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A NUMERICAL STUDY ON AN EARTH DAM SAFETY BY USING FINITE ELEMENT METHOD

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ABSTRACT

A NUMERICAL STUDY ON AN EARTH DAM

SAFETY BY

USING FINITE ELEMENT METHOD

M.Sc. THESIS

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Re-assessment of earth dam's safety from geotechnical point of view is very important issue, because if failure occurs in the dam the losses will be enormous. In the last century the world faced many disasters because of failure of dams. Yet, According to the statistics, in the last two decades there was annually one or more dam failure occurred in worldwide because of faults in design or construction or maintenance. Dam failure is a disaster and makes a huge hazards, it causes damages in everything in the receptor area in downstream. Usually there are huge losses in people, properties and environment. Therefore a careful attention and re-assessment should be focused on earth dams to prevent possibility of failure,because earth dam effected widely by many internal and external factors and that the earth dams are becoming widespread because of its competitive advantages over other types of dam where it is: low cost, easy to construct, it can be constructing in difficult site condition, etc.

Failure of earth dams may occur for many reasons: creating of piping phenomenon and erosion may grow inside or beneath the dam due to seepage of water, also earthquake may causes dam failure, side slopes of the dam may slide during or at: the end of the construction, full reservoir level or in case of rapid drawdown of reservoir level, or dam failure may be due to other factors.

In this thesis geotechnical re-assessment conducted for Shauger earth dam as a case study. Where, slope stability examined for many conditions: at the end of construction, steady state seepage, rapid drawdown, instantaneous and slow drawdown of the reservoir level. While seepage investigated for both static steady state and transient, also piping investigated. Moreover dam examined for earthquake case by using dynamic analysis to check the effect of this case on the dam stability, deformation and seepage. The dam simulated in numerically model and analysed by using finite element method. Geostudio 2012, software used to perform the analysis.

Results show that Shauger earth dam is not in the safe case according to the Ordinary and Janbo methods for U/S slope stability analysis, for the case of rapid drawdown and is not in the safe case according to QUAKE/W Stress method for the D/S slope stability, for the case of full reservoir (maximum water level) with earthquake. Therefore, there is a risk from geotechnical point of view.

Key Words: Earth dams, Embankments, Dam assessment, Dam's safety, Geostudio, Slope stability.

ÖZET

SONLU ELEMANLAR KULLANILARAK BİR BARAJIN GÜVENLİĞİNİN YENİDEN İNCELENMESİ

YÜKSEK LİSANS TEZİ

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Geoteknik disiplini açısındanmevcut barajların "yeniden değerlendirilmesi", herhengi bir göçme durumunda mal ve can kayıplarının büyüklüğü göz önünde bulundurularak, ne denli önem arz ettiği tartışılmazdır. Geçen yüzyılda baraj göçmeleri nedeniyle büyükfelaketler ile karşı karşıya kalınmıştır. Oysa, istatistiklere göre, sadece son yirmi yıla bakıldığında, hatalı tasarım ve inşa nedeniyle yıllık bazda bir veya birkaç barajın yıkılmış olduğu görülmektedir. Baraj göçmeleri menba ve daha aşagısında yer alan her türlü yapı ve değerinyıkımına yol açan felaketler getirmektedir. Bir çok iç ve dış etmene bağlı olarak göçmedurumuna ulaşan barajların, öncesinde çok hassas çalışmalar yürütülerek, göçme riskinin zaman zaman yeniden değerlendirilmesi önem arz etmektedir. Özellikle ucuz maliyet ve hızlı inşa tekniği nedeniyle son zamanlarda daha yagın inşa edilen toprak dolgu barajlar için, "yeniden değerlendirme" daha fazla önemli olmaktadır.

Baraj yıkımımnın çok sayıda nedeni olmasına ragmen bunlar arasında en fazla karşılaşılan; borulanma, ve bunun gerek baraj gövdesinde vetaban zemininde yol açtığı erozyon, gerek inşa ve gerekse işletme aşamasında depremin yol açtığı şev stabilitesi sorunları, dolu rezervuar sonrasında ani su çekilmesine bağlı olarak göçme en sık görülen yıkım nedenleri olarak sıralanabilirler.

Bu çalışma kapsamında Irak sınırları içinde inşa edilmiş olan, Shauger toprak dolgu barajının stabilitesi çok sayıda analiz yapılarak yeniden değerlendirilmiştir. Memba ve mansap şevleri, inşaat aşaması sonrasında, karalı sızma akımının oluştuğu işletme aşamasında, ani ve yavaş su çekilmesi durumları için yeniden analiz edilmiştir.Karalı ve transient akım koşullarında borulanma riski irdelenmiştir.Ayrıca dinamik analizler gerçekleştirilerek,muhtemel bir deprem sonrasında stabilite, deformasyon düzeyleri ve sızma yeniden incelenmiştir.Analizlerin tümü, baraj analizlerinde son zamanlarda çok kullanılmakta olan geostudio-2012 programı kullanılarak gerçekleştirilmiştir.

Yapılan analizler, mevcut barajın çok sayıda göçme kriterini sağlamasına karşın, ani su çekilmesi ve dolu rezervuarda deprem durumundaşev stabilitesiniyeterince sağlamadığını göstermiştir.

Anahtar kelimeler: toprak dolgu baraj, baraj güvenliği, yeniden analiz, şev stabilitesi

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CHAPTER ONE

INTRODUCTION

1.1 INTRODUCTION

From earliest times, the human beings constructed the earth dams, to storewater to handle their primary needs in water supply and agricultural uses. Earth dams have been used since the earliest times; they are simple compacted structures that dependon their mass to resist sliding and overturning and are the most familiar type of dam found worldwide. The background of earth dam constructions in various forms and dimensions returns to long times ago [6] [28].

Modern machines and developments in soil mechanics have greatly increased the safety and life of earth dams. Nowadays, construction of earth dams is greatly appreciated in worldwide, so that; the number of these dams exceeds other dam types. However, the break of earth dams is considerable, comparing with other dams. Studies illustrate that the breakage due to the gradual occurrence of earth dam break, informing downstream inhabitants about the incident and sudden failure of dam as well as transferring them to safer places in upstream is quite possible [6] [28].

Dam engineering is an engineering science which enables the use of water resources as flexibly as possible, to fit the needs and schedule of humans. It could be used to draw benefit from these water resources or to give protection against threats due to sudden changes in the trend followed by these same resources. As a concept, dam engineering has been developing for thousands of years as man's thinking abilities grew and its need to control its environment grew with it [30]. Designing and construction of an earth dam is one of the key challenging in the field of Geotechnical engineering, because of the nature of the varying foundation condition and the range of properties of the material available for construction. The major advantages of the earth dams are easily adapting to the foundation and accommodate even in difficult site condition. The most common and basic earth dams are known as homogeneous. This type of dams entirely constructed with same material. However, at present designing of earth fill dam with relatively impervious core is increased for the purpose of controlling seepage through the dam [5].

Evaluating the stability of slopes and water seepage in soil is an important, interesting, and challenging aspect of civil engineering. However, they are a major concern when those unstable slopes and/or seepage would have an effect on the safety of dam then the safety of people and property. Over the past decades, development of new and more effective types of instrumentation to observe the behavior of dams, and use of computers to perform thorough analyses improved understanding of the principles of soil mechanics that connect soil behavior to dam stability in several critical cases such as stability of the dam slopes at the completing of the dam with and without full water level in the reservoir, rapid draw down of water in the upstream side, earthquake effect, pore water pressure and seepage through and beneath the dam [23].

If the dam break it will leads to release a huge amounts of water, that usually will be in huge waves running toward downstream areas, which causes hazardous disasters floods. Dam break usually leads to inevitable financial losses and the death tolls depend on the population of downstream. The losses may be reduced by alarming before disaster [6].

In the last decades several dams in the worldwide have failed because of inadequate dam engineering works involved. Therefore a careful attention, regular inspection and re-evaluation should be focused on earth fill dam to prevent possibilities of failure [5].

1.2 PROBLEM STATEMENT

According to the statistics, there was in every year in the last decades one or more dam failure occurred in the world. Dam failure is a disaster and makes a huge hazards, it causes damages in everything in the receptor area in downstream. Usually there are huge losses in people, properties and environment. Furthermore in the last years the climate of the region tends to oscillation. On winter 2013, there was an intense rainfall occurred in Iraqi Kurdistan Region that resulted an increase of water level more than expected level, that is critical case. Therefore the geotechnical re-assessment of dam safety is very important issue.

1.3 OBJECTIVE OF THESIS

The purpose of this thesis is to make geotechnical re-assessment (re-evaluation) of Shauger earth dam exists in Iraqi Kurdistan Region and to find out the safety of the dam. The work includes a numerical study on earth dam safety using finite element method by using Geo-Studio 2012 computer software.

1.4 PREVIOUS WORK

The following related work review coducted to find out the previous works and previous researches on topics related to evaluate the earth dam safety. It is worthwhile to point out the following works.

In (2010) Umaru and et al. had studied structural failures of Cham earth dam in Nigeria. The study investigated the reasons of the earth dam failure. Cham dam failed in 1998 after completed in 1992. The study revealed that the failure occurred as a result of hydraulic failure (overtopping) and seepage failure (piping at downstream toe of the dam). There were faults in the design and in the construction of the dam. The freeboard amounted was only to 0.636m. The spillway was designed to have a capacity of 200m3/s, while in the field its capacity was 75 m3/s [32].

In (2010) Ambikaipahan carried out analyzing the seepage and stability of the Arbogen dam in Norway and the reliability for the extreme flood condition caused by the intense rainfall. The analyzed was done by using the GeoStodio software. The dam has been failed during an intense rainfall occurred in 2010 with three major slides along the downstream slope. The seepage and stability analysis have been done for 3 selected profiles across the embankment. Seepage analysis results for the profile 1 shows that the seepage through the embankment at normal conditions was 6.22×10-5m3/s and it is increased up to 9.89×10-5m3/s during the flood. This shows that the deposit is not well compacted.

The factor of safety for the normal dam condition was 1.076 and during the flood, it was reduced up to 0.799 from the stability analysis which shows, getting more critical stability condition [5].

In (2010) Khattab studied the slope stability factor of safety (FoS) against Mosul dam embankment sliding considering a rapid drawdown and earthquake condition by using Bishop, Morgenstern-Price and Lowe-Karafiath methods. Saturated unsaturated condition was considered assuming the shear strength parameter (Ø) to be (0, 0.5Ø, Ø). He used Geo-Slope office software for his study. Seepage analyses were done for three period rapid drawdown (8, 21, 30days). Results indicated that the critical slope stability occur during 8days drawdown. The effect of considering the unsaturated conditions was found to increase the FoS. The results of slope stability FOS during the earthquake shows a very critical FOS near and below 1.0 for all the three rapid drawdown periods (8, 21, 30days) and for all (Ø) values [18].

In (2011) AL- Gazali used a finite element model to analyze the two- dimensional steady state seepage of water through AL- Adheem Dam and its foundation. He used the computer program TDFIELD to determine the head at nodes and the hydraulic gradients. He compared between the obtained hydraulic gradient values and that recommended. He obtained that AL- Adheem dam is safe against seepage and there is no need to use any control devices for seepage (cut-off, dense core, and grout curtain) [3].

Nourani and Ghaffari (2012) in their study assessed slope stability in embankment dams using Artificial Neural Network (ANN) (case study on zonouz embankment dam). For this purpose they use two-dimension model. Feed forward back propagation with Levenberg-Marqardt algorithm was employed for training ANN. The result indicated that the base angle has more influence on safety factor than any other parameters which are (slice width, slice weight, base normal force, base shear resist force, etc) [20].

In (2013) Sachpazis studiedthe slope stability and made analysis of the original Carsington earth dam failure in the UK. In 1984, about 400m length of the embankment upstream slipped about 11 m. At that time, the embankment construction was completed and the dam maximum height was 35 m. The slope stability was analyzed by using Finite Element Analysis technique at the three loading cases: The right after construction condition, the steady seepage condition and the rapid drawdown condition. From the results, the reasons of the slope failed became obvious as follows: The minimum factor of safety forall cases was less than one (0.96, 0.81, and 0.5) which means imminent failure conditions. The mistake of the designer was that none of the assumed slip surfaces passed through the upper part of the core section at crest [10].

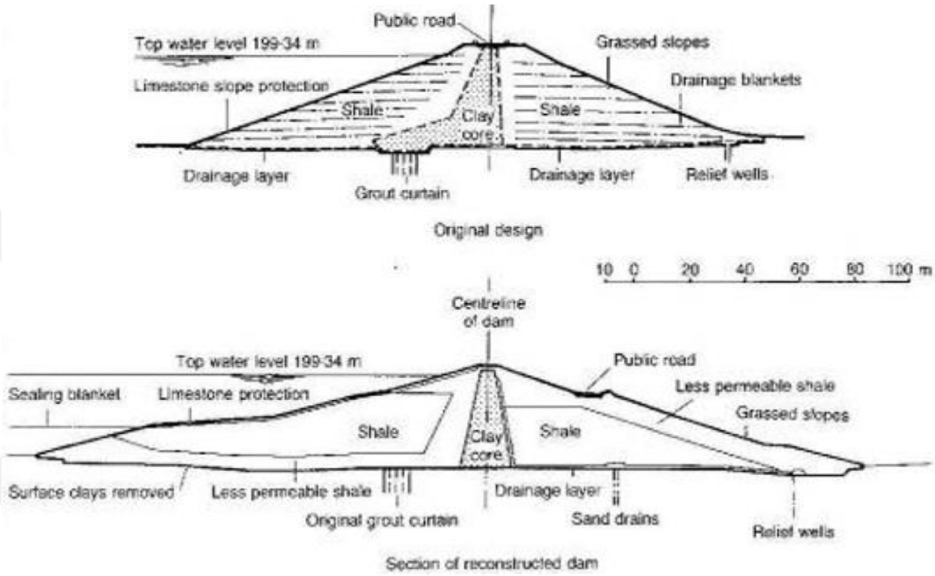

Figure1.1. Representative cross section of the original and reconstructed Carsington earth dam [10].

In (2013) Hasani and et al. used the Seep/W software for seepage analysis in Ilam earth dam. In order to evaluate the effect of the type and size of mesh size on the total flow rate and total head through the dam cross section, they considered four mesh size: coarse, medium, fine and unstructured mesh. Result showed that average flow rate of leakage under the different mesh size for Ilam dam equal 0.836 liters per second for the entire length of the dam. Also they used Slope/W software to analyze each slope with Bishop, Janbu, ordinary method of slides and Morgenstern methods they considered the minimum safety factor in each of these methods, as a safety factor of slope stability. They found out that the slopes are in safe side [16].

In (2013) Pham et al. studied the factor of safety for slope stability of Yashigou dam in China. The factor of safety were checked to the dam without water level, steady-state water level and water level drawdown by using numerical finite element method analysis using Geostudio software and compared with Morgenstern-Price method. He found that the dam is in safe side according to the results of both methods. And he suggested that the finite element method is more realistic, more accurate and more promising [22].

The work in this thesis includes using finite element technique to investigate the safety of Shauger earth dam (which is located in Iraqi Kurdistan Region) against the slope stability, self stresses, seepage, and earthquake, by using Geo-Studio software.

1.5 ORGANIZATION OF THE THESIS

The entire thesis is divided into six chapters along with an appendix included at the end of the thesis.

The first chapter consists of introduction, objective of thesis, problem statement and then review of previous work. The second chapter gives a detailed background about the earth dams begins with introduction, definition, types of dams, purposes, characteristics, main parts, important studies and designs, forces acting on dams, design criteria, dams failure and assessment of dam safety. The third chapter explicitly describes the basic information for the case study dam (Shauger earth dam) and its geotechnical properties. Chapter four speaks about methodology and presents the details of the numerical and finite element analysis methods in Geo-studio software program. Chapter five consists of analysis results and discussion. The last chapter provides conclusions and the scope for further research; this chapter is followed by references and then appendix A.

CHAPTER TWO

EARTH DAMS

2.1 INTRODUCTION

Long times go when earth embankments have been used to store and control water.Earth damis a well compacted soilstructure, it depends on its huge mass to be stable against failure, earth dams are the most common kind of dams found.Modern machines and developments in soil mechanics in the recent centuries lead to increase the age and safety of earth dams [28].Nowadays earth dams are very important to integrate the infrastructure of Nations and have a significant role to manage water resources in rivers.Earth dam is a very economical solution for complicated geological sites which are vary from site to another site, because earth dams have flexibility in the design and construction more than other dam types [9].

The variation in the foundation conditions and material properties that will be used in the dam construction is making the design and construction of earth dams to be some complex. To design an earth dam, at first step hydrological, topographical and geological studies for the whole site should be done. Then the second step includes to test and fined the properties and characteristics of material that will be used to construct the earth dam. Then in the next step, based on the all previous studies the design will be begin take in consideration all technical criteria and important requirements. The next step is the construction of dam. The final step is the operation and management the dam by continuous monitoring, maintenance and periodically re-evaluation the design and the perfonmance of the earth dam [9][25].This chapter gives a detailed background about earth dams.

2.2 DEFINITION

Earth dams are simple structures which stand on their self-weight to prevent the sliding and overturning. These dams are the most widespread type of dams known in the world. At the earlier time the earth dams are constructed to divert massive water body and protect the community. Later it was structurally improved and used to construct the reservoirs [5].

Earth dams are the dams which are built of compacted soil or rock fragments, with a nearly trapezoidal cross section. It is an engineering structure constructed to reserve water for various purposes.

2.3 TYPES OF DAMS

In general dams can be categorized according to their main building materials into two types, concrete dams and earth dams. Earth dams can be categorized into two subtypes, soil fill dams and rock fill dams.

Soil fill dams are built from nearby soil, and had a huge body comparing with concrete dams. While rock fill dams are also has a huge body, but built from nearby rock fragments if the nearby borrow pits is rock and there is no enough amount of soil nearby [30].

The following summarize structure types of dams:

2.3.1 Gravity Dams

Keban dam on the Firat river (Figure 2.1) is an example of a gravity dam in Turkey[13].

Figure 2.1. Keban Dam [13].

2.3.2 ARCH DAMS

Arch dams transmit most of the horizontal thrust of the water behind them to the abutments by arch action and may have comparable thinner cross-sections than gravity dams. Arch dams can be used only in narrow canyons where the walls are capable of withstanding the thrust produced by the arch action [13]. Dokan Concrete Dam on the Small Zab river in Kurdistan of Iraq is an example of arch dams,see figure 2.2.

