent situation.
- Temporary situations: Mainly corresponding to a construction-, service- or other temporary situation during the operation of the dam.
- Accidental situations: Lower probability situations, but once occurred will result in serious consequences. Such situations can be critical flood, earthquake or ineffective drainage.

The probabilities and situations are considered by determining the collapse and serviceability limit states. For a gravity concrete dam, such collapse limit states are stability against sliding and compressive strength, while tensile stress is a serviceability limit state.
2.4.1 Limit Equilibrium Method

The limit equilibrium method defines the factor of safety where the resisting function are compared to the action effect function. The method consists of both basic and contingency principles of force combination - dependent on the load situation (Shen-Hong, 2015)

\[
\delta_0 \Psi S(\delta_G G_K, \delta_Q Q_K, a_K, \delta_d) \leq \frac{1}{\delta_{d1}} R \left( f_K, a_K \right)
\]

\[
\delta_0 \Psi S(\delta_G G_K, \delta_Q Q_K, A_K, a_K) \leq \frac{1}{\delta_{d2}} R \left( f_K, a_K \right)
\]

where \( \gamma_0 \) = structural importance coefficient; \( \Psi \) = design condition coefficient; \( S \) = action effect function; \( R \) = structural resistance function; \( \delta_G \) = sub coefficient of permanent effect; \( G_K \) = standard value of permanent effect; \( \delta_Q \) = sub coefficient of variable effect; \( Q_K \) = standard value of variable effect; \( a_K \) = characteristic value of geometry parameter [m]; \( A_K \) = contingency representative value; \( \delta_{d1} \) = basic composite structure coefficient; \( \delta_{d2} \) = accidental combined structure coefficient; \( \delta_m \) = material properties coefficient and \( f_K \) = standard value of material properties.

2.5 Stress and Strain

Subjecting a material to a planar analysis, the material can be induced by normal- and shear stresses. While the normal stress act perpendicular to the plane, the shear stress acts along it. The result of induced stress on a material body is deformations, determined by strain. The relationship between stress and strain can be described by the linear elastic material model and Hooke’s law, which is described in this chapter.

2.5.1 Linear Elastic

Linear elastic analysis indicates the proportional behavior of strain when induced with a stress, see Figure 2.3.

Figure 2.3. Relation between stress and strain in linear elastic conditions (GEO-SLOPE International Ltd, 2013).
Many stress and strain problems can be analyzed in a two-dimensional plane in elastic condition for a first estimate. The theory of elasticity consists of two general types, namely plane stress and plane strain. These analyses can be done with the assumption that the material is homogenous and isotropic (GEO-SLOPE International Ltd, 2013).

### 2.5.2 Hooke’s Law

Hooke’s law is a linear elastic behavior where the stresses are expressed in relation to the strains and is calculated using (GEO-SLOPE International Ltd, 2013)

\[
\sigma = E\varepsilon
\]  

(2.15)

where \(\sigma\) = normal stress [Pa]; \(E\) = Young’s modulus of the material [Pa] and \(\varepsilon\) = material normal strain [%]. Hooke’s law also applies to a two-dimensional shear stress analysis, where the shear stress and strain are calculated with

\[
\tau = G\Gamma
\]  

(2.16)

where \(\tau\) = planar shear stress [Pa]; \(G\) = material shear modulus [Pa] and \(\Gamma\) = shear material strain [%]. The shear modulus is determined by

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

(2.17)

where \(\nu\) = Poisson’s ratio of the material.

### 2.5.3 Plane stress

Plane stress can be visualized as stress action along a thin plate where the z-direction are considered to be zero, therefore \(\sigma_{zz} = \sigma_{yz} = \sigma_{xz} = 0\) (Efunda, n.d.). The plane stress condition leads to the compliance matrix

\[
\begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{xz}
\end{bmatrix} = \frac{1}{E} \begin{bmatrix}
1 & -\nu & 0 \\
-\nu & 1 & 0 \\
0 & 0 & 1 + \nu
\end{bmatrix} \begin{bmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{xz}
\end{bmatrix}
\]  

(2.18)

By inverting the compliance matrix, the stiffness matrix for plane stress is found, which is given by

\[
\begin{bmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{xz}
\end{bmatrix} = \frac{E}{1 - \nu^2} \begin{bmatrix}
1 & \nu & 0 \\
\nu & 1 & 0 \\
0 & 0 & 1 + \nu
\end{bmatrix} \begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{xz}
\end{bmatrix}
\]  

(2.19)
2.5.4 Plane strain

Plane strain can be visualized in an object where one dimension is much larger than the other, for example a long wire with stresses acting perpendicular to its length. The strains in the z-direction are neglected which gives $\varepsilon_{zz} = \varepsilon_{yz} = \varepsilon_{xz} = 0$ (Efunda, n.d.). The strain-stress stiffness for an isotropic material gives the stiffness matrix

$$\begin{bmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{xz}
\end{bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix}
1 - \nu & \nu & 0 \\
\nu & 1 - \nu & 0 \\
0 & 0 & 1 - 2\nu
\end{bmatrix} \begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{xz}
\end{bmatrix}$$

(2.20)

Inverting the stiffness matrix leads to the plane strain’s compliance matrix

$$\begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{xz}
\end{bmatrix} = \frac{1 + \nu}{E} \begin{bmatrix}
1 - \nu & -\nu & 0 \\
-\nu & 1 - \nu & 0 \\
0 & 0 & 1
\end{bmatrix} \begin{bmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{xz}
\end{bmatrix}$$

(2.21)

2.6 Slope Stability

The identification of potential failure surfaces and slip locations are crucial when determining the stability of a concrete gravity dam. The analysis of slope stability can be conducted using several different methods. In this study, the interslice method Morgenstern-Price is computed using the Mohr Coulomb failure criterion.

2.6.1 Mohr Coulomb’s Failure Criterion

Mohr Coulomb is a failure criterion which describes the shear strength of geotechnical materials according to a set of linear equations of the surrounding principle stresses. (GEO-SLOPE, 2013) The relation is expressed as

$$\tau = c + \sigma_n \tan \phi$$

(2.22)

where $\tau$ = shear stress on the failure plane [Pa]; $c$ = apparent cohesion [Pa]; $\sigma_n$ = normal stress on the failure plane [Pa] and $\phi$ = internal friction angle [°]. The relation is visually represented in Figure 2.4, where the half circles represent the Mohr circles.
2.6.2 Morgenstern-Price Method

The Morgenstern-Price method is a limit equilibrium method which is used in computational analysis of slope stability. The basic theory behind limit equilibrium is that forces, moments or stresses that resists the unstable motion are compared to those who causes it. This method is applied by considering a specific slip surface where the failure surface is divided into vertical slices, see Figure 2.5.

In Morgenstern-Price method, the interslice forces and shear forces are assumed to act between the slices by considering the half-sine function. The half-sine function concentrates the shear forces of respective interslice to the middle of the sliding mass and decreases area around the edges, crest and toe (D. G. Fredlund and J. Krahn, 1977).
2.7 Darcy’s law & Seepage

Darcy’s law is a water flow formula for both saturated and unsaturated soil and is defined by (GEO-SLOPE International Ltd, 2012a)

\[ q = ki \]  

(2.23)

where \( q \) = the specific discharge [m³/s]; \( k \) = the hydraulic conductivity [m/s] and \( i \) = the gradient of total hydraulic head.

The specific discharge in Darcy’s law are often defined as Darcian velocity, where the actual average velocity at which water moves through the soil is linear. In GeoStudio®, SEEP/W computes and presents only the Darcian velocity.

The governing differential equation used in SEEP/W finite element formulation is defined as (GEO-SLOPE International Ltd, 2012a)

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

(2.24)

where \( H \) = the total head [kPa]; \( k_x \) = hydraulic conductivity in the X-direction [m/s]; \( k_y \) = hydraulic conductivity in the Y-direction [m/s]; \( Q \) = applied boundary flux [m³/s/m²]; \( \vartheta \) = volumetric water content [%] and \( t \) = time [d].

The formula states that the difference in water flow (flux) through an element volume at some point in time is equal to the difference in water storage in the soil systems. Under steady-state conditions, the flux in and out through an elemental volume remains constant, which applies to Steady-State analyses in SEEP/W

\[ \frac{\partial}{\partial x} \left( k_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial H}{\partial y} \right) + Q = 0 \]  

(2.25)

2.8 Finite Element Method

The application of the laws of physics for space- and time-dependent issues are often expressed in terms of partial differential equations. In practices like civil engineering, complicated geometry, boundary conditions and material properties can prove analytical methods to be difficult, if not impossible. Thus, the application of numerical methods can approximate the partial differential equations in a way making it possible for the engineer to validate the results. This procedure is called discretization which approximates the partial differential equations with numerical model equations. The finite element method (FEM) is one such method that computes such approximations (COMSOL, n.d.).
2.8.1 Basic Principle

The basic principle behind the finite element method is the division of a structural body into elements, called finite elements. The division makes it possible to instead of solving a continuous problem for a whole structure, the finite elements are solved independently and thus a solution of the entire structure can be obtained (COMSOL, n.d.). The discretization of a structure is divided into elements and nodes, creating a finite element model (Figure 2.6).