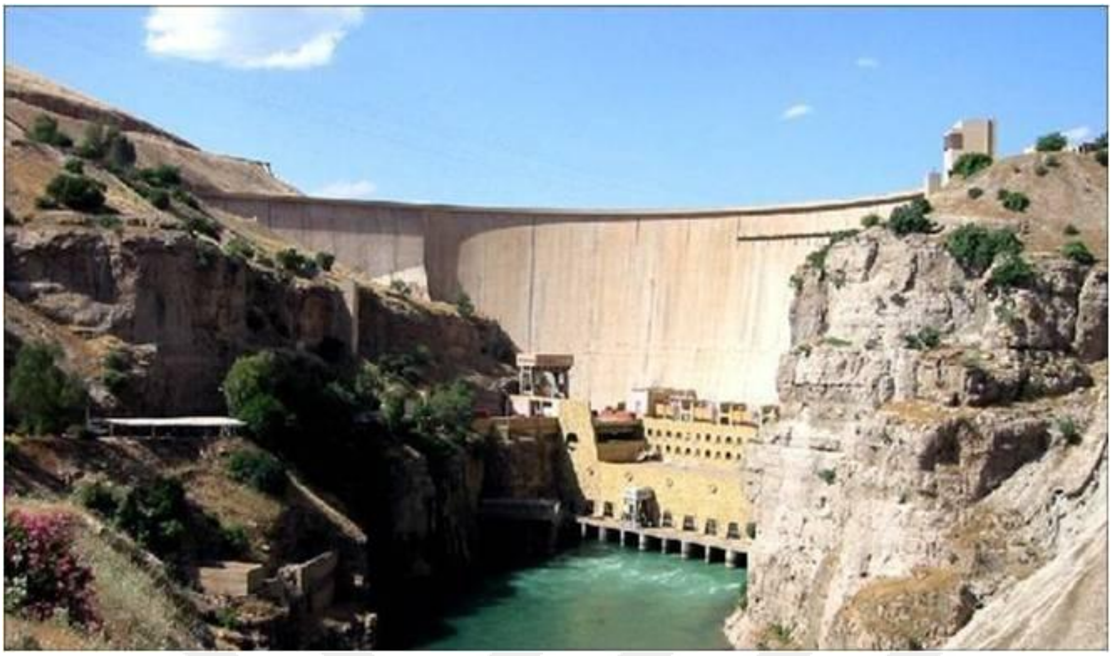

Figure 2.2. Dokan Concrete Dam.(Source: Web_1 2015)

2.3.3 BUTTRESS DAMS

Dam which has a series of supports in its face called butter dam. The face of butter dams may take forms such as curved or flat.

Figure 2.3. A buttress dam Koot dam.(Source: Web_2 2015)

2.3.4 EMBANKMENT DAMS

Embankment dams can be divided into two types as earth dams and rock dams [13].

2.3.4.1 ROCK DAMS

The main body of a rock dam have arock mass, that is supply to obtain the repose angle. By this way slope of about 36 degrees is obtained. Stability and impervious zone can be constructed by using differnt size of rock element. By using the one of material such as concrete face, asplalt, impermeable soil, concrete slash or other materials an impervious membrane can be formed. Some example of rock dam in Turkey are Hirfanlı on Kızılırmak and Hasan Uğurlu which constructed on Yeşilırmak (figure 2.4).[13].

Figure 2.4. Hasan U_urlu Dam [13].

2.3.4.2 EARTH DAMS

An earth dam is made up partly or entirely of pervious material which consists of fine particles usually clay, or a mixture of clay and silt or a mixture of clay, silt and gravel. They are principally constructed from available excavation material. The dam is built up with rather flat slopes [13]. Duhok damis an example of earth dams in Kurdistan of Iraq.Figure 2.5 shows an earth dam.

Figure 2.5. Duhok Earth dam.(Source: Web_3 2015)

In general there are two types of earth dams:

2.3.4.2.1 HOMOGENEOUS EARTH DAMS

Homogenous earth dams are mainly composed of one type of material, besides the slope protection material [13]. In homogenous there are problems of pressureand seepage. In case of seepage is excessive then it can lead to instability of the downstream face. Usually using a rock toe or gravel drainage layer as a blanket to solve the problem of seepage in the downstream where there is impervious foundation. Coarse sand and gravel shouldoverlain onrock toe to prevent embankment materials being washed into it, which could reduce the permeability of the toe. If the foundation is exist stream beds then the natural drainage layer can be worked as blanket or drainage layer. The seepage relief structure should only be beneath the downstream and not extend to upstream that could lead to direct seepage from upstream.

In general, this type usually have flat slops (1:3 U/S and 1:2 D/S) to giveslope stability. The reson of that theupstream slope is flatteris to Allow the part under water level to resist failure. And also combined force of weight of the water with the weight of the dam, may be than the horizontal thrust gives by the water against the embankment. The reservoir level should not rapidly dropdown or rise too fast. A rapid rise in the reservoir level may leads to erosion in the cracks and fissured parts [28]. Figure 2.6 shows a homogeneous earth dam.

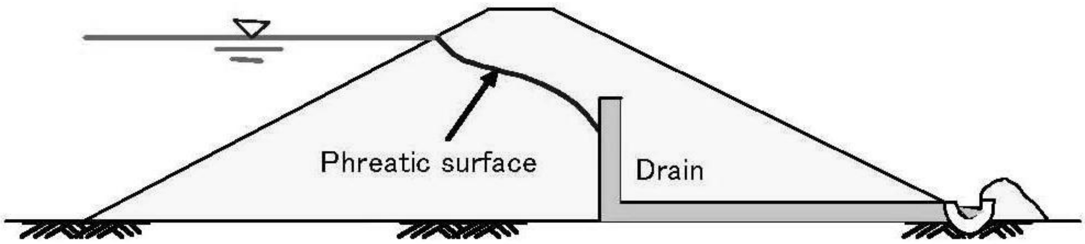

Figure 2.6. A simple design of a Homogeneous earthfill dam [5]

2.3.4.2.2 ZONED EARTH DAMS

This type of This type is a good alternative especially for large dams.In zoned type damsthe seepage hazards are reduced to a minimum. The costs are higher compared with homogeneous type, Because the works are done in three zones: upstream zone which is semi impervious, core zone which is impervious and downstream which is pervious, the materials of all these three zones should be excavated from various borrow areas, thus increasing excavation and movement costs. Howeverslopescan be reduces to 1:22 for upstream and to 1:1.75 for downstream, sometimes may be 1:2.25 for upstream and 1:2 for downstream when the permeability of available materials is poor. Rock toe should be built, which is required for draining the seeped water in the downstream and the riprap(stone) facing on the upstream face is very important to protect the face from waves attack. The end of embankments and abutments and spillways need to protect from erosion by using concrete or shielded by gabions. As an alternative to clay cores, such as heavy duty plastic sheetinghave successfully used. However these materials may attacked termites and may borrowed by animals and not resist the settlement of the soil after dam construction completed [28]. Figures 2.7 and 2.8 shows a zoned earth dam.

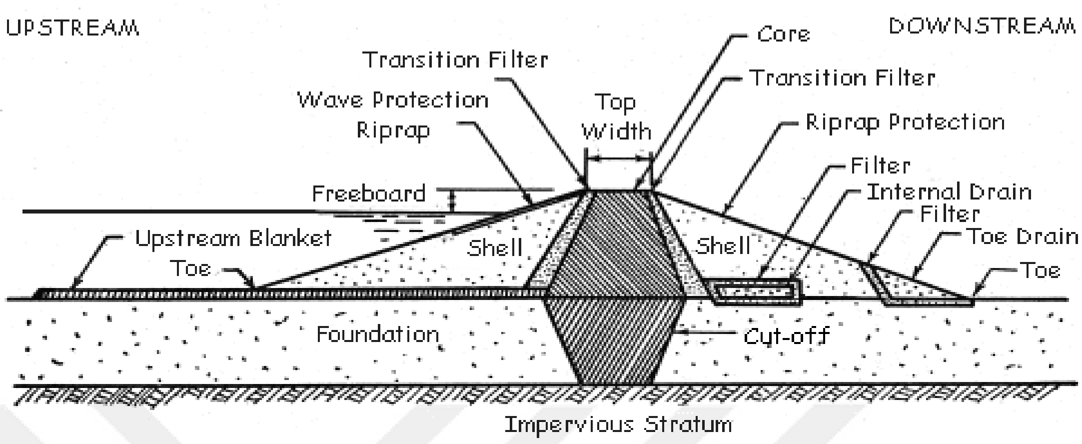

Figure 2.7. A simple design of a Zoned earthfill dam [28]

Figure 2.8. A zoned earthfill dam under construction(Source: Web_4 2015)

2.4 PURPOSES OF EARTH DAMS

Earth dams are the most common type of dams known in the world. At the earlier time the earth dams are constructed to divert massive water body and protect the community. Later it was structurally improved and used to construct the reservoirs [5].

Dams are constructed to reserve water to use for various purposes. The main purposes of any dam are [13]:

- For Agriculture and Irrigation.
- For Water supply.
- For Flood protection.
- For Tourist purposes.
- For Power generation.
- For Navigation.
- Recharging ground water.

The function of a dam may be one, many or all above purposes.

2.5 CHARACTERISTICS OF EARTH DAMS

The advantages ofconstruction of earth dams are local natural materials are used, the unit cost of compacted fill is much less as compared to concrete dams, so their total cost is usually less, they are designed as gravity dams, comparing with other types of dams earth dams needs small equipments and plants for construction, and its foundation is very simple and less requirements, it is natural foundation with few treatments, because earth dams have wide base which means spreads the load and no needs to construct a huge rigid foundation, just the natural earth almost is sufficient and resists settlement and movements. The disadvantage of earth dams are: it is easily damaged by overflow water or raining, adding adequate spillway to solve the problem of overflow [28] [33].

2.6 MAIN PARTS OF EARTH DAMS

Most dams are generally comprised of some main components. Some of the main components included are an impervious core, a drain and/or a filter, earth fill material, etc. Most dams are constructed by putting an impervious core in the middle and more permeable material on the sides and this is achieved by compacting successive layers of soil. The following principal dam sections' functions:

- **Impervious core** which is provided for the purpose of keeping the retained material on the upstream side by controlling seepage through the dam.
- **Drain** the seeped water through the dam should collect and drained out to reduce uplift pressure, erosion and piping [9]. Controls erosion of core material and dam foundation by seepage water by being used as a horizontal drain in the latter case. When used as a vertical drain, it also controls buildup of pore pressure in the downstream face.
- **Filter** which will prevent particle migration/erosion of the core material due to excessive seepage forces into the fill material or through the rip rap.
- **Fill material(shell)** for giving the whole structure weight for stability against an immense destabilizing force from the retained material and it is usually free draining to allow passage of seepage under and through the dam [30].
- **Cutoff trench and core(diaphragm)**Cutoff is very important for the earth dams it increases the stability of the dam and reduces the seepage because usually clay material is used. Cutoff trench should be excavating until reaching a good foundation or rock layer. In case of underlying rock bed beneath the trench is fissured then usually grouting with thick slurry and cement mixture should be used [28].
- **The spillway**It is very important part for earth dams and should be carefully designed to be adequate for the dam in case of peak flood. Spillway should be carefully constructed and protected by concrete to prevent erosion [28].
- **Discharge (Outlet works and conduits)**It is used for releasing water to downstream regularly and to control the reservoir level to be within required levels according to the regulation requirements. It should has capacity to discharge 90% of the storage within 14 days supposing no inflow. It should has

gate or valves to control the discharge. Outlet conduits should have capacity to resist the loads of embankment and water [21].

2.7 IMPORTANT STUDIES AND DESIGNS FOR THE EARTH DAMS

The following main studies should be done to design an earth dam satisfies the engineering requirements and specifications [31]:

- Geological studies: geological survey should be achieved for the site, it includes the tectonic and earthquake studies for the Stratigraphy stratification formation, magma formation, faults if there is and etc. of the dam site and reservoir area, to find out that the location is suitable or not for dam construction from geological point of view.
- Hydrological studies: collecting all the weather and climate reports data from meteorological stations near the site of the concerned area for all previous years, to analyze the data and find out the maximum amounts of rainfall and runoff for the catchment area and the river discharge.
- Topographical studies: This study aimed to invest (exploitation) abilities of the earth surface of the dam site and the reservoir area.
- Geotechnical studies: it includes in-situexploration and taking samples for laboratory tests to find out the geotechnical properties of the site earth materials, and designing the dam body and its all accessories depending on the above mentioned studies information.

2.8 FORCES ACTING ON DAMS

Main forces which are acting on dams can be summarized as follows:

2.8.1. WATER PRESSURE

Water pressure acts perpendicular to the surface of the dam (see figure 2.9) and is calculated per unit width as follows:

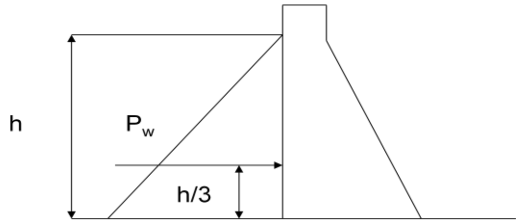

Figure: 2.9. Illustration sketch for hydrostatic water pressure.

$$P = \frac{1}{2}\gamma_w h^2 \qquad \text{..2.1}$$

Where γ_w is the specific weight of water and h is the height of water.

Dams are subjected also to uplift force (U) under its base. Uplift acts upward see figure 2.10.

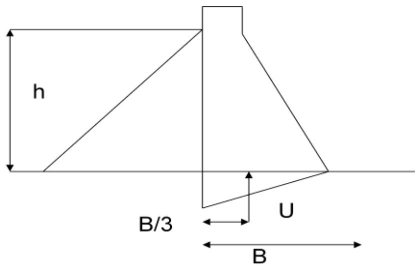

Figure. 2.10: Illustration sketch for uplift pressure.

$$U = \frac{1}{2}\gamma_w hB \qquad \text{..2.2}$$

Where B is the width of the base of the dam.

Water has additional force it is internal pore pressure effects the dam besides the external pressures.

2.8.2. WEIGHT OF DAM

The weight of the dam itself is another force that acts on dam structures.

The weight of the dam is calculated as follows:

$$W = \gamma_m x Volume \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.3$$

Where γ_m is the specific weight of the dam's material.

2.8.3. EARTHQUAKE FORCES

Dams are subjected to vibration during earthquakes. Vibration affects both the body of the dam and the water in the reservoir behind the dam. Vibration forces are function of both the intensity of the earthquake (Rechter Scale) and its duration. The most danger effect occurs when the vibration is perpendicular to the face of the dam.

The earthquake exerts horizontal force and the level of this force is about ten percent of total weight of Dam [13].Body force acts horizontally at the center of gravity see figure 2.11 and is calculated as:

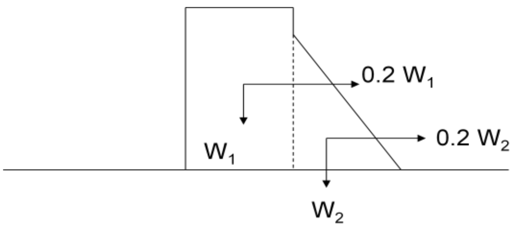

Figure: 2.11. Illustration sketch for earthquake forces.

$$P_{em} = \alpha W \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.4$$

Where α (Is taken 0.2 for practical reasons) is the earthquake coefficient and W is the weight of the dam.

Water vibration produces a force on the dam acting horizontally, see figure 2.12.

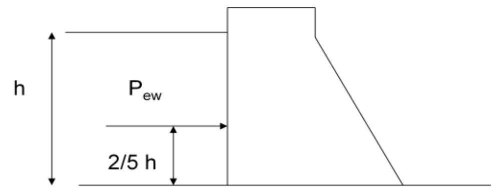

Figure: 2.12. Illustration sketch for Water vibration forces.

$$P_{ew} = \frac{2}{3} C_e \alpha h^2 \quad \text{......} 2.5$$

Where C_e is another coefficient (0.82) and h is the height of the water.

2.8.4. ICE AND WAVES FORCES

A thichk layer of ice can be formed on reservoir surface in cold climates. Due to expansion of ice sheet a big amount of force can ocur about top of the dam. Therefore this part of the dam should be constructed in enough tickness to bear the pressure. The upper part of the dam (above the water level) is subjected to the impact of water waves. The maximum wave pressure per unit width is:

$$P_v = 2.4 \gamma_w h_v \quad \text{......} 2.6$$

Where h_w is the wave height which depends on reservoir length and the wind speed.

2.9 EARTH DAM DESIGN CRITERIA

An earthfill dam should be designed with relatively flat slopes to reduce the possibility of failure. The design is unique for each earthfill dams because of the location of the dam and the variety of materials to be used for construction. Purpose of the dam also plays an important role on design criteria. The factors mentioned above bring hard to define general design criteria. However, every design criteria must be included the following fundamental design aspects [5]:

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

A dam may lose its performance by time because of the long term changes in the properties of constructed materials. To maintain the performance of a dam, critical conditions such as earthquake, overtopping and unexpectable increase of seepage quantity are should be overcome with controlling structures such as filter – protected chimney drains, horizontal drain blankets, foundation cut – offs, relief wells and abutment drainage curtains [5].

2.10 DAMS FAILURE

The failure is happen due to uncontrolled and unintended seepage of storage water, or an event whereby a dam is rendered unfit to safely retain water because of a total loss of structural integrity [4].

2.10.1 MODE OF FAILURE

The mechanism that the hazards occurs failure is named as mode of failure. In following some of important mode of failure were presented. [4] [34].

2.10.1.1 OVERTOPPING Uncontrolled flow over the dam crest. Figure 2.13 illustrates this case.

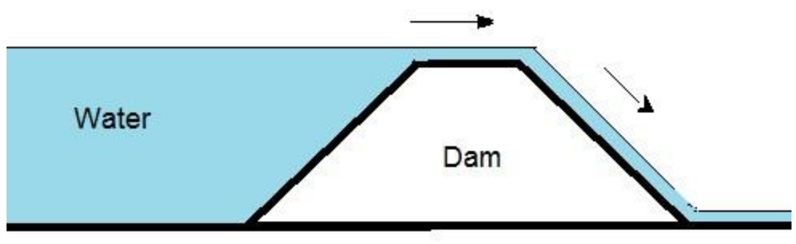

Figure: 2.13. illustration of overtopping case.

2.10.1.2 DAM BREACH

Uncontrolled flow of water though the dam's main structure, which includes its foundation. Figure 2.14 shows dam breach case.

Figure 2.14. dam breach case.(Source: Web_5 2015)

2.10.1.3 FOUNDATION FAILURE OF EMBANKMENT DAM

When the design loads are greater than the bearing capacity of foundation the ultimate failure of foundation of dam occurs. Figure 2.15 foundation failure type of embankmentwere presented.

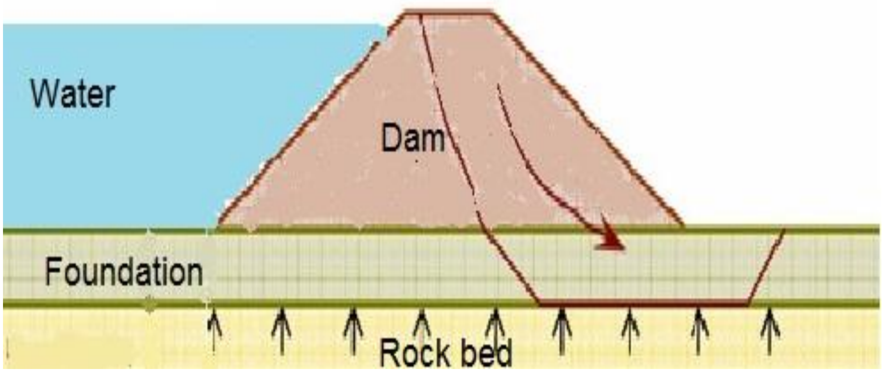

Figure 2.15. Foundation failure of embankment dam.

2.10.1.4 INSTABILITY OF EMBANKMENT DAM

One or more slips occur within the embankment because design loads are exceeded or through a reduction in shear strength of embankment-fill materials over time.

2.10.1.5 OVERFLOW FAILURE DUE TO SPILLWAY DAMAGE

Overflow or spillway is unable to contain flow during flooding and uncontrolled flow results. Structural failure may occur. Figure 2.16 shows a spillway damage example.

Figure 2.16. Flood event of June 2005 that resulted spillway damage at Boltby reservoir [4].

2.10.1.6 STRUCTURAL FAILURE OF EMBANKMENT DAM

An increase in permeability and/or reduction in strength of core occurs over time, and results in ultimate failure of the embankment dam structure. Muddy discharge downstream of dam as a result of increased seepage and internal erosion is an indication of this case.

2.10.1.7 STRUCTURAL FAILURE OF EMBANKMENT DAM (LOAD EXCEEDED)

Ultimate failure of embankment dam structure occurs because of design loads are exceeded and/or through deterioration of embankment-fill materials over time. Figure 2.17 illustrates this case.

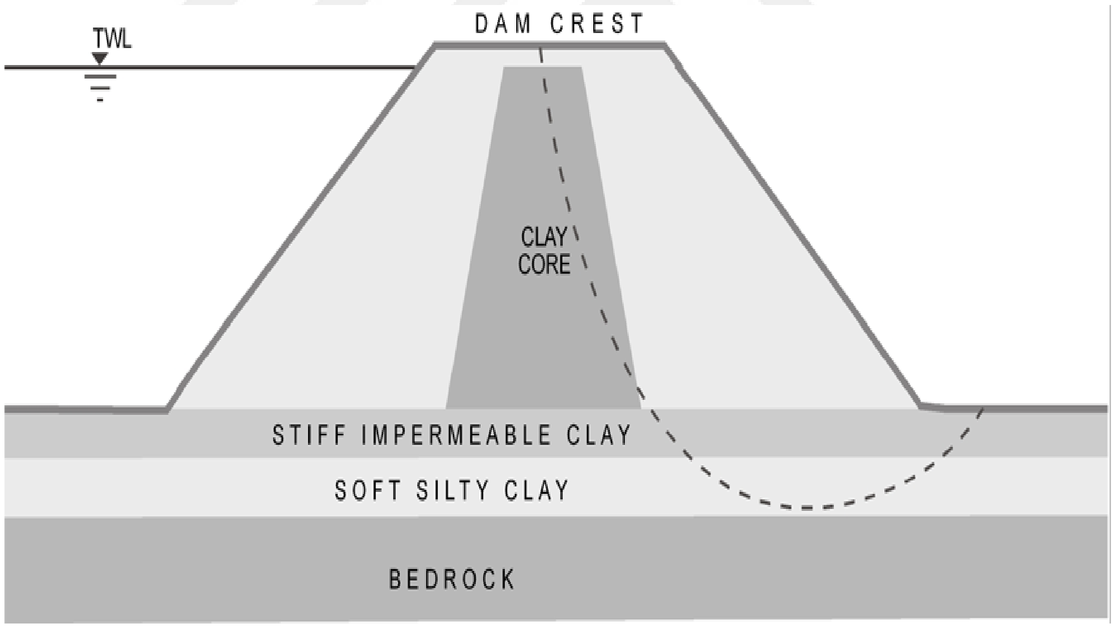

Figure 2.17. Structural failure of embankment dam (load exceeded) [4].

2.10.1.8 UNCONTROLLED FLOW BECAUSE OF APPURTENANT WORKS FAILURE

Structural failure of one or more appurtenant works occurs because of a change in load conditions or deterioration of works over time that results in the uncontrolled release of water.

2.10.1.9 UNCONTROLLED SEEPAGE

Uncontrolled passage of water through, underneath or around a dam.

2.10.2 REASONS OF DAM FAILURE

The following threats are some reasons of failure events [4]:

• ageing

• aircraft strike

• animal activity

• changes in groundwater flow/chemistry

• earthquake

• extreme rainfall/snow flood

• failure of nearby infrastructure

• failure of reservoir in cascade upstream

• human activity

• ice/frost

• layout, design or construction inadequate or inappropriate

• sunlight

• terrorism/sabotage/accident

• wind

The threats listed above may lead to development of one or more following damages which cause dam failure [4]:

- Differential settlement or deformation
- Rapid drawdown Lowering the water level in a reservoir faster than the pore pressures can dissipate from the upstream shoulder of an embankment dam.
- Reduced freeboard due to settlement or internal erosion.
- Seepage can cause internal erosion.
- Concentrated erosion
- Backward erosion (piping)
- Internal erosion along an appurtenant structure
- Animal activity by create burrows, holes, tracks etc.
- High pore-water pressures lead to increased seepage, hydraulic fracture or instability.
- Blockage of spillway
- Inadequate energy dissipation at the end of a spillway.
- Out-of-channel flow in spillway.
- Overflow capacity exceeded insufficient hydraulic capacity.
- Deterioration of core or foundation or upstream face.
- Cut-off deterioration.
- Blocked drains and relief wells no longer carry out their design function to control uplift pressure.
- Blocked screens reduces flow capacity.
- Failure of controls, valves or gates.
- Local run-off on the upstream or downstream side this can cause local erosion.
- Wave attack due to wind or external effect.

Researches indicate that the main factors in dam breaks are as follows: Overflowing due to low capacity of water discharger (34%), foundation defects (30%) and piping and seepage (26%) [6].

2.10.3 HAZARDS DUE TO DAM FAILURE

If the dam break it will leads to release a huge amounts of water, that usually will be in huge waves running toward downstream areas, which causes hazardous disasters floods. Dam break usually leads to inevitable financial losses and the death tolls depend on the population of downstream. The losses may be reduced by alarming before disaster [6].According to the statistics, in every year in the last decades there is one or more dam failure occurred in the world. Dam failure is a disaster and makes a huge risks, it causes damages in everything in the downstream. The following are some of losses:

- Human victims (killing, injuring).
- Dam failure means lose the dam itself, its benefits and purposes.
- Destroys Roads bridges, airports and cars.
- Destroys houses, buildings, warehouse, power plants and factories.
- Destroys agricultural lands, and projects.
- Destroys power lines and water irrigation and drainage channels.
- Losses in goods, furniture and etc.
- Indirect losses such as: diseases, pollution, the need to remove the damage things and flood remains.

All above losses mean too financial losses in billions. Therefore the assessment of dam safety is very important issue.

2.11 ASSESSMENT OF DAM SAFETY

The earth dam location, height, long, capacity, etc. determine after completing the topographical, hydrological, geophysics and geological studies. Then geotechnical studies will begin to design the earth dam body from the available nearby borrow materials. The safety assessment of an existed dam includes geotechnical, hydrological, ageing, etc. assessments. Geotechnical assessment involves checking the dam safety against the slope stability analysis in all critical conditions, seepage analysis, and self weight stresses, these analyses usually done by using new professional computer software, like Geo-Studio or/and Plaxis computer programs.

CHAPTER THREE

SHAUGER EARTH DAM

3.1. PREFACE

This chapter includes basic geotechnical information about Shauger earth dam. This information collected from directory of dams in Erbil.Shauger earth dam selected for this study due to the availability of extensive geotechnical data for the analysis and that this dam is a good represented sample for many earth dams constructed or under construction in the Iraqi Kurdistan Region.

3.2. INTRODUCTION

The Shauger earth dam is located in Iraqi Kurdistan Region, it is about 24 meter height its crest crown is designed to be at 506.2 m.a.s.l level at the highest sections its width is 7 meter. At the crown the crest slopes 2% to both upstream and downstream directions. The crest of the dam is designed to carry normal car vehicle traffic. The upstream face of the dam has a slope of 1:2.75. The downstream slope of the dam designed to have a 1:2.5 slope with 2 meter berm. The upstream slope is covered with a hand placed riprap of 40 cm thickness, A bedding material of 20 cm thickness underlie the rock fragments of riprap, it consists smaller fragments of stones and cobbles. The downstream slope is covered by coarse gravel and fine cobbles of 40cm thickness to protect the slope from erosion. A surface drainage canals is provided at the contact areas of the dam with the abutments and natural ground to drain the rain water at the lowest level at the downstream to prevent erosion at this area. A cofferdam which its crest is at 494 m.a.s.l. level with 4m width is a part of the main dam body and its crest will be a berm for the upstream slope. The upstream face of the cofferdam has a slope of 1:2.75, while the downstream slope of the cofferdam designed to have a 1:2.25 slope.

3.3. CORE SEAL MATERIAL

The dam consists of a seal core of clayey silt soil it is classified as (CL) according to unified soil classification system. The crest of the core is at 505m.a.s.l level and its width at the core crest is 4 meter it expanded with a slope 1:3.75 to the direction of the foundation to reach about 15 meter at lowest level of the dam. Core material is extended to different depths into the alluvium deposit at the different locations to act as a sealing cut off. It reach the bed rock and it extracts it at least 0.75 meters. The dry density for the core is 1.95 gr/cm^3.

3.4. SHELL MATERIAL

Shell material classified as poorly graded gravel; it compacted to a relative density not less than 85 % or 95% degree of compaction of modified compaction test.The dry density for the shell is 2.0 gr/cm^3.

3.5. FILTER MATERIAL

The filter material that provides both permeability and piping criteria sandwiches the core slopes from both upstream and downstream sides it has a 1.8 m thickness. The filter material is designed with two layers each of 90cms. The filter layer attached to the core is designated filter 1 it comprises from sand material. The filter layer attached to the shell material side is designated filter 2 it comprises gravel material. A blanket filter with same sand and gravel material each of 100cm thickness is provided at the downstream of the dam.

3.6. EARTHQUAKE

Shauger dam site is located at zone 2.0 indicating k_h value of 0.2 , which is used in the stability analysis. The earthquake effects has been considered by introducing the horizontal acceleration coefficient k_h equal to 0.20 .

3.7. PERMEABILITY

The values of coefficient of permeability are as follows:

Shell material k(cm/sec) = $1x10^{-4}$

Core and cutoff material k(cm/sec) = $3x10^{-7}$

Foundation material Quaternary k(cm/sec) = $2.5x10^{-5}$

Rock bed (claystone) k(cm/sec) = $5x10^{-6}$

Filter k(cm/sec) = $5x10^{-2}$

3.8. SHEAR STRENGTH PARAMETERS

The values of Shear strength parameters are as follows:

Table 3.1. Shear strength parameters.

	Core of dam		Shell		Filter		Bed Rock	Quaternary	
	C (kPa)	φ (deg.)	C (kPa)	φ (deg.)	C (kPa)	φ (deg.)	C (kPa)	φ (deg.)	C (kPa)
During construction and End of construction	150	0	20	35	0	33	4000	35	0
Steady state seepage under active conservation pool	70	11	20	35	0	33	4000	35	0
Steady state seepage with earthquake	70	11	20	35	0	33	4000	35	0
Sudden draw down	70	11	20	35	0	33	4000	35	0

3.9. SUMMARY

The basic information about Shauger earth dam is given below:

1-Type of Dam: (Earth Dam)

2-Height of Dam: 24 m

3- Stream Bed level : (483) m.a.s.l

4-Width of the valley at the Dam site: (40) m

5-Crest width: (7) m

6-Length of the Dam (at the crest level): (93.5) m

7-Max flood level: (504) m

8-Max dam operation level: (502.5) m

9-Max capacity: (1,083,023) m^3

10-Dead storage : (273849) m^3

11-Reservoir area: (0.120783) Km^2

12-Catchment area : (20) km^2

13-Spillway : (65) m^3/Sec

14- Discharge during floods: (65) m^3/Sec

3.10. GEOMETRY OF SHAUGER EARTH DAM PROFILE

The general geometry and the foundation layers of the Shauger earth dam profile is shown in figure 3.1. It should be noted that this profile is not to scale.

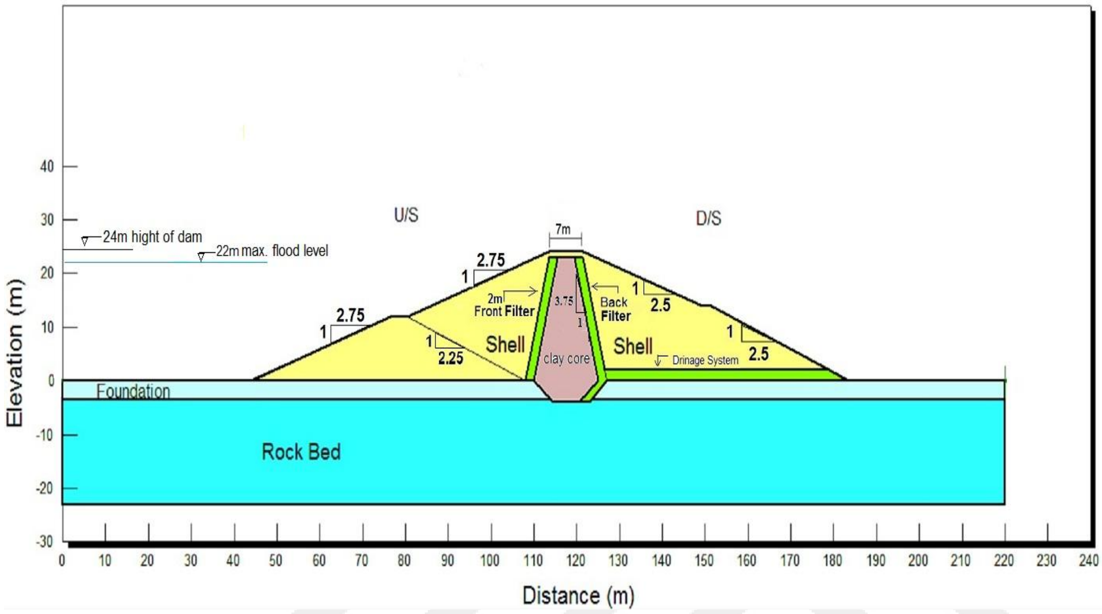

Figure 3.1. Geometry of the cross section of the Shauger Dam.

CHAPTER FOUR

METHODOLOGY

4.1 INTRODUCTION

This chapter presents the details of the numerical analysis and finite element analysis. It includes the description of the constitutive model, type of mesh, and conditions employed in the study by using Geostudio program to re-evaluate Shauger earth dam from geotechnical view.

Experience has important role in the design of earth dams. The detailed analysis is needed to understanding the sensitivity against material properties. To validate the experience and judgment of the designers comparisons of actual versus predicted failure modes of a dam should be done.

Evaluating the performance of earth dams is paramount to assuring their safety and continued operation. Thorough and timely judgment of engineers with the knowledge of case specific characteristics of the structure is necessary for an effective evaluation.It is strongly recommended to use computer software to analyze the stability of the slopes where the conditions of analyze are complex, especially where significant data are available. Software has ability to analyze a wide variety of load conditions and slope geometry [9] [25].

4.2 NUMERICAL ANALYSIS

The mathematical simulation of a real case is called numericalmodel. There are a wide difference between numericalmodeling and laboratory modeling (physical modeling). The numericalmodeling advantages over the laboratory modeling are:

- Can be constructed minutes, hours while laboratory modeling construction may take months.

- Numericalmodeling can be used to check wide different set of conditions, while laboratory modeling can be applied for a limit conditions.
- In laboratory models gravity cannot scaled, while there is no problem for accounting gravity in numericalmodels.
- Laboratory modeling usually deals with heavy equipments and there is a danger of personal harms, while there are no dangers with numericalmodeling.
- Laboratory models give results only at discrete instrumented points and external visual points, while numericalmodels can give results at any section inside or outside the model.
- Laboratory models has limitations in accommodate the boundary conditions, while numericalmodels can accommodate wide range of possible boundary conditions.

But it should be aware that numerical models has its limitations associated with changes in temperature, volume, chemical and seepage, it cannot include all these changes in one formula because mathematically this is very complex and cannot join all these changes mathematically in relationship, because of its complexity. May the engineers in the future overcome this problem [29].

4.3 FINITE ELEMENT METHOD

The finite element method was first introduced to geotechnical engineers in 1966 Berkeley conference on stability of slopes and embankments by Clough and Woodward (1967). Unlike the limit equilibrium method, the finite element method considers linear and non-linear stress – strain behavior of the soil in calculating the shear stress for the analysis. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to traditional limit equilibrium method [19] [25]. Finite element methods are well known for the estimating the realistic deformations of the slopes and changes in seepage amount and pore water pressure etc. in embankments due to various critical conditions. Some of the advantages of using a finite element analysis are,

1) The movement of the slopes at a particular location can be calculated. This helps in monitoring the movement of the slope. Also, soil stresses and pore water pressure responses to different external factors such as load, water level, reservoir level etc. can be calculated.

2) Stability of the slope during staged construction such as step by step excavation or construction of embankments, levees etc. can be calculated by performing incremental analysis.

3) Seepage amounts in various cases.

4) Pore water pressure, total head and pressure head amounts.

5) Earthquake forces, displacements and other effects.

A numerical method can be used to obtain an approximate solution when an analytical solution cannot be developed. Especially, the finite element method has been one of the major numerical solution techniques. One of the major advantages of the finite element method is that a general purpose computer program can be developed easily to analyze various kinds of problems. In particular, any complex shape of problem domain with prescribed conditions can be handled with ease using the finite element method [2].

The types of soil stress-strain relationships that can be used are linear elastic, elastoplastic, hyperbolic, Modified Cam Clay, elastoviscoplastic and multilinear elastic models. The selection of a particular stress-strain relationship depends on the state of the soil structure to be analyzed, its purpose of analysis and its laboratory and field properties available. The determination of soil properties in the field involves a large amount of uncertainty and so the application of finite element analyses imposes complexity on the stability problem [19] [24].

An approach to solve a slope stability problem by finite element method is to compute finite element stresses of the geotechnical structure and to implement them inside a limit equilibrium frame work to analyze its stability. It is known as finite element stress-based approach (SLOPE/W). So, the distribution of stresses in the ground is calculated by finite element analysis and then these stresses are used in a stability analysis. For the present case study involved in this thesis, this approach is followed. The software SIGMA/W is used for calculating the insitu stresses in the landslide and SLOPE/W is used for the slope stability analysis using the stresses calculated by SIGMA/W (SLOPE/W) [19]. QUAKE/W is used for calculating the earthquake stresses in the dam and it effect on the slope stability. SEEP/W is used for calculating the seepage amounts pore water pressure, total head, head pressure, etc in the Shauger dam.

4.4 FINITE ELEMENT MODEL

The cross section of the dam profile is shown in figure 3.1. Profile a section was chosen for finite element analyses (FEA) in Geostudio program. The mesh pattern used was Quad&Triangles elements quadrilateral four node and triangular three nodes with approximately element size 1m. Geostudio program automatically generates the mesh of geometry according to the given element size. The total mesh elements were 7127 elements and the total mesh nodes were 7305 nodes. The material models used for the analyses are available in the Geostudio program materials library, which were Mohr-Coulomb for stability analysis, Saturated only for steady state seepage analysis and Saturated/Unsaturated for transient drawdown analysis. The major difference between water flow in saturated and unsaturated soil are that the coefficient of permeability is not constant but is a function of degree of saturation or metric suction in unsaturated soils and the volumetric water content of unsaturated soil can be vary with time [18]. The basic data (c, φ, γ and k) for all materials entered in the model and material well defined. Besides, boundary condition for each case defined, to use and simulate multiple scenarios for analysis, by changing the boundary conditions and material properties for the same geometry for the numerical model.

Then the model has been implemented numerically in Geostudio for multiple cases and results reported out. The analysis tree for Shauger earth dam is shown in figure 4.1.