Figure 2.6. Structure discretized to a finite element model, showing different element types (FEM Expert, n.d.).

The choice of the most suitable element type depends on the type of structure that is analyzed. Triangular elements and quadrilateral elements, which can be seen in Figure 2.6, are the most common in civil engineering and are used for structural issues such as plate bending and planar stress and strain (Malm, 2016).
3 METHODOLOGY

3.1 Layout

In this thesis, the layout for two cases are considered, namely design and critical water level. The design water level is the water reservoir based on normal conditions, where the critical water level considers accidental situations. Using some of Longtan Dam’s parameters and dimensions as reference (Table 3.1), the slope turning point and top elevation for both cases are calculated (Figure 3.1).

Table 3.1. Input parameters for layout design (Dai, 2018).

<table>
<thead>
<tr>
<th>∇b [m. a.s.l.]</th>
<th>n</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>195</td>
<td>0.15</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 3.1 shows \( \nabla b \) = bottom elevation [m]; \( m \) = inclination of the downstream slope and \( n \) = inclination of the upstream slope.

Using the top elevation, the total width is determined by applying 8 ~ 10 % of the top elevation (Dai, 2018), which gives the width 17 m. For overview layout of the dam, see APPENDIX III – OVERVIEW LAYOUT.
3.2 Acting Forces

The forces covered in chapter 2.3 are calculated in this thesis and are illustrated in Figure 3.2.

The loads in Figure 3.2 are the following:

1. Seismic dynamic water pressure
2. Horizontal sediment pressure
3. Vertical sediment pressure
4. Wave pressure
5. Horizontal hydrostatic pressure, upstream
6. Vertical hydrostatic pressure, upstream
7. Gravitational force, self-weight
8. Horizontal earthquake force
9. Uplift pressure
10. Vertical hydrostatic pressure, downstream
11. Horizontal hydrostatic pressure, downstream.

In reality, the dam consists of void spaces containing turbines, utility tunnels and storage. However, the additional weight is neglected as it is assumed that the weight of voids filled with concrete cancel out the weight of machineries. Therefore, the dam is considered to consist of 100% concrete for simplicity (Federal Energy Regulatory Commission, 2016).
The loads are determined for both the normal and critical water level. However, the earthquake forces are not calculated for the critical water level as it’s assumed that the possibility of a critical flood and earthquake occurring at the same time is unlikely (Dai, 2018). The forces are summarized for both conditions in horizontal and vertical directions.

When determining the moments and moment arm, the bottom of the dam is used as reference point. Therefore, the moments acting in counter clockwise direction is considered positive and clockwise negative. The moments are divided into restoring- and overturning moments.

### 3.3 Limit States

The limit states considered in this thesis is the anti-sliding stability of concrete interface between dam body and bedrock, compressive stress at the dam toe and tensile stress of the dam heel. Acting forces and moments for respective limit state can be seen in Figure 3.3.

![Figure 3.3. Limit states of the non-overflow section.](image)

The stability for respective limit state is calculated for both basic and contingency load combinations. For the basic combination the normal water level is analyzed and for contingency the critical water level and earthquake situation are considered. The unreduced compressive strength of the concrete is 25.4 MPa (Dai, 2018).
3.4 Modeling in GeoStudio

The numerical analyses in this thesis are conducted using the software GeoStudio®. By drawing the models and simulating the results, conclusions can be made based on the software’s computation. GeoStudio® is a modeling software which is mainly used by geo-engineers and earth scientists.

In this thesis the analysis types of SIGMA/W, SEEP/W and SLOPE/W are used to analyze the non-overflow section of the concrete gravity dam and conclude the objective of this study.

3.4.1 Model setup

The dimensions of the non-overflow section of the Longtan Dam are directly imported into GeoStudio® using AutoCAD®. Regions and respective material layers are created based on given layout from the project background (Dai, 2018). See APPENDIX I – FEM LAYOUT for imported CAD layout. Mesh properties are set to triangles only with 5 m approximate global element size. See Figure 3.4 for model layout.