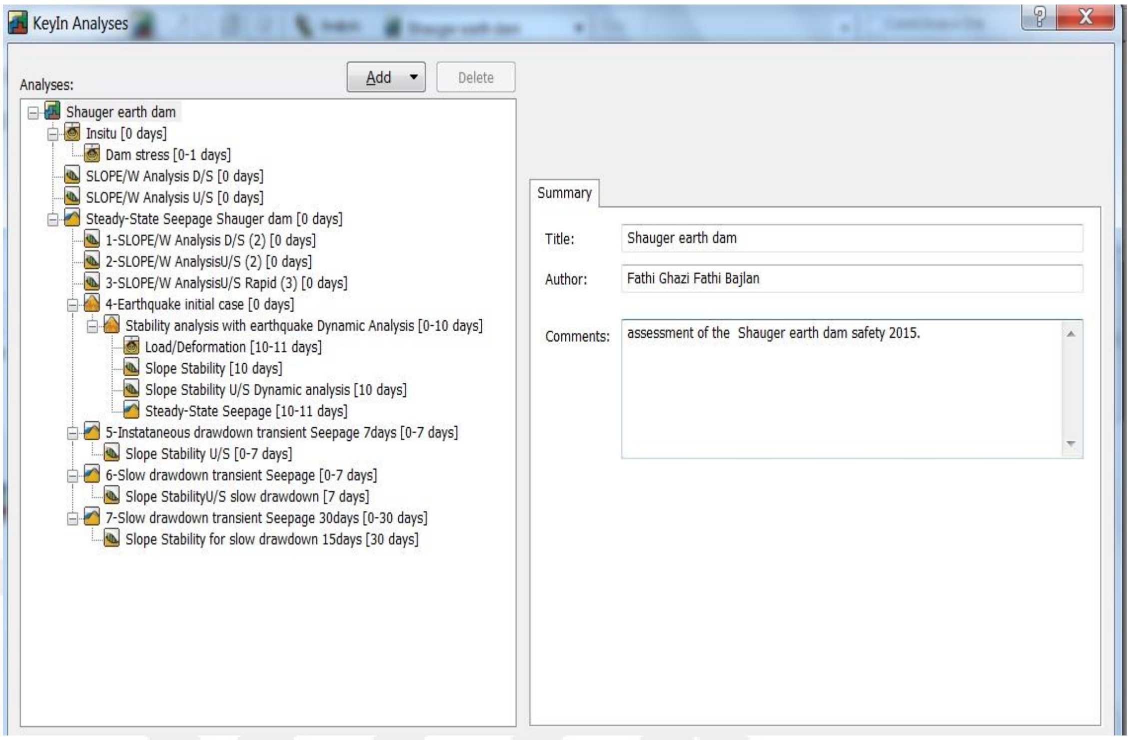

Figure 4.1. Analysis tree for Shauger earth dam.

More details can be found in the appendix A in the report of Shauger earth dam analysis.

4.5 CRITICAL LOADING CONDITIONS

The Shauger earth Dam and its foundation was analysed against failure. Consideration of loading conditions which may result to instability must be made for all likely combinations of reservoir levels after construction. Four construction and / or loading conditions were examined in particular [11] [25]:

a. At the end of construction condition

The critical condition to be analyzed is at the completion of earth dam construction but prior to filling with water. In this case the water table present at the river bed level.

b. At the full reservoir condition

For Shauger zoned type dam when the reservoir is full of water and some steady seepage into the embankment is established, the conditions to be considered for the steady state seepage analysis should be steady state seepage pore pressures which are fully developed as a result of the reservoir have being storing water over a long period of time. In this case there is a Phreatic Surface Line under steady seepage state.

c. The Rapid Drawdown Condition in the reservoir

Rapid drawdown in reservoir water level may cause the upstream face stability to become critical mainly due to the removal of the supporting water. When the reservoir is rapidly evacuated and drawn down, pore water pressures in the dam body are reduced in two ways. There is a slower dissipation of pore pressure due to drainage and there is an immediate elastic effect due to the removal of the total or partial water load. The exact mechanism of this phenomenon is as follows: It is assumed that the reservoir has been maintained at a high level for a sufficiently long time so that the fill material of the dam is fully saturated and steady seepage established. If the reservoir is drawn down at this stage, the direction of flow is reversed, causing instability in the upstream slope of the earth dam. The “instantaneous” drawdown is a hypothetical condition that is assumed. The most critical condition of sudden drawdown means that while the water pressure acting on the upstream slope at “full reservoir” condition is removed, there is no appreciable change in the water content of the saturated soil within the embankment. The saturated weight of the slope produces the shearing stresses while the shearing resistance is decreased considerably because of the development of the pore water pressures which do not dissipate rapidly [10]. Therefore, it was considered very important such an analysis to be carried out and included in this research.

A more accurate way of analyzing drawdown is to use the seepage analysis results. This more advanced approach uses the exact pore-water pressures that were in the soil during the transient drawdown process. Using the pore-water pressure estimated from seepage analysis, the factor of safety of the earth dam at different times during drawdown process can be evaluated [26].

d. Earthquake condition

Earthquake may cause many changes in the internal and external conditions, usually earthquake shaking apply additional lateral quake force. The response and behavior of earth dams subjected to earthquake is highly complex and multifaceted. Generally, there are the issues of: the motion, movement and inertial forces that occur during the earthquake,the generation of excess pore-water pressures, reduction of shear strength, the inertial forces, excess pore-water pressures, and the redistribution of excess pore-water pressures after the shaking has stopped [11][25]**.** Some of these issues performed on the Shauger earth dam in this thesis.

4.6 SOFTWARE USED IN THE SHAUGER EARTH DAM ANALYSIS

In this thesis, GEOSTUDIO 2012 version is used for the purpose of two dimensional finite element analysis. In the GEOSTUDIO, SLOPE/W was used to perform the limit equilibrium analysis, SIGMA/W was used to perform insitu stress and deformation analysis, SEEP/W was used to perform seepage analysis and QUAKE/W was used to perform the earth quake analysis.

4.6.1 SIGMA/W

SIGMA/W is a finite element software product mainly used for determination of stress-deformations in earth structures. In addition, it is also used to model soil-structure interaction, staged constructions, consolidation analyses etc. SIGMA/W is a component of GEOSTUDIO 2012. It was designed in a way to analyze both simple and complex problems because of its available options for different analyses. Performing both simple linear elastic analysis and complex non-linear plastic analysis is possible using this software. Mesh generation for finite elements is an automated process in SIGMA/W. A choice for the shape of mesh elements – Quadrilateral, triangular rectangular and mixed shapes are also available. Density of the mesh can be varied as required. Boundary conditions must also be defined for a SIGMA/W model. The available boundary conditions are Fixed X, Fixed Y and Fixed X/Y. Generally, for in calculating stresses for a slope stability problem the boundary for the base is selected as Fixed X/Y, as the displacement in both X and Y directions is not allowed at the base. The significance of

SIGMA/W in slope stability analysis is that it can model the stress deformations along with the pore water pressures that arise due to stress change. Also, it readily provides a graphical representation of variation of the stress strain values with variation in loading, time, ground water level etc. In this thesis, for a finite element analysis of the Shauger earth dam this program is coupled with SLOPE/W to obtain the safety factor calculated considering the stress – strain relationship of each soil layer.

Illustrative examples are included in the SIGMA/W manual (2012) for verification of its formulation. These examples show that the values calculated using SIGMA/W program match with the values obtained from hand calculations and published values [19].

4.6.2 SEEPAGE MODELING IN SEEP/W

Earth dams have always been associated with seepage as they impound water in it. The water seeks paths of least resistance through the dam and its foundation. Seepage will become a problem only if it carries dam materials also along with it. Seepage must be controlled to prevent the erosion of embankment or its foundation. One of the important factors causing failure of embankment dam is seepage and hence seepage analysis of embankment dam is of greater importance. Dams must be designed and maintained to safely control seepage. Excessive seepage leads to dam safety issues, if not treated carefully. Seepage analyses usually needed for many reasons [2]:

- To find and estimate the phreatic line within dam.
- To find pore pressures within and beneaththe dam.
- To find uplift pressures at the toe of thedam.
- To estimate the seepage amount passes through the dam.
- To evaluate the efficiency of drainage systems.

All these are factors taken into consideration while suggesting remedial measures in existing dams.

SEEP/W is numerical modeling software which used to solve the practical seepage problems. This is a part of the most popular geotechnical software called GeoStudio. The SEEP/W program is created with the combination of seepage theory and finite element method and working on saturated/unsaturated soil region. The practical seepage problems are never easy to convert into a numerical modeling because of the heterogeneity of the natural soils and the varying boundary condition. Generally the boundary conditions for a seepage problem never being as same as found in the initial stage. Therefore the seepage analysis in SEEP/Wprogram is divided into two categories:

a. Steady - state analysis

In the steady state the fundamental water flow properties such as water pressure and water flow rates never going to be changed and it assumed that water flow occur in saturated zones of soil only. Practically achieving steady state is impossible. The purpose of the steady-state analysis is only to know how the initial input parameters respond to a given boundary condition. This analysis never state that how long it takes to reach a steady state. It returns a set of solved values for water pressures and water flow parameters for particular boundary conditions. A constant pressure (H) and a constant flux rate are the important boundary conditions used for a steady-state analysis.

b. Transient analysis

Transient analysis is used to know how long the embankment takes to responds for a given boundary condition. Therefore the fundamental flow properties (pressures and water flow rate) will vary with time. It assumed that water flow occur in saturated and unsaturated zones of soil only. The analysis required an initial boundary condition as well as a destination boundary condition [14].

To check the safety of dams against **piping**, it can occur when hydraulic gradients at the downstream end of a dam are large enough to move soil particles.When upward seepage occurs and the hydraulic gradient is equal to one, piping originates in the soil mass [25] [8]:

$$i_{cr} = \frac{\gamma'}{\gamma_w} \quad \text{...} 4.1$$

$$\gamma' = \gamma_{sat} - \gamma_w = \frac{G_s\gamma_w + e\gamma_w}{1+e} - \gamma_w = \frac{(G_s-1)\gamma_w}{1+e} \quad \text{...} 4.2$$

$$i_\alpha = \frac{\gamma'}{\gamma_w} = \frac{G_s-1}{1+e} \quad \text{...} 4.3$$

Where i_{cr} is critical gradient, γ' is the submerged soil density, γ_w water density, γ_{sat} is saturated soil density, G_s specific gravity of soil and e is the void ratio of soil.

The factor of safety against piping, F_s, can be defined as:

$$F_s = \frac{i_\alpha}{i_{exit}}$$

Where: i_{exit} is the maximum exit gradient. The maximum exit can be determined from the flow net. The maximum exit gradient can be given by $\Delta h/l$ where Δh is the head lost between the last two equipotential lines, and *l* is the length of the flow element.

The SEEP/Wprogram has ability to read the initial condition from another analysis (may be SEEP/Wor SIGMA/W) and generally obtained from a steady-state analysis [5].

4.6.3 QUAKE/W

QUAKE/W iscomputer software used for analysis of earth dams subjected to earthquake; this software uses finite element technique. QUAKE/W is part of GeoStudio and is, consequently, fully integrated with the other components such SLOPE/W, SEEP/W, SIGMA/W for example. The integration of QUAKE/W and other products within GeoStudio greatly expands the type and range of problems that can be analyzed beyond what can be done with other geotechnical dynamic analysis software. QUAKE/W can be used as one of its main attractions is the integration with the other GeoStudio products

The response of the earth dams in case of earthquake is very complex. Generally, shaking produces movements, motion and forces, excess pore pressure, losses in shear strength, and redistribution ofexcess pore pressure.

Subtracting the QUAKE/W computed stresses from the initial static stresses gives the additional shear stresses arising from the inertial forces. This information together with the Newmark Sliding Block concepts can be used to estimate the permanent deformation. In GeoStudio, SLOPE/W uses the QUAKE/W results to perform these calculations [11][25].

Liquefaction stability (the phenomenon of soil liquefaction means Reduction shear strength and stiffness due to increase in pore pressure is the main reason of earthquake damage to dams and slopes), in another words the term liquefaction is used to describe large deformations occurring in a soil mass caused by monotonic (cyclic), transient disturbance of saturated cohesionless soil. Generation of excess pore pressure under undrained loading is the main ingredient for liquefaction. Engineers of earthquake have a line of thinking that says the factor of safety should not be below 1.0 in any case even under the worst conditions. If the factor of safety is below 1.0 in worst conditions then the risk is high for catastrophic liquefaction failure [27].

The seismic demand placed on a soil layer (expressed in term of cyclic stress ratio CSR), the capacity of soil to resist liquefaction (presented in term of cyclic resistance ratio CRR). The calculation of CSR originally defined by Seed and Idriss [15] is given by the following equation:

$$\mathrm{CSR} = 0.65 \times \frac{\sigma_{\mathrm{v}}}{\sigma'_{\mathrm{v}}} \times \frac{a_{\max}}{g} \times r_{\mathrm{d}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.4$$

where, σ_v and σ_v are total and effective vertical overburden stresses respectively, a_{max} is the peak horizontal acceleration at ground surface generated by the earthquake, g is the acceleration of gravity and r_d is a stress reduction coefficient.

Generally, this analysis is applied for case at end of shaking to find the stress and pore pressure. The analysis can be applied for any time step.

4.6.4 SLOPE/W

SLOPE/W is a powerful commercially available tool for analyzing the stability of the slope. It is a component in the entire tool kit of GEOSTUDIO 2012. This software works on a limit equilibrium framework and includes methods such as the Morgenstern-Price, Spencer's method, Bishop method, Janbu method and Ordinary method slices etc. A finite element stress-based can also be solved using this software.

Civil engineer often are expected to make calculations to check the safety dam slopes, this check involves determining the shear stress developed along the most likely rupture surface (mobilized shear stress) and comparing it with shear strength of the soil. This process is called slope stability analysis. The most likely rupture surface is the critical surface that has the minimum factor of safety [8]. Factor of Safety (FoS) for different shapes of slip surfaces circular, noncircular and wedged surfaces can be determined.

Where the $$\mathrm{FoS} = \frac{\text{Shear strength of soil}}{\text{mobilized shear stress}}$$

The soil models such as Mohr-Coulomb model, anisotropic strength model, SHANSEP model, bilinear models etc. are available for modeling the material properties of the different layers of the soil. User can also easily define a particular slip surface in the slope. This feature is mainly advantageous in post slope failure analyses. The main advantage of this program is its ability to be coupled with other programs such as SIGMA/W, SEEP/W etc. By integrating these programs for a particular analysis a slope can be analyzed considering the aspects of the insitu stress – strain behavior, earthquake acceleration factors, seepage factors etc. Stability of reinforced slopes can be assessed using this software. Different types of reinforcements – anchors, geo-fabrics, soil nails, piles, sheet-piles etc. can be designed. Pseudostatic analysis and analysis for liquefaction stability can also be performed (SLOPE/W).

Illustrative examples are provided in the SLOPE/W manual (2012) for verification of the analyses using SLOPE/W program. These examples show a detailed comparison of the analysis results from SLOPE/W with solutions obtained from the

Stability charts developed by Bishop and Morgenstern in 1960's, a comparison with published results and a comparison with theoretical calculations of earth pressures. The analyses results from SLOPE/W prove to be the same as the values obtained from the other methods, indicating that the results obtained using SLOPE/W program are reliable [19].

There are many material strength representation models in the SLOPE/W, ***Mohr-Coulomb Model*** is used in this thesis and here are some details about this model,

Coulomb's equation is the most common soil shear strength which is:

$\tau = c + \sigma n \tan\varphi$ ………………………………………………..................................4.5

where:τ = shear strength (shear at failure),

c = soil cohesion,

σn= normal stress on shear plane, and

φ = angle of internal friction (phi).

This equation gives a line on shear strength vesus normal stress plot. (Figure4.2). Intercept of the line with shear axis gives the cohesion value, where the slope of this line gives the (φ).

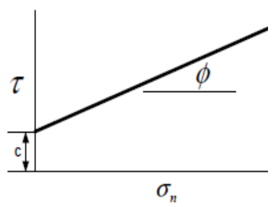

Figure 4.2. Graphical representation of Coulomb equation.

The envelope is usually determined from triaxial tests and halfMohr circles are presented the results, seefigure 4.3.

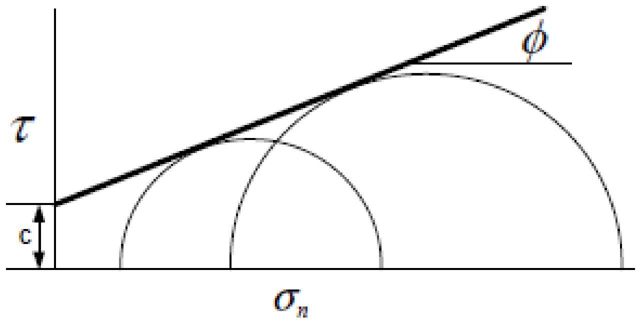

Figure 4.3. Mohr-Coulomb failure envelope.

Figure 4.4. shows failure envelope for undrained case when φ is zero, i.e. the soil strength will be equal to c.

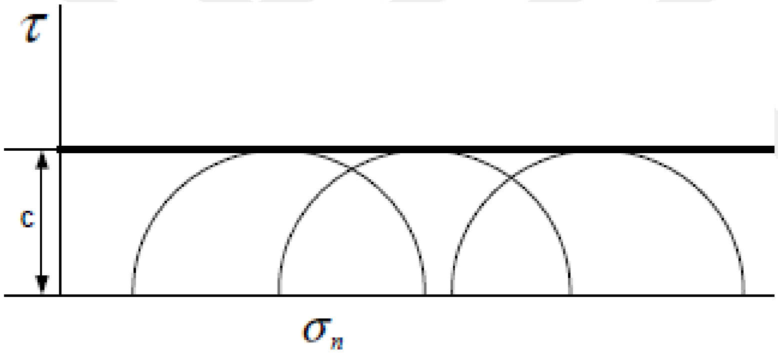

Figure 4.4. Undrained strength envelope.

Strength parameters *c* and φ can be effective or total strength parameters in SLOPE/W.

For staged rapid drawdown analysis, itshould use the Mohr-Coulomb model in SLOPE/W, and use effective parameters*c* and φ , and total *c* and total φ for representing the undrained case. For case of free drained a zero total *c* and φ in a Staged rapid drawdown analysis [27].

There are many methods for making the **stability analysis**, and the following are the methods which used in this thesis:

- **Bishop's modified method**

This method is suggested by Professor Bishop in 1950‟s. The method is a modified version of the ordinary method of slice and the normal forces between interslices are included. But Bishop did not include the shear forces between the slices and developed a new equation for the factor of safety. The new equation was a non-linear equation because, the normal force between two slices has obtained using the factor of safety hence, the equation contains the variable factor of safety in both side. Therefore an iterative method is compulsory to solve the equation [5] [25]. The final equation which Bishop derived is,

$$F=\frac{1}{\sum_{i=1}^{n} Wi.sin\propto}.\sum_{i=1}^{n}\left(\frac{ci.li+\left(Wi.cos\propto\frac{ci.li}{F}.sin\propto\right).tan\varphi}{m\propto}\right)\ldots\ldots\ldots\ldots 4.6$$

In the equation Bishop has included a new tem $m\alpha$ and defined as;

$$m\alpha=\cos\alpha+\frac{\sin\alpha.\tan\varphi}{F}\ldots\ldots\ldots\ldots 4.7$$

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

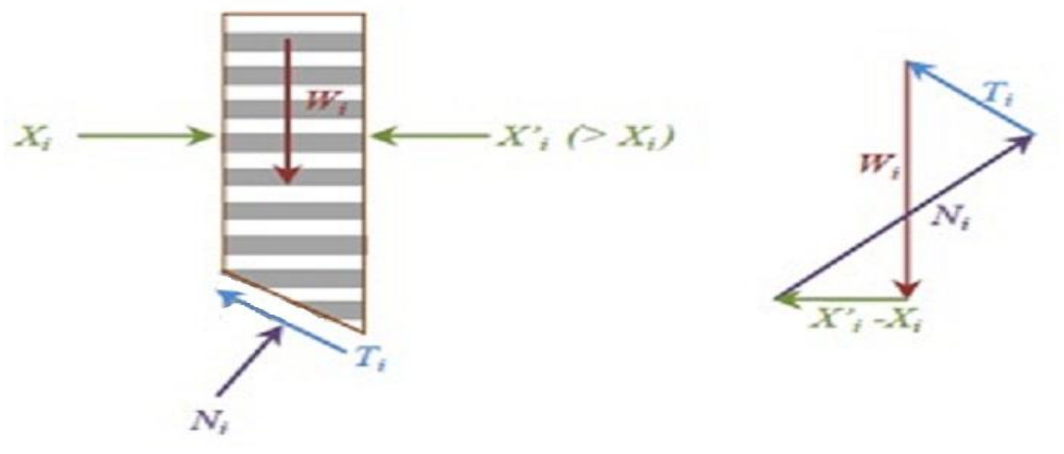

Figure 4.5. Free body diagram and force polygon for Bishop's method[2].