![Figure 3.4. Model layout in GeoStudio.](image)
Two scenarios were simulated in GeoStudio®. The scenarios are dependent on the upstream and downstream water levels, where the first scenario reflects a normal case and the second scenario replicates flooded water levels – referred as critical case. See Table 3.2 for applied water levels.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Upstream [m]</th>
<th>Downstream [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>182.1</td>
<td>30.5</td>
</tr>
<tr>
<td>Critical</td>
<td>185.8</td>
<td>65.2</td>
</tr>
</tbody>
</table>

### 3.4.2 SIGMA/W

Analysis in SIGMA/W simulates the resulting stresses and deformations that occurs in the dam dependent on the water level scenario. The analysis is set up with a Load/Deformation case and an in-situ analysis as its source. This is to establish in-situ conditions on the dam body and bases, which allows the Load/Deformation analysis to compute the end condition based on the in-situ case (GEO-SLOPE International Ltd, 2013). The time duration applied to the Load/Deformation analysis is only one day with the time increment one day.

The material category used is Total Stress Parameters with the Linear Elastic (Total) model explained in Chapter 2.5. Applied material properties are shown in Table 3.3.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E$ [GPa]</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$\nu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>20</td>
<td>24</td>
<td>0.163</td>
</tr>
<tr>
<td>R2</td>
<td>20</td>
<td>24</td>
<td>0.163</td>
</tr>
<tr>
<td>Curtain</td>
<td>20</td>
<td>26.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Base1</td>
<td>16</td>
<td>25</td>
<td>0.3</td>
</tr>
<tr>
<td>Base2</td>
<td>16</td>
<td>25</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Boundary conditions are defined with a zero value X-displacement along the edge boundary walls and a zero X/Y-displacement at the boundary base. The water levels from Table 3.2 are defined as hydrostatic pressures. The boundary conditions of the induced uplift pressures can be found in APPENDIX II – BOUNDARY CONDITIONS.


3.4.3 SEEP/W

Seepage analysis in SEEP/W are computed to determine the pore water pressure, hydraulic head and water flux distribution and penetration through the dam body and base layers. The analysis of seepage velocity and direction is crucial for a stable and safe dam (GEO-SLOPE International Ltd, 2012a). SEEP/W is formulated on the theory of Darcy’s law of water flow through both saturated and unsaturated soil, see Chapter 2.7.

The seepage through the dam is analyzed using the Steady-State Seepage analysis. The material model used is Saturated/Unsaturated where the water volume content is undefined, making the hydraulic conductivity and its anisotropy the only input data. See Table 3.4 for material seepage properties.

<table>
<thead>
<tr>
<th>Material</th>
<th>Hyd. Conductivity, $k$ [-]</th>
<th>Anisotropy, $k_y'/k_x'$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>2.5E-9</td>
<td>1</td>
</tr>
<tr>
<td>R2</td>
<td>2.5E-11</td>
<td>1</td>
</tr>
<tr>
<td>Curtain</td>
<td>2.5E-9</td>
<td>1</td>
</tr>
<tr>
<td>Base1</td>
<td>2.5E-7</td>
<td>1</td>
</tr>
<tr>
<td>Base2</td>
<td>-</td>
<td>1</td>
</tr>
</tbody>
</table>

The upstream and downstream water pressures are defined as Water Total Head boundary conditions with the constant normal/critical water levels for each boundary. On the downstream slope a Water Rate boundary with the constant rate of 0 m$^3$/s is drawn to simulate the impermeable concrete.

3.4.4 SLOPE/W

Stability analysis of earth structures in SLOPE/W is simulated to determine a key issue in geotechnical engineering, namely if the structure will remain stable or not. SLOPE/W applies the theory of limit equilibrium formulations explained in Chapter 2.4.1, making it possible to deal with complex stratigraphy, various shear strength models and many kinds of slip surface shapes (GEO-SLOPE International Ltd, 2012b). The factor of safety method applied in this study is Morgenstern-Price which is covered in Chapter 2.6.2.

The material models used differs on the material type. To start with, the drainage curtain itself is assumed to have no impact on the slope stability, thus it is not defined in the model. The concrete of the dam is defined as a high strength material with no cohesion or friction angle. The upper base layer has cohesion and friction angle defined but the lower has none due to it being bedrock, see Table 3.5.
Table 3.5. Material input data for SLOPE/W analysis (Dai, 2018).

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$c$ [MPa]</th>
<th>$\varphi$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>24</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Base1</td>
<td>23</td>
<td>1.09</td>
<td>54.1</td>
</tr>
<tr>
<td>Base2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Induced water pressure is defined by a piezometric line spanning from the upstream water level along the dam slope to the downstream water level. In addition, sediment, uplift and wave pressures are simulated by point loads in the model.
4 RESULTS

4.1 Dam Dimensions

According to the procedure in Chapter 2.2, the slope turning point is assumed to be 90 m based on the criteria from Chapter 2.2.1. The calculated additional water levels and reservoir elevation for both design and critical case is presented in Table 4.1.

Table 4.1. Top water elevation for design and critical water levels.

<table>
<thead>
<tr>
<th></th>
<th>Design [m]</th>
<th>Critical [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>∆h</td>
<td>4.35</td>
<td>1.82</td>
</tr>
<tr>
<td>∇Top</td>
<td>381.05</td>
<td>380.80</td>
</tr>
</tbody>
</table>

The table shows that the top elevation for the dam is the design elevation, which has a higher elevation than the critical case. The top elevation is therefore 381 m a.s.l. and the width or crest 17 m, see Figure 4.1.

Figure 4.1. Design for non-overflow section of the dam.
4.2 Stability Analysis

The induced forces and moments acting on the dam have been calculated according to Chapter 2.3. The results are in turn applied in the procedure of calculating stability according to limit states presented in Chapter 2.4.

4.2.1 Forces and Moments

The acting forces and moments are summarized in tables below. See Table 4.2 for horizontal and vertical loads and Table 4.3 for positive and negative moments acting on the dam.

Table 4.2. Summarized horizontal and vertical design loads.

<table>
<thead>
<tr>
<th></th>
<th>$\Sigma F_H$ [MN]</th>
<th>$\Sigma F_V$ [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>200.6</td>
<td>290.8</td>
</tr>
<tr>
<td>Critical</td>
<td>174.5</td>
<td>287.5</td>
</tr>
</tbody>
</table>

Table 4.3. Summarized positive and negative moments.

<table>
<thead>
<tr>
<th></th>
<th>$\Sigma M^+$ [MNm]</th>
<th>$\Sigma M^-$ [MNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>8348.8</td>
<td>-13789.7</td>
</tr>
<tr>
<td>Critical</td>
<td>9224.1</td>
<td>-13922.3</td>
</tr>
</tbody>
</table>

The arm of force in both X- and Y-direction from dam center (Figure 3.3) are calculated and presented in Table 4.4.

Table 4.4. Center of mass for the arm of force in X- and Y-direction.

<table>
<thead>
<tr>
<th></th>
<th>X [m]</th>
<th>Y [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>23.1</td>
<td>56.6</td>
</tr>
<tr>
<td>Critical</td>
<td>18.7</td>
<td>54.3</td>
</tr>
</tbody>
</table>
4.2.2 Limit States

The factor of safety against sliding is presented in Table 4.5.

<table>
<thead>
<tr>
<th>Calculation Condition</th>
<th>$\delta_0 \psi S$ [MN]</th>
<th>$1/ \delta_0 R$ [MN]</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>202.6</td>
<td>256.0</td>
<td>1.26</td>
</tr>
<tr>
<td>Critical</td>
<td>163.1</td>
<td>253.7</td>
<td>1.56</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>187.6</td>
<td>256.0</td>
<td>1.37</td>
</tr>
</tbody>
</table>

The induced compressive stress compared to the concrete’s compressive strength is listed in Table 4.6.

<table>
<thead>
<tr>
<th>Calculation Condition</th>
<th>$\delta_0 \psi S$ [MPa]</th>
<th>$1/ \delta_0 R$ [MPa]</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>5.2</td>
<td>14.1</td>
<td>2.73</td>
</tr>
<tr>
<td>Critical</td>
<td>4.5</td>
<td>14.1</td>
<td>3.13</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>4.8</td>
<td>14.1</td>
<td>2.93</td>
</tr>
</tbody>
</table>

Table 4.7 shows the tensile stress induced on the dam heel, where values < 0 indicates tensile failure.

<table>
<thead>
<tr>
<th>Calculation Condition</th>
<th>$\delta_0 \psi S$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>0.9</td>
</tr>
<tr>
<td>Critical</td>
<td>0.6</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>0.5</td>
</tr>
</tbody>
</table>

4.3 Numerical Analysis

The computations and results presented in this chapter are obtained using the setup according to Chapter 3.4. Results of interest from the SIGMA/W analysis are the max. Shear stress, max. & min. total stress and XY-displacements, while from SEEP/W the interesting results are the water total head, pore-water pressure and the water flux. As for SLOPE/W the safety factor against sliding is of interest. Numerical computation for both normal and critical cases has been conducted and the differences are commented.
4.3.1 SIGMA/W

The maximum shear stress acting on the dam for normal water level show high concentrations of stress at the dam heel and dam toe, see Figure 4.2. At the range of 2 m inwards from the dam heel the maximum shear stress spans from 1.4 MPa to 3.2 MPa.