Figure 4.5 shows the free body diagram and the force polygon for the Bishop'smodified method. Compared with the ordinary method of slice, the horizontal force (*Xi,X'i*) exerted by the adjacent slices are additionally included in both free body diagram and the force polygon. In the force polygon, the resultant horizontal force (*Xi-X'i*) is placed with the direction.

- **The Ordinary Method of Slices**

The Ordinary Method of Slices (OMS) was developed by Fellenius (1936). The forces on the sides of the slice are neglected (see figure 4.6). To find the normal force it should take summation of forces perpendicular to the bottom of slice. Then, to compute the factor of safety it should take the moment about the circle center. The factor of safety for figure 4.7 is calculated by using the equation 4.8[25].

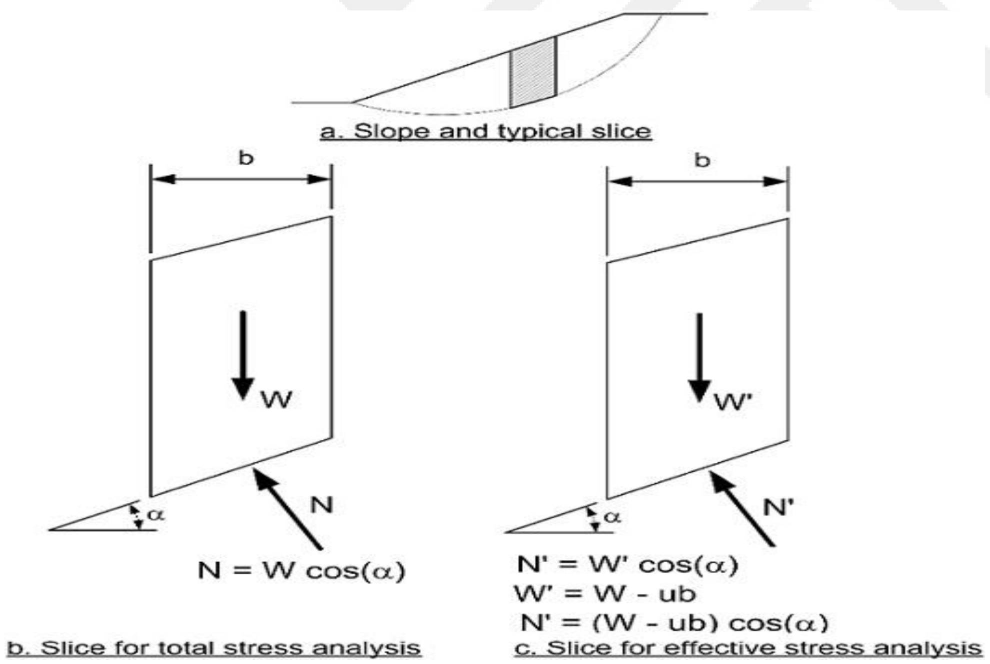

Figure4.6. Typical slice and forces for Ordinary Method of Slices [25].

$$\mathrm{F} = \frac{\sum[c^{-}\Delta l + (W cos\alpha - u\Delta l cos\, \cos^2 \alpha) tan\emptyset']}{\sum Wsin\alpha} \quad \ldots\ldots 4.8$$

Where:

c' and Ø' = shear strength parameters for the center of the base of the slice

W = weight of the slice

α = inclination of the bottom of the slice

u = pore pressure at the center of the base of the slice

Δl = length of the bottom of the slice

In the case where water loads act on the top of the slice, the expression for the factor of safety (Equation 4.9) must be modified to the following after some simplification:

$$F = \frac{\sum\left\{c'\Delta\ell + \left[W\cos\alpha + P\cos(\alpha-\beta) - u\Delta\ell\cos^2\alpha\right]\tan\phi'\right\}}{\sum W\sin\alpha - \frac{\sum M_P}{R}} \qquad \ldots\ldots\ldots 4.9$$

Where:

P = water force acting on to the top of the slice

β = inclination of the top of the slice

MP = moment about the center of the circle produced by the water force acting on the top of the slice

R = radius of the circle

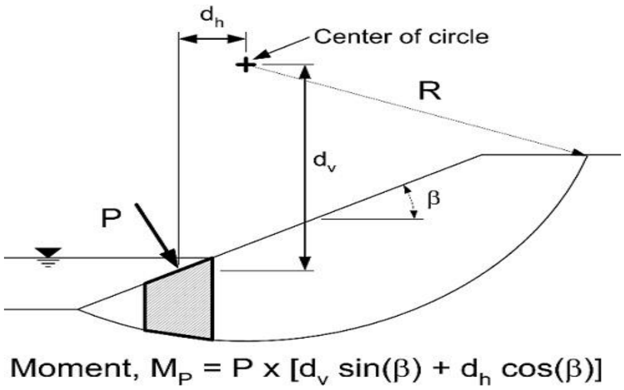

Figure 4.7. Slice for Ordinary Method of Slices with external water loads[25].

- **Morgenstern and Price's Method**

The Morgenstern-Price method was developed by Morgenstern and Price (1965), which considers not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular and non-circular slip surfaces. In this method, a simplifying assumption is made regarding the relationship between the interslice shear forces (X) and the interslice normal forces (E) as:

$$X=\lambda \cdot f(x) \cdot E \qquad \text{……………………………………………… 4.10}$$

Where, f(x) is an assumed function that varies continuously across the slip, and (λ) is an unknown scaling factor that is solved for as part of the unknowns. The unknowns that are solved for in the Morgenstern and Price method are the factor of safety (FOS), the scaling factor (λ), the normal forces on the base of the slice (P), the horizontal interslice force (E), and the location of the interslice forces (line of thrust). Once the above unknowns are calculated using the equilibrium equations, the vertical component of the resultant force on the interslice forces (X) is calculated from equation 4.11.

$$P=\frac{\left[w-(X_R-X_L)-\frac{1}{f}\left(c^{-}l\sin\alpha-ul\tan\phi^{-}\sin\alpha\right)\right]}{m_\alpha} \qquad \text{……………………………………………4.11}$$

where:

P: Normal force on the base of the slice.

W: Weight of slice.

XR : Vertical component of the resultant force on the interslice (from right side of slice).

XL : Vertical component of the resultant force on the interslice (from left side of slice).

Two factor of safety equations are computed, one with respect to moment equilibrium (Fm) and the other with respect to force equilibrium (Ff). The moment equilibrium equation is taken with respect to a common point as:

$$F_m = \frac{\sum[c^{-}l + (P - ul)\tan\phi^{-}]R}{\sum(Wd - P_f)} \quad \text{.........4.12}$$

Forcircular slip surfaces: $f = 0$, d = Rsinα and R = constant;

$$\mathrm{Fm} = \frac{\sum[c^{-}l + (P - ul)tan\emptyset^{-}]}{\sum Wsin\alpha} \quad \text{.........4.13}$$

The factor of safety with respect to force equilibrium is:

$$F_f = \frac{\sum(c^{-}l + (P - ul)\tan\phi^{-})\cos\alpha}{\sum P\sin\alpha} \quad \text{.........4.14}$$

On the first iteration, the vertical shear forces (XR and XL) are set to zero. On subsequent iterations, the horizontal interslice forces are first computed from:

$$(ER - EL) = P\sin\alpha - \frac{1}{f}\left[c^{-}l + (P - ul)\tan\phi^{-}\right]\cos\alpha \quad \text{.........4.15}$$

where:

ER: Horizontal interslice force (from right side of slice)

EL: Horizontal interslice force (from left side of slice)

l: The inclined width of slice

u: Pore water pressure

P: Normal force on the base of the slice

This method of slope stability analysis, which is valid for slip surfaces of any arbitrary shape, is considered as the more general rigorous method. It stems its generality from the fact that no stringent restriction is imposed neither on the direction or location of the interslice forces nor on the shape of the slip surface analyzed [17].

- **Janbu's simplified method**

The Janbu's Simplified method is similar to the Bishop's Simplified method except that the Janbu's Simplified method satisfies only overall horizontal force equilibrium, but not overall moment equilibrium. Figure 4.8 shows the free body diagrams and force polygons of the Janbu's Simplified method.

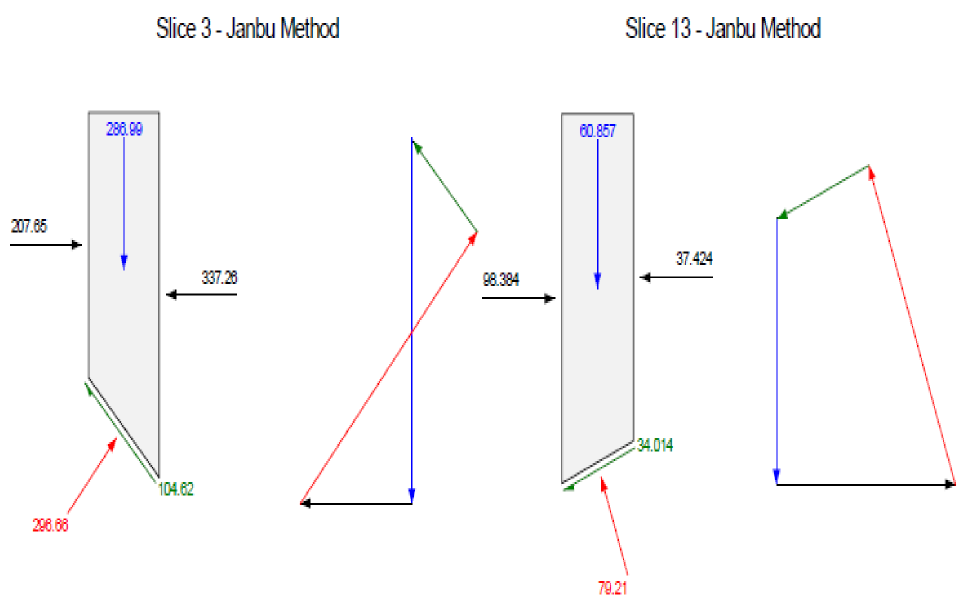

Figure 4.8. Free body diagram and force polygon for the Janbu method[27].

CHAPTER FIVE

RESULTS AND DISCUSSION

5.1 INTRODUCTION

This chapter presents the results and discussion of the insitu stresses, slope, earthquake and seepage analysis of Shauger earth dam. The results are shown in texts, detailed dam cross section figures and graphs, extended detailes for the results are available in the appendix A.

The analyses were performed using two dimension finite element methods. The software used were SLOPE/W, SEEP/W, SIGMA/W and QWAKE/W from GEOSTUDIO (2012) program.

TheShaugerearth dam level at end of the construction was reported to be 24 meter height its crest crown is designed to be at 506.2 m.a.s.l level. The ground water level has been varied proportionally with the reservoir level, but the initial ground water level used as same level of the river bed.

5.2 ANALYSIS OF SHAUGER EARTH DAM

For purposes of verification of the global stability Analysis of the Shauger earth dam, for the loading cases (a, b, c, d) analysis method which mentioned in the previous chapter, based on the Finite Element Analysis technique (FEA) was executed. The most cases analyzed and the results are shown as follows:

5.2.1 INSITU ANALYSIS FOR SHAUGER DAM

Insitu analysis conducted on the Shauger earth dam foundation before dam construction. The stresses results were as shown in the figure 5.1 and the insitu Pore Water Pressure (PWP) as shown in figure 5.2.

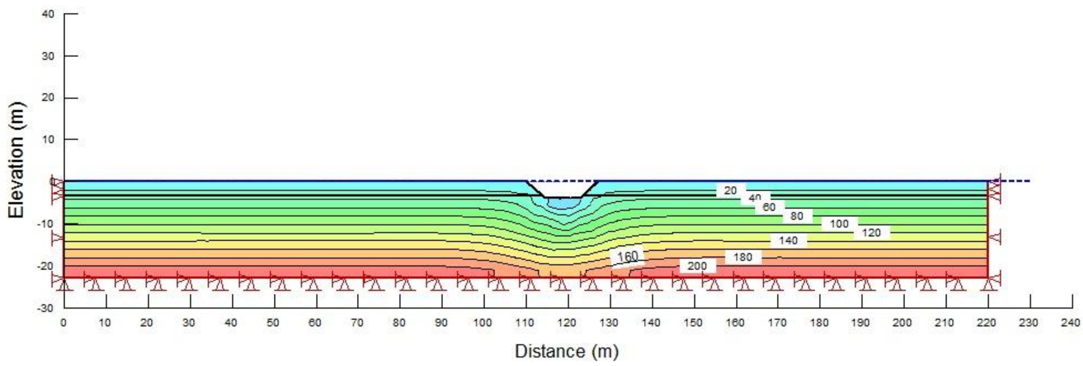

Figure 5.1. Insitu vertical effective stress for Shauger dam.

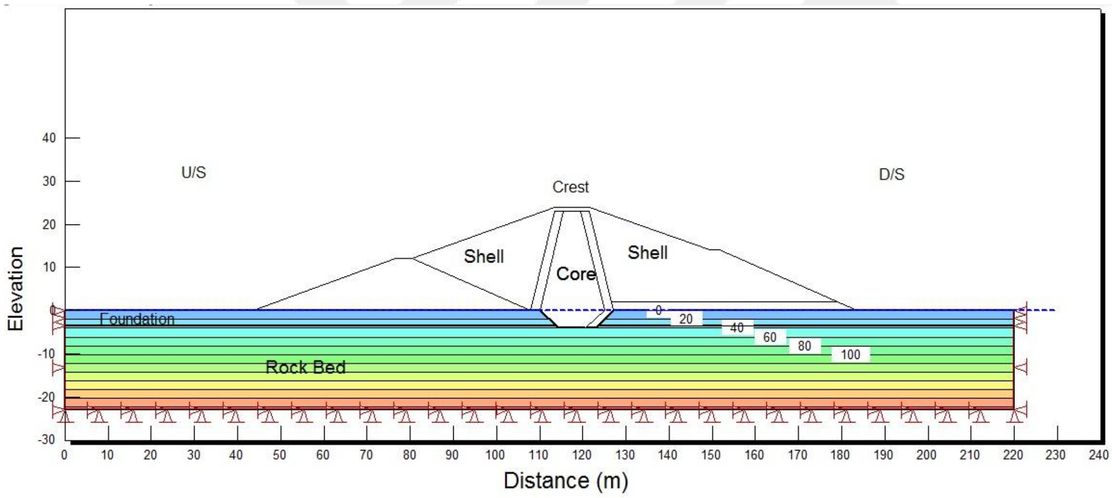

Figure 5.2. Insitu pore water pressure for Shauger dam.

5.2.2 SHAUGER DAM STRESSES (LOAD/DEFORMATIN) ANALYSIS

Load/Deformation analysis conductedon the Shauger earth dam after dam construction completed, the initial water table assumed to be at the level of river bed. The deformation results were as shown in the figure 5.3 and the deformation direction shown in figure 5.4. While the displacements graph shown in figure 5.5 shows that there is a heave in heel and toe of the dam, while settlement occurred beneath the center of the dam. The maximum settlement was at distance between 110-125m and its value was

-0.49m, while the maximum heave occurred at distance about 40m and 180m and its value was about +0.27m.

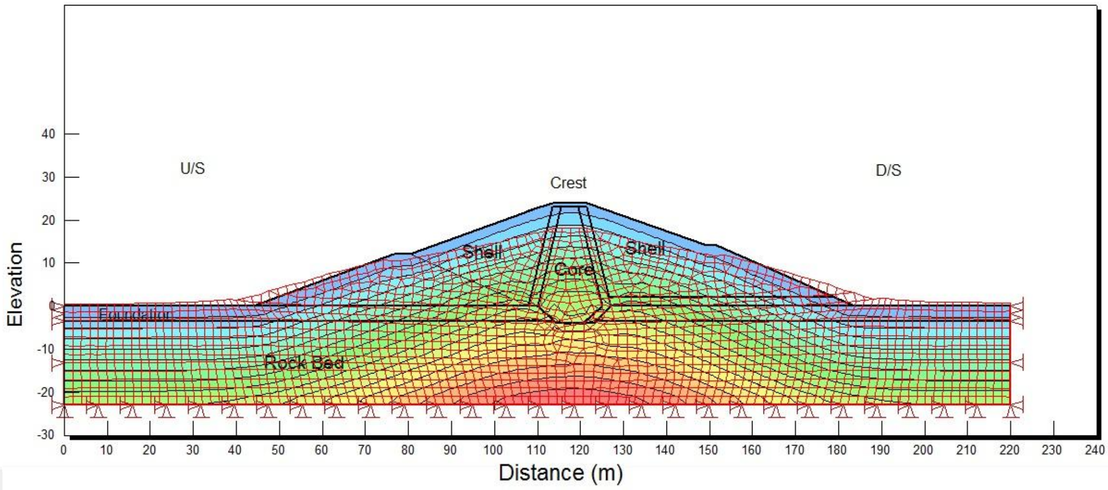

Figure 5.3. Deformation for Shauger dam (magnified).

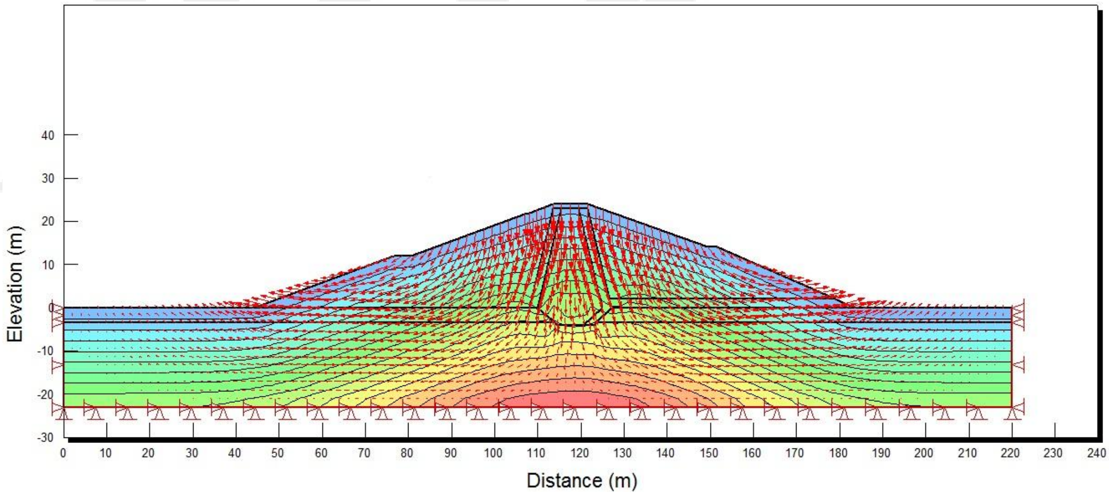

Figure 5.4. deformation direction for Shauger dam (magnified).