Compared to the normal water level, the critical maximum shear stress shows the same pattern where high concentrations occur in the dam heel and toe. In addition, similar stresses are induced, with only a slight difference of 0.2 MPa near the boundaries, see Figure 4.3 for horizontal boundary plots.
The max. Total stresses induced on the dam show high concentration of compressive stress at the dam toe, see Figure 4.4. According to the contour the amount of stress rapidly increases at the dam toe’s boundary. The stress spans from 4.0 MPa to 6.5 MPa.

The critical analysis of the max. Total stress shows the same behavior and concentration of stresses. However, the amount of compressive stress is declining when applying the critical water levels. The main difference in stress is due to the increase of the downstream level, which is double the normal case. See Figure 4.5 for stress plot.
The min. total stress result show tensile stresses acting on the dam heel, see Figure 4.6. The range from the boundary and inwards span from -6 MPa to -2 MPa, indicating that there are tensile stresses present which could lead to failure due to the tensile strength of concrete being 0 MPa according to Chinese standards.

Computing the critical water case, the difference of tensile stress is noted to be approximately 0.3 MPa higher compared to the normal case. This indicates higher amount of tensile failure in the dam heel in the critical case compared to normal case, but only marginally so according to Figure 4.7.

![Figure 4.6. Min. Total Stress, normal water case.](image)

![Figure 4.7. Min. Total Stress in relation to boundary distance, heel.](image)
The displacements in XY-direction for normal water level show displacements up to 7 cm at the top, aiming to tilt the dam towards the downstream side, see Figure 4.8. The displacement is a combination of both X- and Y-displacements, where the X-displacement is around 6.5 cm at the top and Y-displacement 1.2 cm on the dam upstream face. For the deformed mesh, see Figure 4.9.

When comparing the normal case to the critical case, it is observed that the XY-displacements in the critical case has a 0.5 cm higher displacement. This is due to the X-displacement being 1 cm higher, which is a result of the increase of elevation in the critical case.
4.3.2 SEEP/W

The water total head for normal water level show the head pressure interval after the defined upstream and downstream levels, namely 182.1 m and 30.5 m, see Figure 4.10. The equipotential lines of the flow net are adjusted after the structure of the dam, where the influence of the drainage curtain can be observed in the figure.

![Figure 4.10. Water Total Head, normal water case.](image1)

When comparing to the critical case, similar flow net and equipotential lines in the base layer can be observed. However, the increase in both upstream and downstream levels result in higher water total head pressure in the dam body. Specifically, on the downstream side where the water level is doubled in the critical scenario, see Figure 4.11.

![Figure 4.11. Water Total Head, critical water case.](image2)
The pore-water pressure acting on the dam show an increase in pore-water pressure in relation to depth. In the dam structure the water pressure increases based on the difference in water levels of the upstream and downstream sides, see Figure 4.12.

![Figure 4.12. Pore-water Pressure, normal water case.](image)

For the critical case, the pore-water pressure declines with depth as expected. But the pore-water pressure also follows the downstream slope due to the higher water level, see Figure 4.13.

![Figure 4.13. Pore-water Pressure, critical water level.](image)
Stability Analysis of Non-overflow Section of Concrete Gravity Dams

The water flow for the normal water condition shows a flow chart with high concentrations of flux at the dam heel and the grout curtains, see Figure 4.14. Due to the impermeable conditions of concrete and rock, no water flux is present in the dam and the lower base layer.

![Figure 4.14. Water Flux, normal water case.](image)

Like the water flow in the normal case, the flux concentrates at the dam heel and around the grout curtains. The main difference between the cases is the amount of water flux, where the flow is higher in the normal case. This is visualized by plotting the water rate through the base layer in relation to the dam curtains, see Figure 4.15 for subdomains and Figure 4.16 for plot.

![Figure 4.15. Subdomain elements chosen for water rate plot.](image)
The graph shows that at the point of grout curtains, the water rate is higher in the normal scenario than the critical scenario. However, due to the higher downstream water level the water rate at the dam toe is higher in the critical case than in the normal case.
4.3.3 SLOPE/W

The numerical computation of slope stability of the dam shows 55 potential slip surfaces where all show stable results. In Figure 4.17 are the most critical slip surfaces visualized. The results show that the lowest safety factor is 2.26. It is observed that all potential slip surfaces are very shallow in the base layer.

Applying the critical water case, a similar result is observed. Specifically, the safety factor of the most critical surface increases from 2.26 to 2.36. The same potential slip surfaces apply to the critical case as the normal, although slightly higher safety factor.