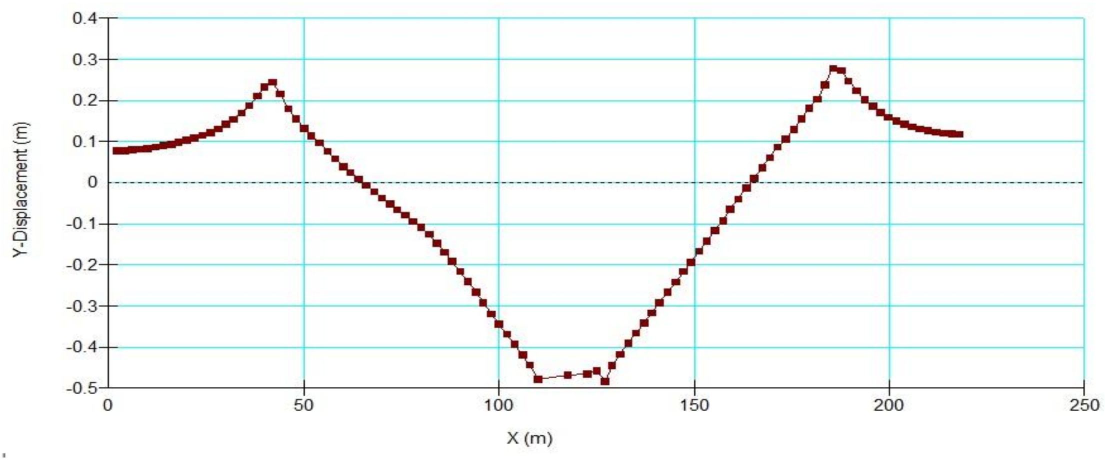

Figure 5.5. displacements graph for Shauger dam.

5.2.3 SLOPE STABILITY ANALYSIS AT THE END OF CONSTRUCTION

Slope stability analysis at the end of construction conducted by using many methods (Bishop, Ordinary, Janbu and Morgenstern – Price) for both D/S and U/S, the initial water table assumed to be at the level of river bed 0m.Table 5.1 summarizes the FoS results for D/S and U/S using many methods.The results are as shown in figures below, where figure 5.6 shows FoS for D/S slope stability analysis by using Bishop method for Shauger dam and figure 5.7 shows FoS for U/S slope stability analysis.

Table 5.1. FoS results at end of construction using many methods.

	Bishop	Ordinary	Janbu	Morgenstern – Price
D/S	2.327	2.280	2.273	2.323
U/S	2.709	2.644	2.639	2.708

Table 5.1 shows that Janbu method gives fewest values for FoS than the others, and that the D/S had the less value for FoS which was 2.273, that is more than the required value from the USDA-NRCS (U.S. Department of Agriculture-Natural Resource Conservation service) (210-VI-TR60, July 2005) specification which is 1.4 and from U.S. Army Corps Of Engineers (USACE)Engineering And DesignSlope Stability EM 1110-2-190231 Oct 2003 [25] which is 1.3 for this case. So the dam is in safe side at the end of construction.

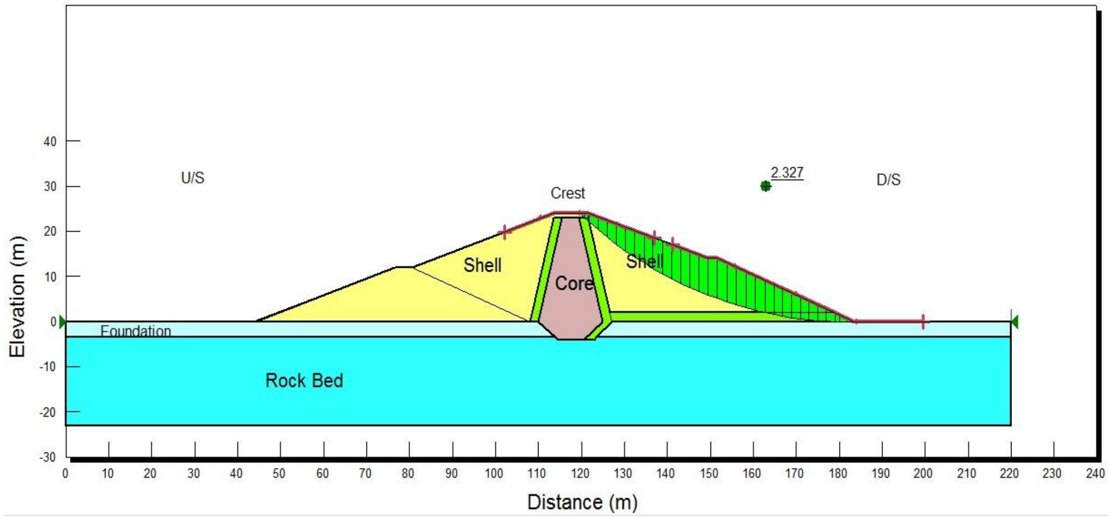

Figure 5.6. FoS for D/S Slope Stability analysis for Shauger dam at the end of construction.

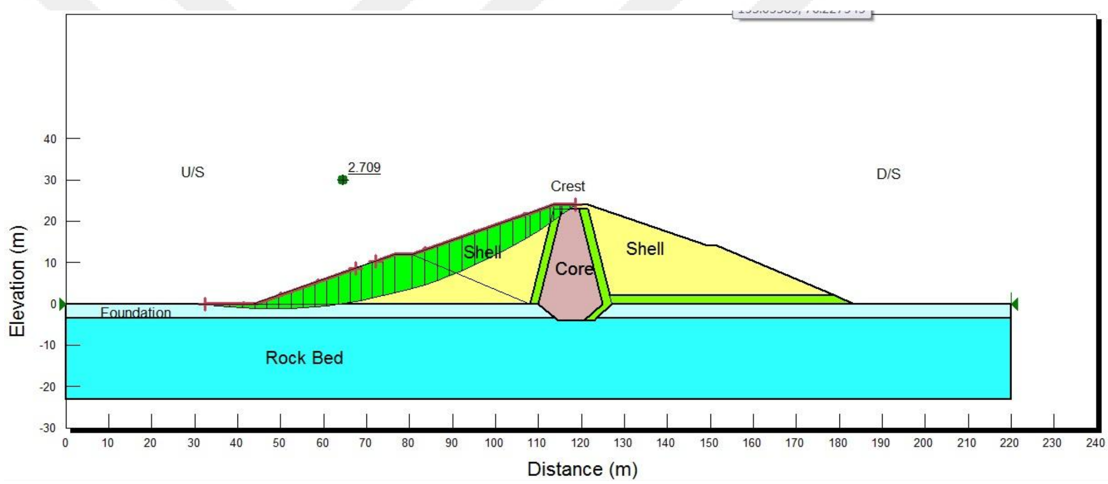

Figure 5.7. FoS for U/S Slope Stability analysis for Shauger dam at the end of construction.

5.2.4 STEADY-STATE SEEPAGE ANALYSIS

Steady state seepage for full reservoir without earthquake and with earthquake analysis conducted. Figure 5.8 shows the steady state seepage flow paths and flux section, and figure 5.9 shows total head in case of steady state seepage. When, figure 5.10 shows the results for dynamic steady state seepage analysis.

The flow paths in figure 5.8 show that there are some paths above the zero pressure line (phreatic line) i.e. in unsaturated zone; this case is normal and acceptable.

From flux sections the total seepage through the dam was:

For case of without earthquake=5.3311e-6 m^3/sec = 0.46060 m^3/day

For case of after earthquake=4.6185e-6 m^3/sec =0.3990 m^3/day

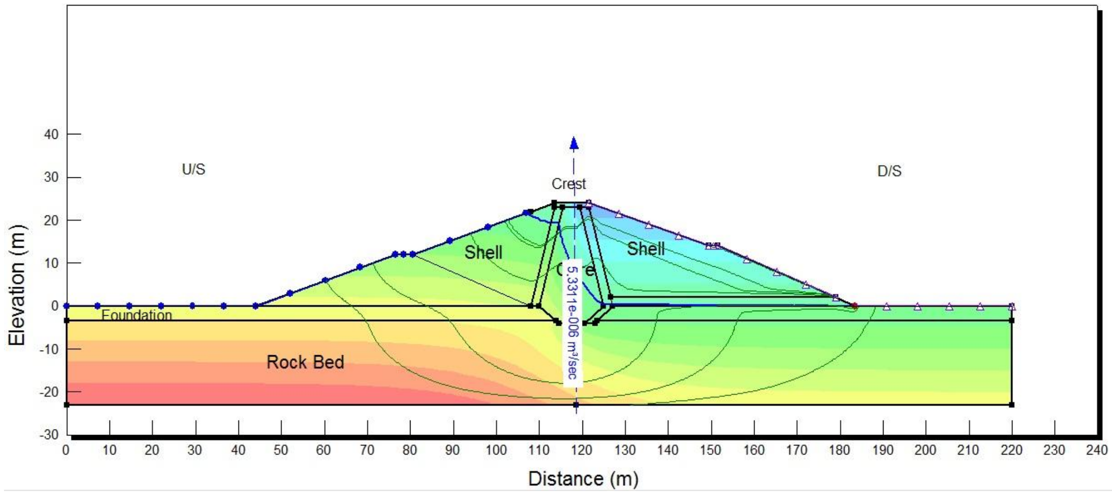

Figure 5.8. Flow paths and flux section in steady state seepage for Shauger dam.

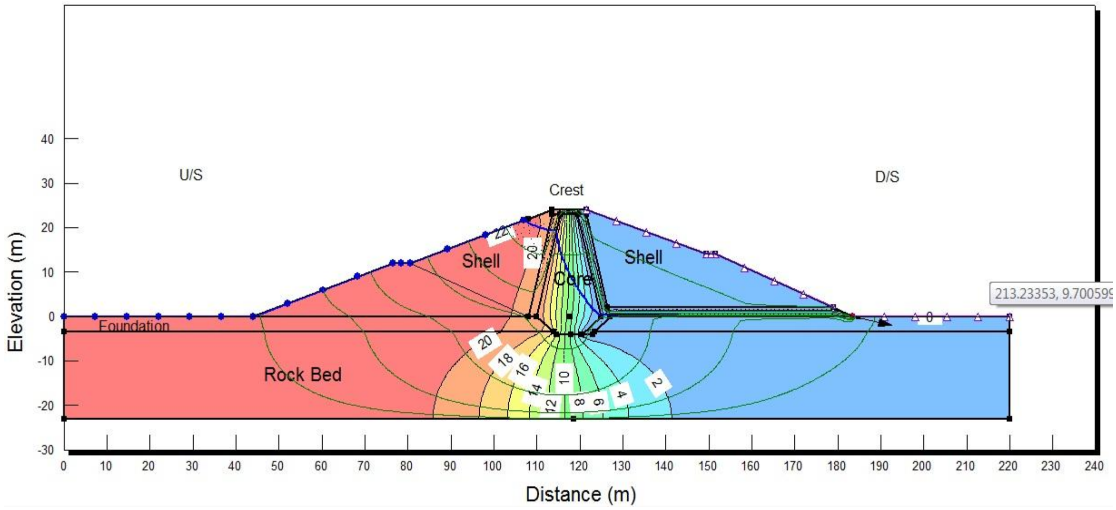

Figure 5.9. Total head in steady state seepage for Shauger dam.

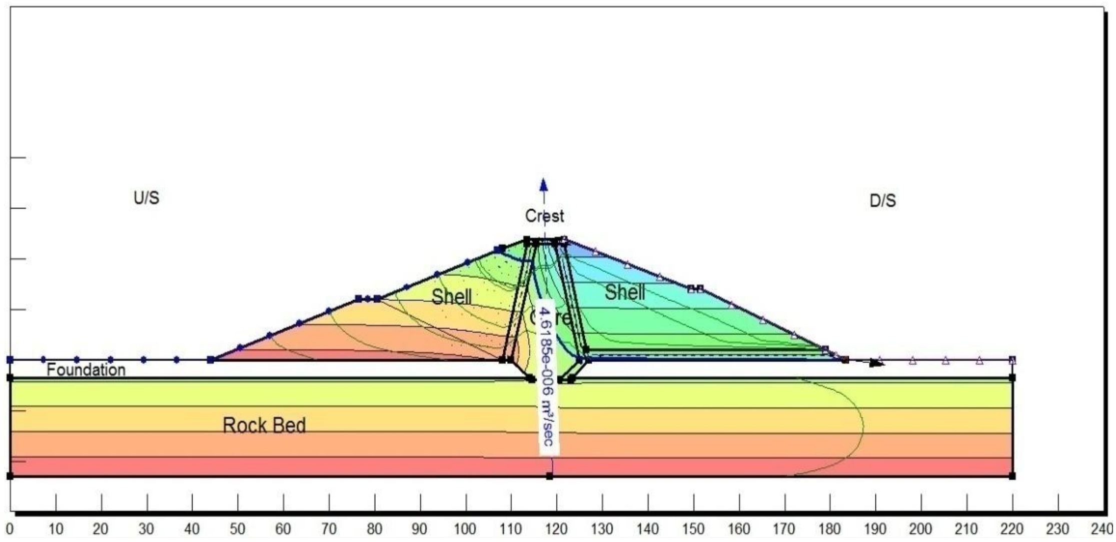

Figure 5.10. Steady state seepage with dynamic analysis for Shauger dam.

While figures 5.11, 5.12 and 5.13 shows total head graph, pressure head graph and pore water pressure graph respectively. The graphs show that there is a steep drop in pressure in range of distance between 110-125m, this drop is concordant with the phreatic line drop inside the core of dam.

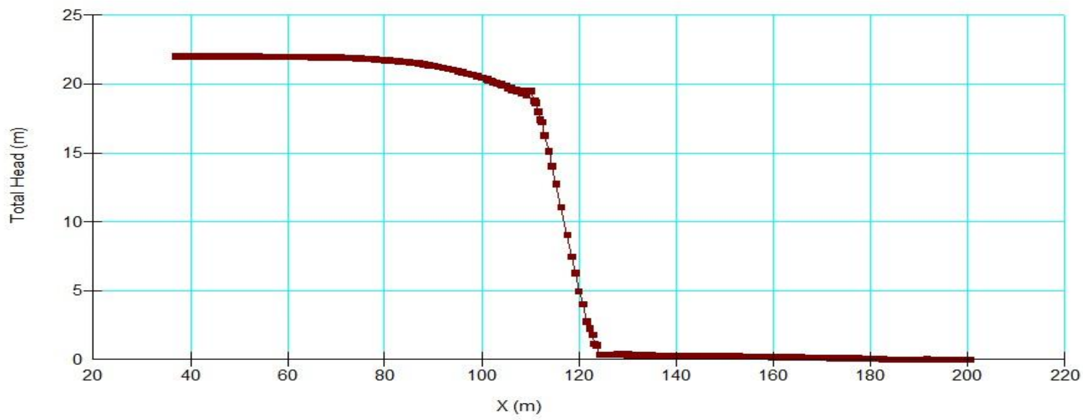

Figure 5.11. Total head graph in steady state seepage for Shauger dam.

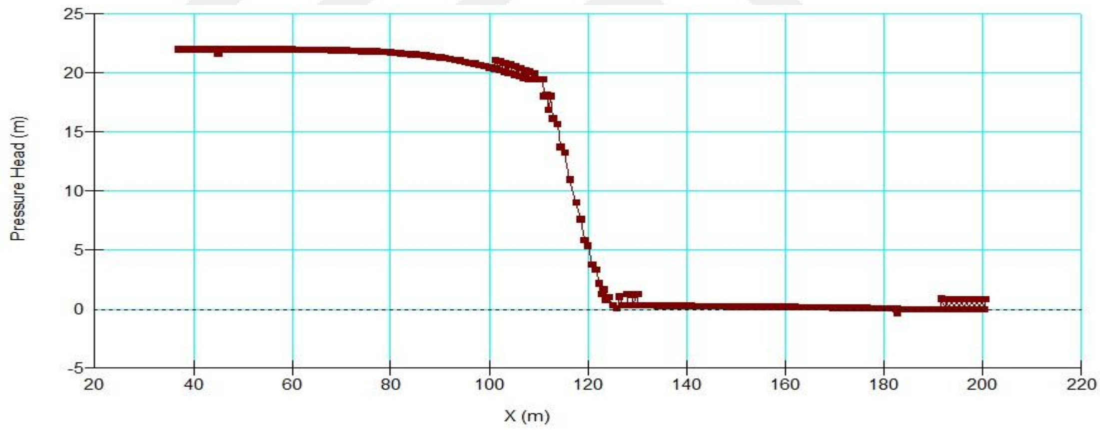

Figure 5.12. Pressure head graph in steady state seepage for Shauger dam.

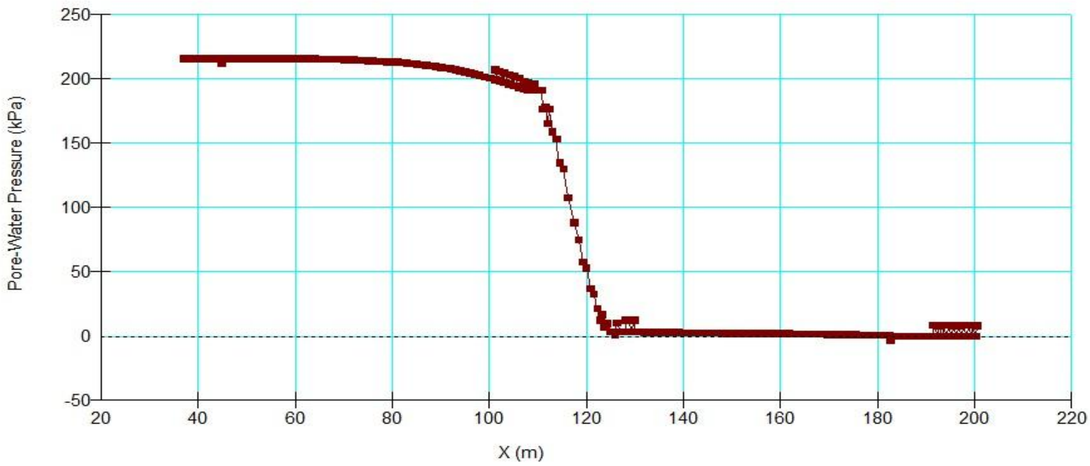

Figure 5.13. Pore water pressure graph in steady state seepage for Shauger dam.

Figures 5.14, 5.15 and 5.16 show the Y-Gradient, X-Gradient and XY-Gradient vs. distance respectively, at level 0m. Values between distances 110-125m of Gradient are the highest values for X and XY Gradient, and these values are expected because of steep drop of phreatic line in this range of distance inside the dam which due to low permeable clay core. Also the flow direction components in this range inside the dam are downward in Y-direction and toward D/S in X-direction which interprets the small and negative values (less than 1) of Y-Gradient, which refer to no piping (boiling) will be occur in Y-direction. While the values of the X and XY-Gradient in the distance ranged between 110-125m are very high and referring to risk of erosion occurring for the fine material inside the core in X-direction.