4.4 Comparing Hand- and Numerical Results

In this chapter, the hand calculated limit states are compared with its respective numerical analysis. The comparisons apply to the normal and critical scenarios, while the normal + earthquake scenario are not computed in GeoStudio©.

The hand calculated anti-sliding is compared to the SLOPE/W analysis using Morgenstern-Price method according to Chapter 3.4.4. The safety factor retrieved from the numerical analysis is the worst-case slip surface. See Table 4.8 for comparison.
Table 4.8. Factor of safety against sliding, comparing hand- and numerical results.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>202.6</td>
<td>256.0</td>
<td>1.26</td>
<td>2.26</td>
</tr>
<tr>
<td>Critical</td>
<td>163.1</td>
<td>253.7</td>
<td>1.56</td>
<td>2.36</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>187.6</td>
<td>256.0</td>
<td>1.37</td>
<td>-</td>
</tr>
</tbody>
</table>

The results show stable safety factor for both methods. However, the hand calculations imply a lower factor of safety against sliding when compared to the numerical results.

When comparing the factor of safety against compressive failure, numerical data from the boundary and 2 meters inwards from the dam toe was used. This results in a stress interval and thus a factor of safety interval, see Table 4.9.

Table 4.9. Factor of safety against compressive failure, comparing hand- and numerical results.

<table>
<thead>
<tr>
<th>Calculation Condition</th>
<th>Stress, hand [MPa]</th>
<th>SF</th>
<th>Stress, numerical [MPa]</th>
<th>SF, numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>5.2</td>
<td>2.73</td>
<td>5.8 – 6.6</td>
<td>2.13 – 2.43</td>
</tr>
<tr>
<td>Critical</td>
<td>4.5</td>
<td>3.13</td>
<td>5.4 – 5.9</td>
<td>2.39 – 2.66</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>4.8</td>
<td>2.93</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The table show that the factor of safety in the numerical analysis is lower than the hand calculations, but show similar patterns comparing the normal and the critical case. The factor of safety are above 2.0 for both methods.

Tensile stress data from the boundary and 2 meters inwards are collected for the comparison with the hand calculations, see Table 4.10. The tensile strength of concrete in this analysis is set to 0 MPa.

Table 4.10. Tensile stresses at dam heel, comparing hand- and numerical results.

<table>
<thead>
<tr>
<th>Calculation Condition</th>
<th>Stress, hand [MPa]</th>
<th>Stress, numerical [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>0.9</td>
<td>(-5.8) – (-4.2)</td>
</tr>
<tr>
<td>Critical</td>
<td>0.6</td>
<td>(-6.2) – (-4.5)</td>
</tr>
<tr>
<td>Normal + Earthquake</td>
<td>0.5</td>
<td>-</td>
</tr>
</tbody>
</table>

The comparison shows that the hand calculations indicate that the concrete is stable against tensile failure while the numerical results show the complete opposite with a huge marginal.
5 ANALYSIS AND DISCUSSION

5.1 Model Validation

One limitation mentioned in Chapter 1.3 is the hardware used for the numerical modeling, which limited the number of elements and nodal points which determines the accuracy of the computation. Arguably more elements give better results, but it affects the computation speed and due to the time limitation, the amount of computation and mesh size had to be limited. More accurate numerical computation could be a potential subject for further study in this matter.

However, it was noted that when computing a finer mesh, the result itself did marginally differ from a more course mesh. Most notably was the stress at the dam heel and toe boundaries which increased rapidly when using a finer mesh. This is due to stress singularity where the defined uplift pressure acts like a point load, causing the stress to increase rapidly as the element area the force is distributed lowers. Therefore, it is argued that the true values were located a few meters inwards of the dam and not at the boundary. This was later confirmed when returning to Sweden where computations on finer meshes show the same stresses as the course mesh in China a few meter inwards from the dam body’s boundary.

5.2 Limit States

The results show no big difference between the normal and critical water level cases. The stress distribution and values are almost identical in both cases, where a small increase in stresses and displacement can be observed in the critical case. The increase of water levels does result in a higher displacement, where the dam tilts 0.5 cm more than the normal case of 7.0 cm.

Points of interest lies mainly on occurring tensile stress at the dam heel and compressive stress at the dam toe. Using the tensile strength of 0 MPa for concrete, the numerical results indicates that the dam is going to fail at the dam heel. The tensile stresses range from -6.0 MPa to -2.0 MPa, but the highest value is present on the boundary which rapidly decreases further inwards. This is due to stress singularities in FEM, where the value increases the finer the mesh is due to the uplift pressure being defined as a point load.