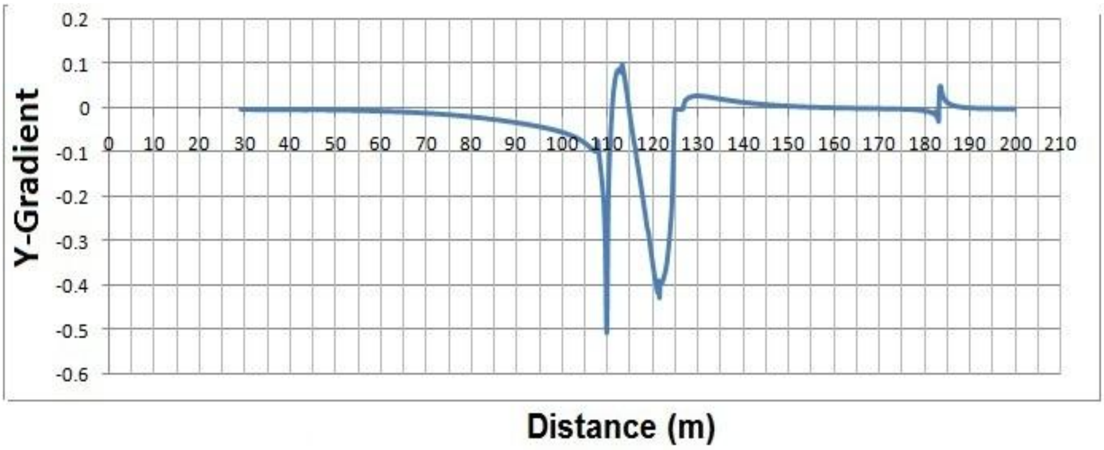

Figure 5.14. Y-Gradient vs. distance at level 0m.

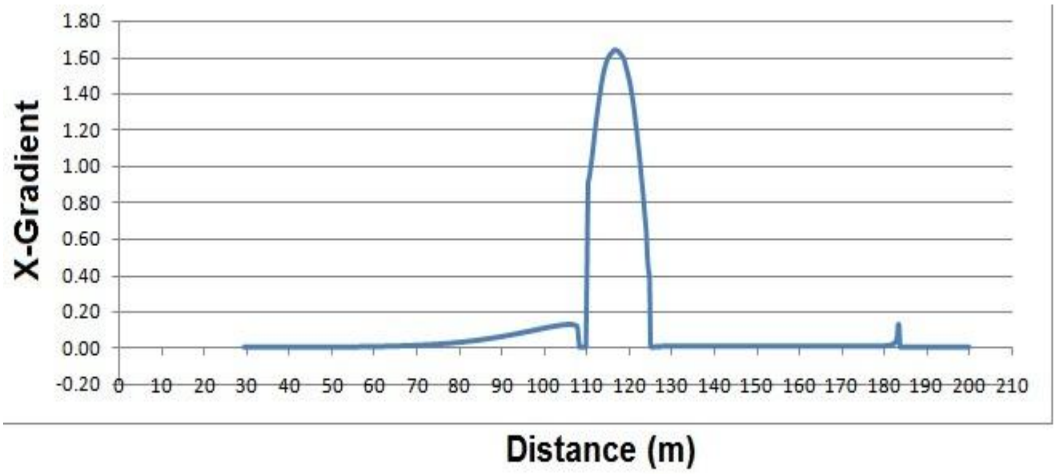

Figure 5.15. X-Gradient vs. distance at level 0m.

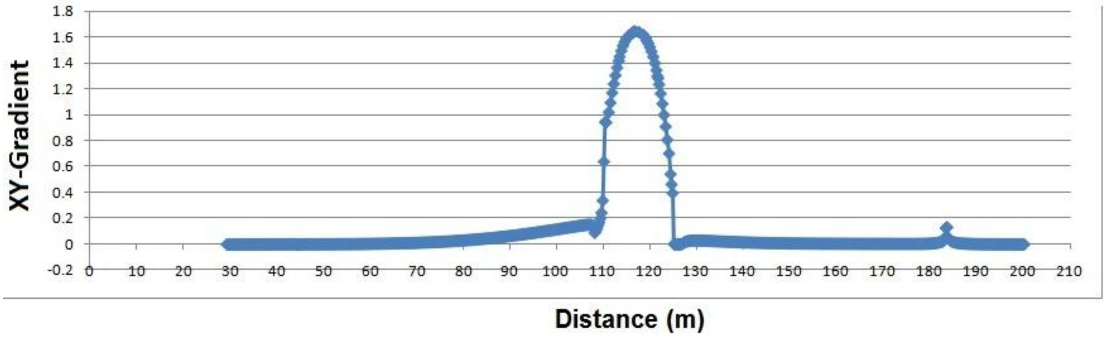

Figure 5.16. Y-Gradient vs. distance at level 0m.

5.2.5 SLOPE STABILITY ANALYSIS AT FULL RESERVOIR AND STEADY STATE

Slope stability analysis at the full reservoir coducted by using many methods for both D/S and U/S. Table 5.2 summarizes the FoS results for D/S and U/S using many methods for full reservoir. The results are as shown in figures. Where figure 5.17 shows FoS for D/S slope stability analysis by using Bishop method for Shauger dam and figure 5.18 shows FoS for U/S slope stability analysis.

Table 5.2. FoS results for full reservoir using many methods.

	Bishop	Ordinary	Janbu	Morgenstern – Price
D/S	2.156	2.025	2.025	2.157
U/S	2.999	2.745	2.914	3.002

Table 5.2 shows that Ordinary and Janbo methods give fewest values for FoS than the others, and that the D/S had the less value for FoS was 2.025, which is more than the required value from the USDA-NRCS (U.S. Department of Agriculture-Natural Resource Conservation service) (210-VI-TR60, July 2005) specification which is 1.5 and from U.S. Army Corps Of Engineers (USACE)Engineering And DesignSlope Stability EM 1110-2-190231 Oct 2003 which is 1.5 for this case.That means the dam is in safe side for this case.

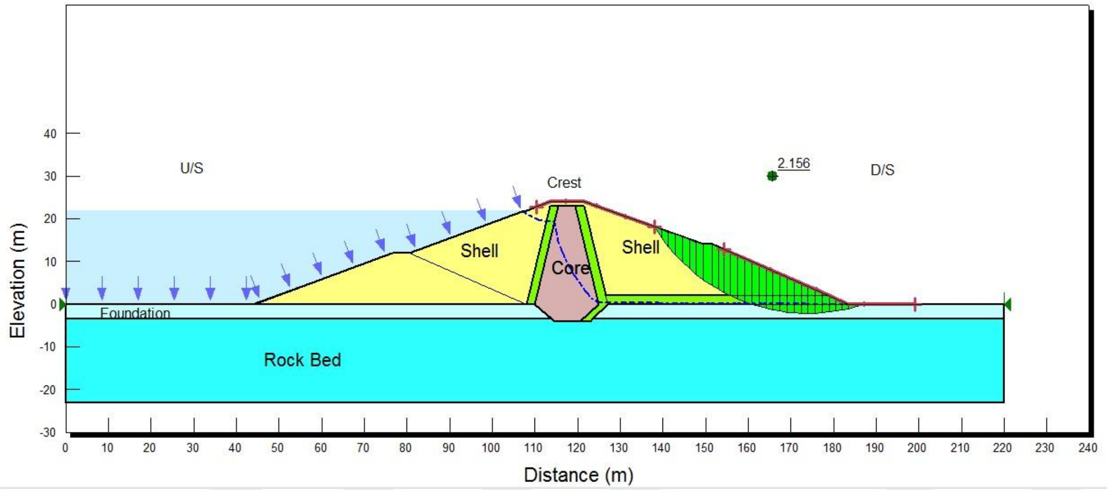

Figure 5.17. Slope stability D/S analysis with full reservoir for Shauger dam.

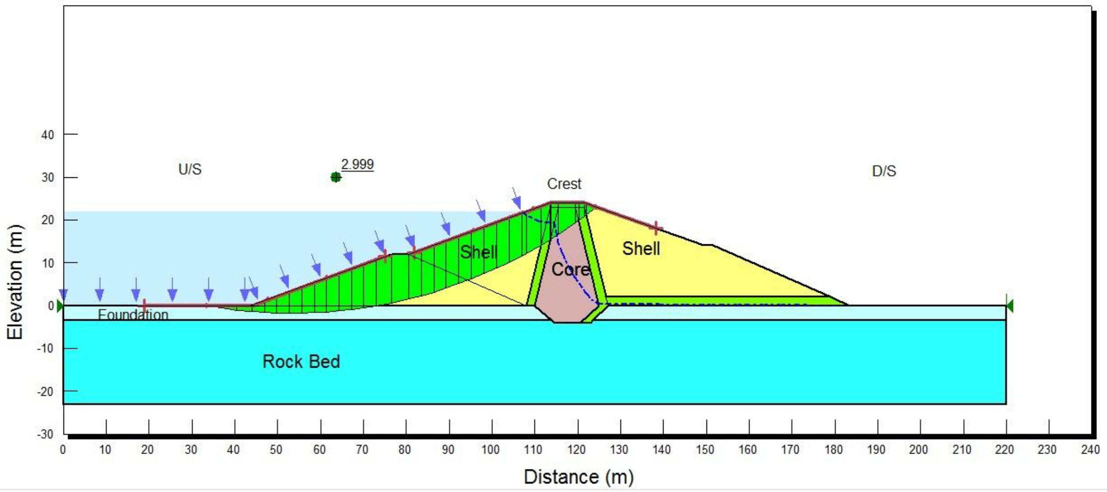

Figure 5.18. Slope stability U/S analysis with full reservoir for Shauger dam.

5.2.6 SLOPE STABILITY ANALYSIS IN CASE OF RAPID DRAWDOWN

Slope stability analysis at the rapid drawdown and steady state seepage from the full level to lowest level at river bed conducted by using many methods for U/S which is critical case, table 5.3 summarizes the FoS results for D/S and U/S using many methods for rapid drawdown case. The results are shown. Figure 5.19 shows FoS for U/S slope stability analysis.

Table 5.3. FoS results for rapid drawdown using many methods.

	Bishop	Ordinary	Janbu	Morgenstern – Price
U/S	1.372	1.181	1.268	1.388

Table 5.3 shows that Ordinary method gives fewest values for FoS than the others, and that the U/S less value for FoS was 1.181, which is less than the required value from the USDA-NRCS (U.S. Department of Agriculture-Natural Resource Conservation service) (210-VI-TR60, July 2005) specification which is 1.2.And according to U.S. Army Corps Of Engineers (USACE)Engineering And DesignSlope Stability EM 1110-2-190231 Oct 2003 the value required is 1.3 for this case that means both values of Ordinary and Janbu methods (gives FoS equal to 1.181 and 1.268 respectively) are less than the required one. Therefore it can be consider that the dam is not safe for this case and there is a risk of U/S face failure. But if Bishop and Morgenstern-Price methods results considered it can be say that the dam is in safe side.

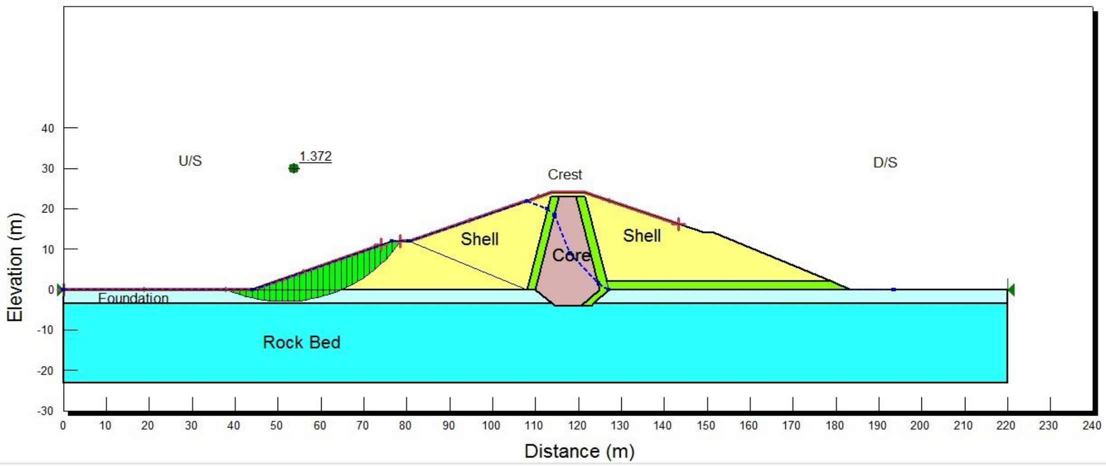

Figure 5.19. Slope stability U/S analysis with rapid drawdown for Shauger dam.

5.2.7 SLOPE STABILITY DYNAMIC ANALYSIS WITH EARTHQUAKE

Slope stability dynamic analysis conducted for full reservoir and earthquake case, by using k_h coefficient equal to 0.2 because the dam is located at zone 2. By using many methods for both U/S and D/S.Table 5.4 summarizes the FoS results for D/S and U/S using many methods for full reservoir earthquake case. The results are as shown, figure 5.20 shows FoS for U/S slope stability earthquake dynamic analysis by using Bishop method for Shauger dam and figure 5.21 shows FoS for D/S slope stability earthquake dynamic analysis.

Table 5.4. FoS results for dynamic analysis using many methods.

	Bishop	Ordinary	Janbu	Morgenstern – Price	QUAKE/W Stress	QUAKE/W Stress Newmark deformation
D/S	1.203	1.257	1.322	1.353	1.015	1.887
U/S	2.824	1.246	2.668	2.823	2.593	2.613

Table 5.4 shows that QUAKE/W Stress method gives fewest values for FoS than the others, and that the D/S less value for FoS was 1.015, which is less than the required value from USDA-NRCS specification which is 1.1 for this case.But if the results of other three methods consider which all were more than the specification one then the dam is in safe side. While for U/S Ordinary method gave the less value for FoS which was 1.246 and this is more than the required value from specification which is 1.1 for this case also. Therefore the dam is in safe side for this case.

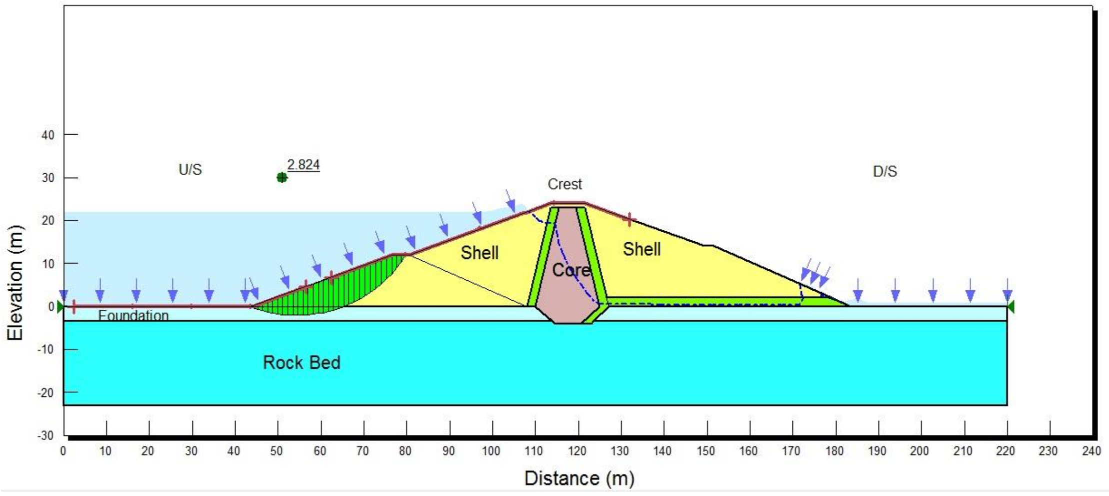

Figure 5.20. Slope stability U/S with earthquake dynamic analysis for Shauger dam.

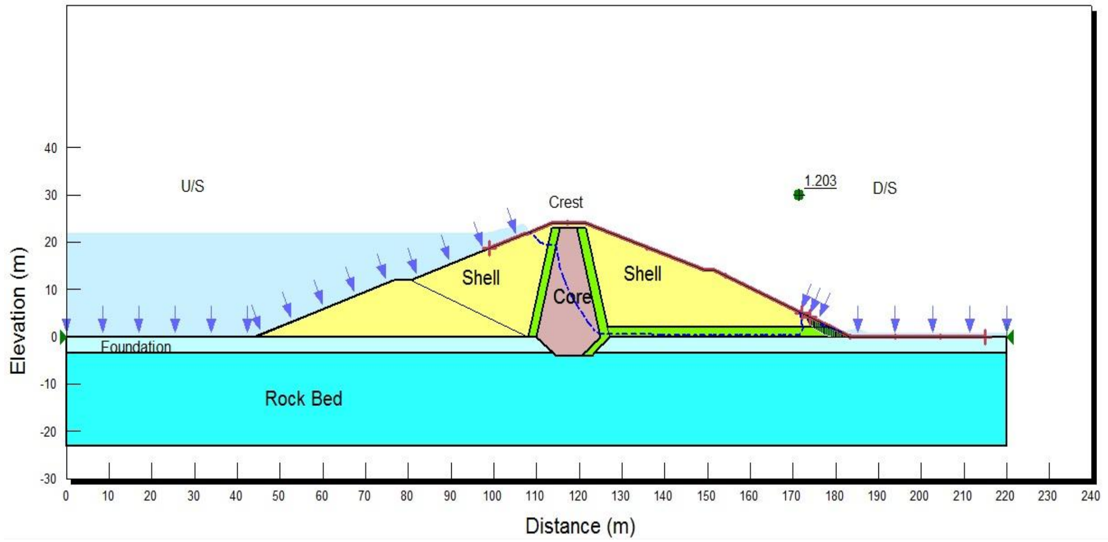

Figure 5.21. Slope stability D/S with earthquake dynamic analysis for Shauger dam.

5.2.8 DISPLACEMENT, EFFECTIVE STRESS AND LIQUEFACTION IN DYNAMIC ANALYSIS

Initial case analysis before earthquake occurring at the full reservoir case conducted. Then stability analysis in case of earthquake dynamic analysis conducted for Shauger earth dam by using k_h earth acceleration coefficient equal to 0.2. Figure 5.22 shows displacement with earthquake dynamic analysis. While figure 5.23 shows Y-displacement vs distance graph in case of earthquake dynamic analysis. Figure 5.24 shows X-displacement vs Y graph in case of earthquake dynamic analysis.

From figure 5.23 graph results show that the maximum settlement due to earthquake was at distance about 90m and its value is -0.012m, while the maximum heave due to earthquake was at distance about 140m and its value is +0.013m.

From figure 5.24 graph results show that the maximum lateral displacement due to earthquake was at level about (Y) 22m and its value is -0.094 m, while the minimum lateral displacement due to earthquake was at level(Y) about -4m and its value is -0.039m.