When compared to hand calculations, no tensile stresses are shown at the heel which implies that the dam is safe. But the hand calculations are very simplified compared to GeoStudio® as the computation takes a more precise approach with material parameters, stress distribution and dam geometry into account.
Thus, GeoStudio® show more reliable results than the hand calculations. But in a real scenario the tensile strength of concrete is arguably higher than 0 MPa. But as the area is one of the cores in dam stability, one should reinforce it to hinder any tensile failure.

When comparing the compressive stresses in GeoStudio® and hand calculations, both show a stable result with stress below the compressive strength of 14.1 MPa. In addition, the stress values from FEM and hand calculation show similar results according to Table 4.9. Therefore, it can be concluded that the dam is safe from compressive failure as two different approaches show stable results and similar stresses.

The anti-sliding results show that the dam is safe against sliding. However, it is noted that in the hand calculations the safety factor for the normal case is close to 1.0. Arguably the safety factor should be over the limit with reasonable marginal to ensure the stability of the dam. The numerical results show higher safety factors when compared to hand calculations. As a result, it is concluded that the dam is safe against sliding.

5.3 Seepage

Judging from the seepage results there are no significant risk for the stability of the dam due to seepage. The results show high concentrations of seepage velocity near the heel and the grout curtains, while underneath the dam lower velocity is noted. This implies that any risk of erosion and high stresses are induced at the heel and grout curtains, but with the maximum velocity of 3.4E-7 m³/s/m² it is deemed unlikely. This applies to both the normal and critical case, where notably the critical case is more stable due to the phreatic surface being higher.

5.4 Future Study

In this analysis a linear elastic material model was used for the concrete. This means in the computation the strain will keep increasing constantly with the stress even though it may have surpassed the failure limit. Since our computation show tensile stresses present in the dam heel and concrete being a brittle material, it is of interest to do a plastic analysis to analyze how the stability of the dam behaves when exposed to tensile failure. But in a linear elastic analysis this is not taken to account, where the concrete is assumed to keep deforming with increased stress. For example, cracks will most likely occur in the dam judging from the results, which causes water to seep into the cracks and create uplifting pore pressure. The impact of such events on the stability of the dam is worth investigating.
In addition, the numerical simulation only covered the normal and critical water scenarios. Further study using the analysis type QUAKE/W should be conducted to compare the normal earthquake scenario with the hand calculations. Also, the SLOPE/W analysis should be complemented as the material properties and model are based on assumptions by the modelers and supervisor. Due to time limitations only one SLOPE/W analysis was computed, more methods should be investigated where the results are compared to reach a conclusion.

Lastly, this report only covers the non-overflow section of the dam. A stability study of the overflow section should be conducted and with the remaining sections a detailed 3D analysis of the whole dam should be investigated.
6 CONCLUSIONS

The results covered in this thesis conclude that the non-overflow section of the concrete gravity dam is safe against compressive stress and anti-sliding. However, the risk for tensile failure is high and therefore reinforcements in the dam heel should be applied. Even though the simulations and the procedure conducted in this study implies that the dam is stable in some regards, the study is based on several simplifications and assumptions. While the dam design has the potential to be recommended for practical use, further studies discussed in Chapter 5.4 should be considered. In addition, the design methodology and stability limits are based on Chinese standards, which should be converted or considered when applying the design internationally.
7 REFERENCES


Stability Analysis of Non-overflow Section of Concrete Gravity Dams


Figure 8.1. Imported CAD layout for FEM analysis.
## APPENDIX II – BOUNDARY CONDITIONS

### Uplift Pressure

Table 9.1. Uplift pressure forces for respective Section (Figure 9.1). Values in kN.

<table>
<thead>
<tr>
<th>Case</th>
<th>Normal</th>
<th>Critical</th>
<th>Normal</th>
<th>Critical</th>
<th>Normal</th>
<th>Critical</th>
</tr>
</thead>
<tbody>
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<td>$P_1$</td>
<td>8037</td>
<td>8199</td>
<td>2529</td>
<td>2629</td>
<td>520</td>
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<td>$P_2$</td>
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<td>1540</td>
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</tr>
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<td>1290</td>
<td>1343</td>
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</tr>
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<td>$P_{2,3}$</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$P_4$</td>
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<td>4156</td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>
Figure 9.2. Visualization of uplift pressures for respective section.
APPENDIX III – OVERVIEW LAYOUT

Figure 10.3. Elevation arrangement of the dam.
HORIZONTAL ARRANGEMENT 1:2000

Figure 10.4. Horizontal arrangement of the dam.