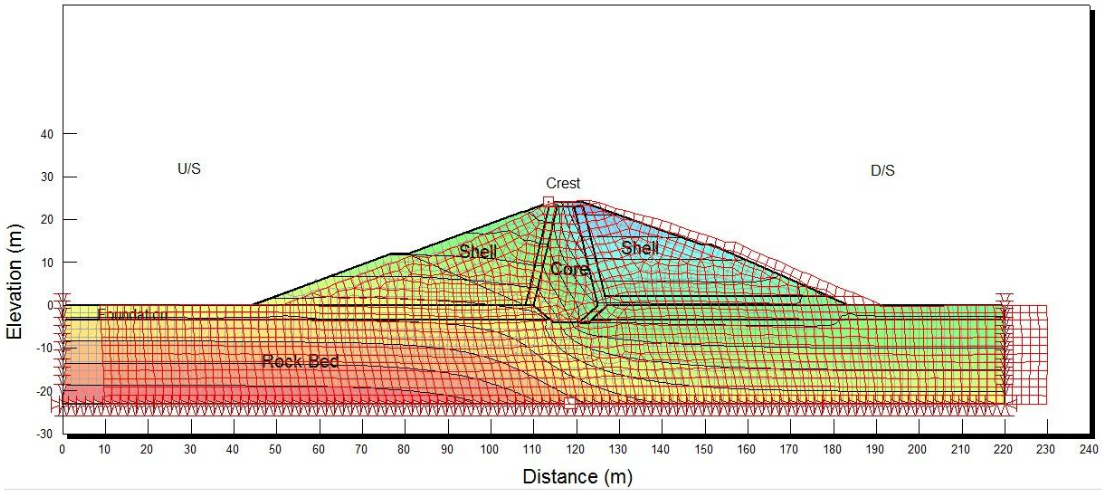

Figure 5.22. Displacement with earthquake dynamic analysis at 10 seconds (magnification100 times).

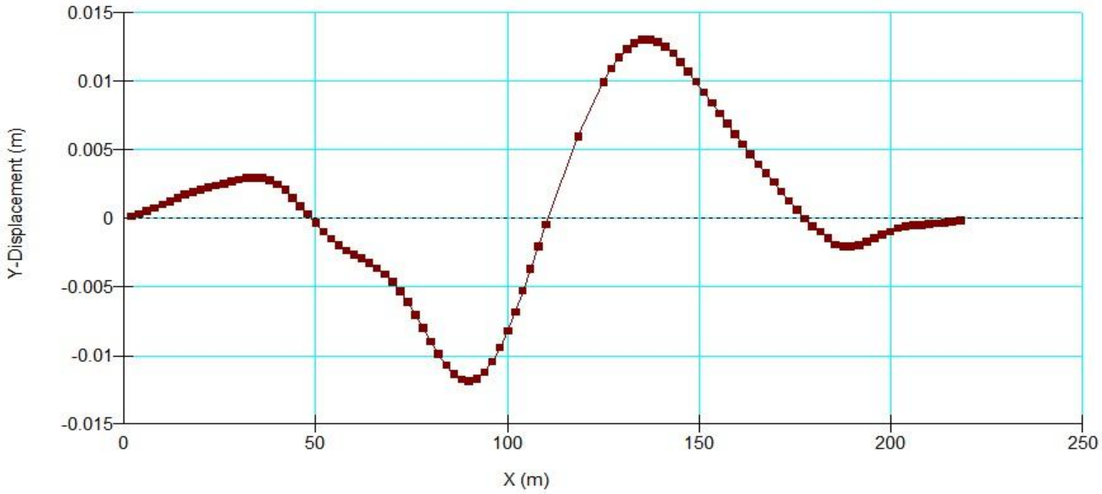

Figure 5.23. Y-displacement vs distance graph with earthquake dynamic analysis.

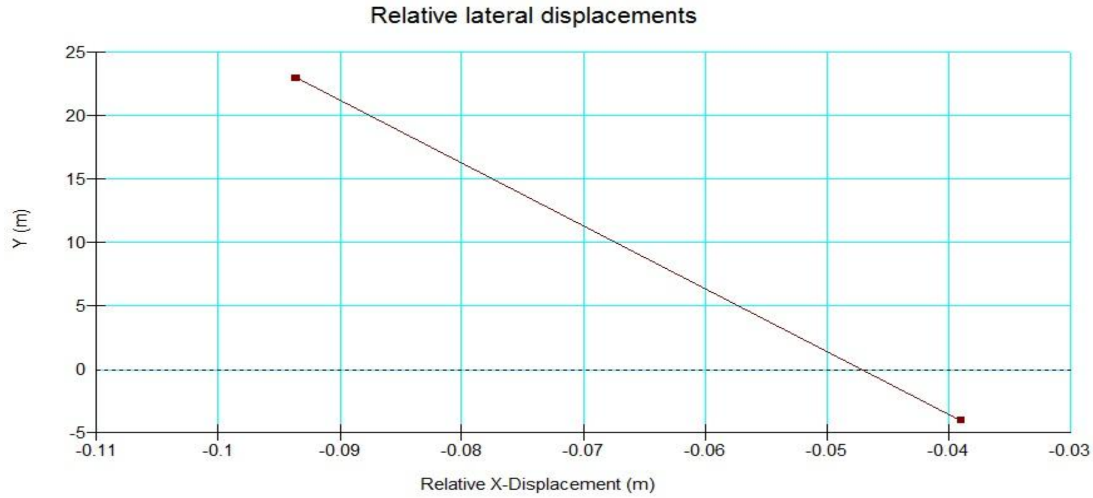

Figure5.24. X-displacement vs Y graph with earthquake dynamic analysis.

Figure 5.25 shows effective stress in case of earthquake dynamic analysis. Where, the contour of the stress deformed and pushed toward D/S laterally that concord with lateral displacement due to earthquake.

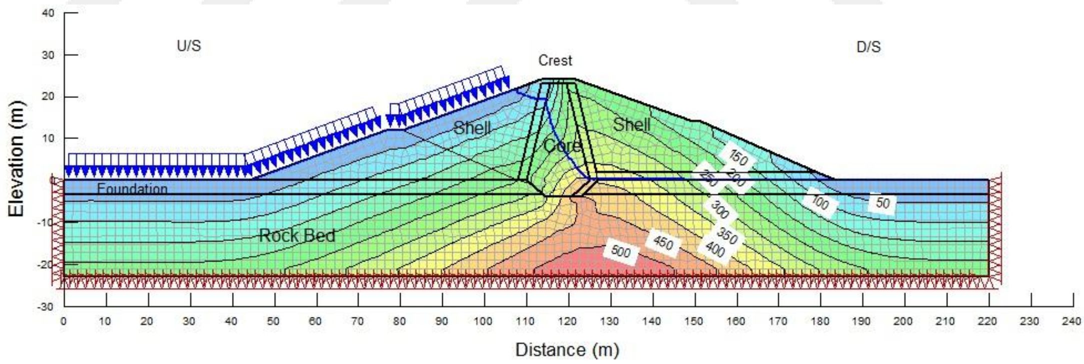

Figure 5.25. Effective stress in case of earthquake dynamic analysis.

While figure 5.26 shows Liquefaction zone (cyclic stress ratio) with earthquake dynamic analysis. From figure 5.26 results show that the maximum critical liquefaction zones (cyclic stress ratio) were at the U/S face which its maximum value reaches to 1.4, and less values at the toe of the D/S which its value reaches to1. These values indicate that the probability of liquefaction occurring is higher at U/S face.

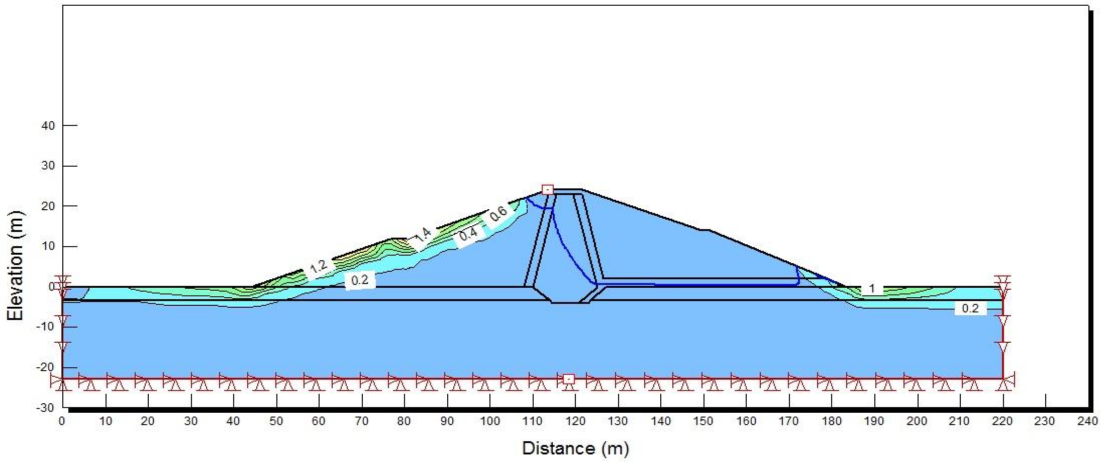

Figure 5.26. Liquefaction zone (cyclic stress ratio) with earthquake dynamic analysis.

5.2.9 INSTANTANEOUS TRANSIENT DRAWDOWN ANALYSIS

Instantaneous transient drawdown analysis conducted for 7days full drop from 22m to 0m reservoir level. Figure 5.27 shows PWP in case of instantaneous transient drawdown analysis at 7^{th} day.

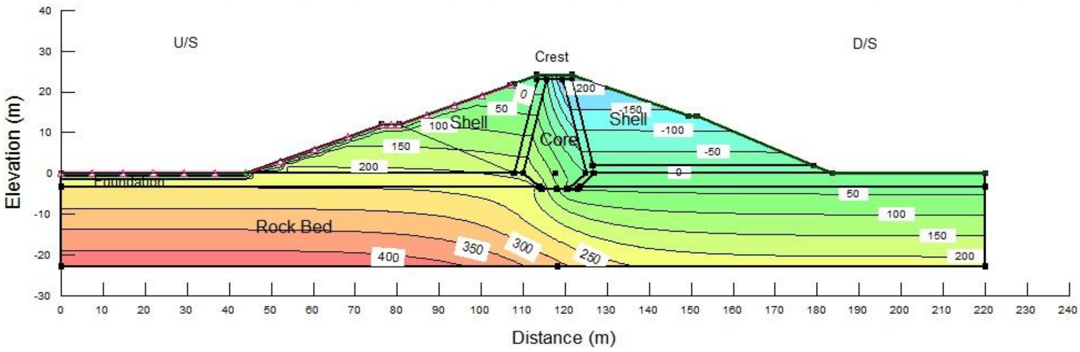

Figure 5.27. PWP in case of instantaneous drawdowntransient analysis at 7^{th} day.

While figures 5.28 and 5.29 shows slope stability U/S analysis in case of instantaneous drawdown at 6hrs and at 7^{th} day respectively. Where figure 5.30 shows minimum FoS vs. time steps in instantaneous drawdown for 7days.

The FoS increases with time this is very logically because release of PWP with time which due to dissipation of water from the soil. FoS for the first four days was less than one then at 7th day rises above one, these FoSs are critical because they indicate to slope slide and failure, but the whole case of instantaneous drawdown is hypothetical case not really, therefore this case can be neglected.

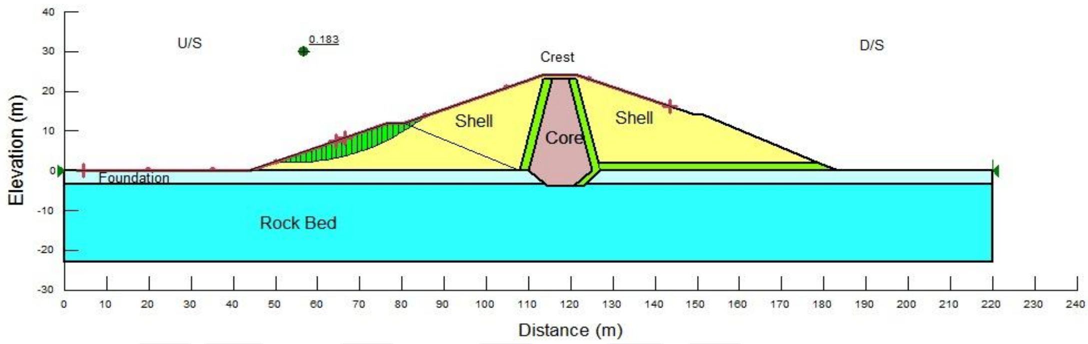

Figure 5.28. Slope stability U/S analysis in case of instantaneous drawdown at 6hrs.

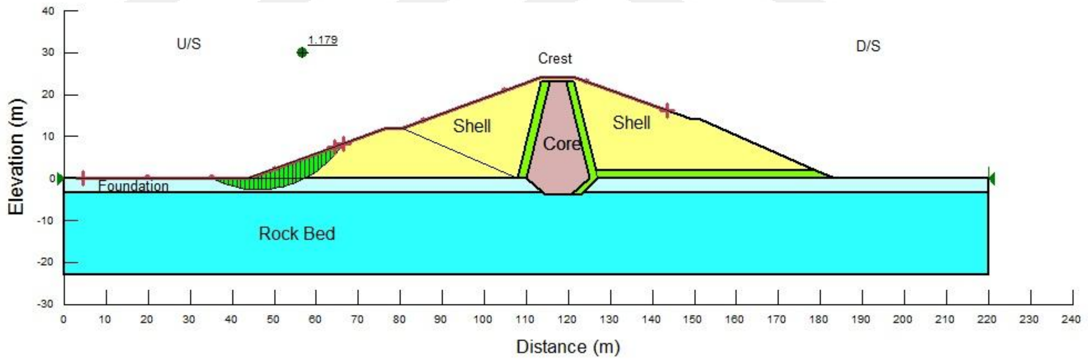

Figure 5.29. Slope stability U/S analysis in case of instantaneous drawdown at 7th day.

Figure 5.30. Minimum FoS vs. time steps in instantaneous drawdown for 7days.

5.2.10 SLOW DRAWDOWN TRANSIENT ANALYSIS AT 7 DAYS

Slow drawdown transient analysis conducted for 7days drop from 22m to 0m reservoir level in steps time. Figure 5.31 shows total head in case of slow drawdown transient analysis at 7th day. And figure 5.32 shows PWP in case of slow drawdowntransient analysis at 7th day. These two figures show disturbance in pressure in U/S face due to releasing the hydrostatic pressure of reservoir on this face, while the PWP still not released because of low conductivity of the soil.

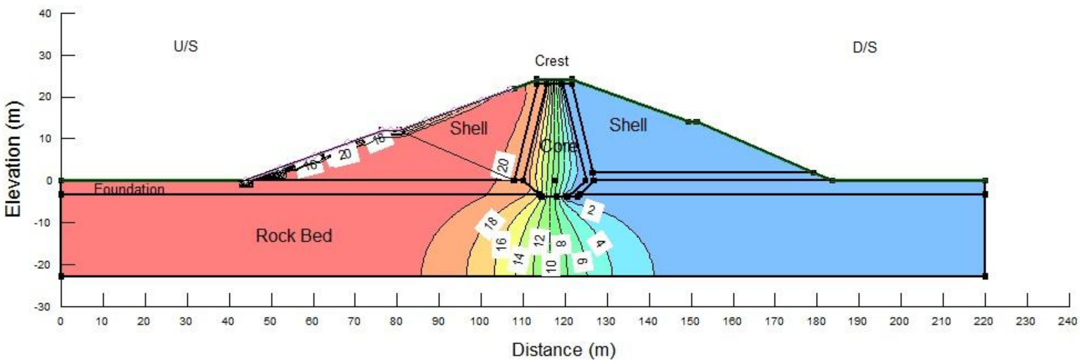

Figure 5.31. Total head in case of slow drawdown analysis at 7th day.

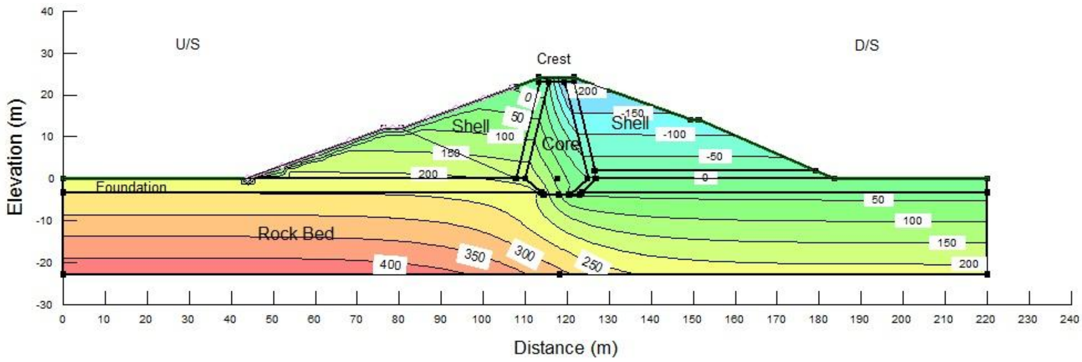

Figure 5.32. PWP in case of slow drawdown analysis at 7th day.

While figure 5.33 shows phreaticlines for all time steps. The phreatic line lowers near U/S side with time, which due to lower in reservoir level in step time. Phreatic line lowers slowly with each time step because of low conductivity of the soil, which in turn slows the dissipation of pore water pressure inside the soil.

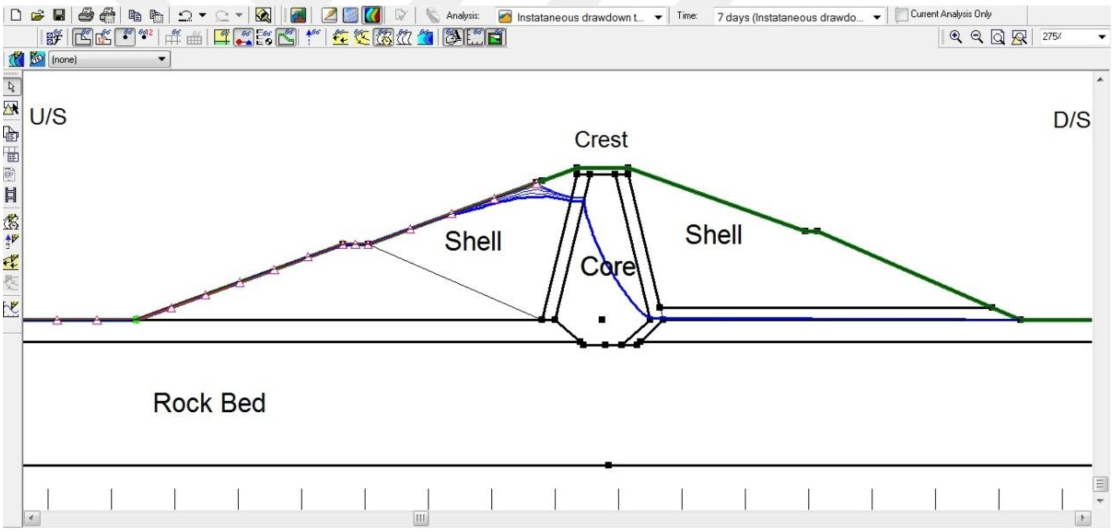

Figure 5.33. Phreaticlines for all time steps in transient slow drawdown in 7days.

Figure 5.34 and figure 5.35 are showing slope stability U/S analysis in case of slow drawdown at 6hrs and at 7th day respectively. And figure 5.36 shows minimum FoS vs. time steps in transient slow drawdown for 7days.ThevalueofFoS reduces with time steps from 2.78 at 6hrs to 1.44 at 7th day, because in each time step the pore water hasn't enough time to dissipate moreover the next time accumulate and the reservoir level lowers slow with each time step and not instantaneously in one step. While in the previous case (sec. 5.2.10) the reservoir level drops instantaneously then the pore water dissipation increased with time, i.e. increasing the FoS with time.

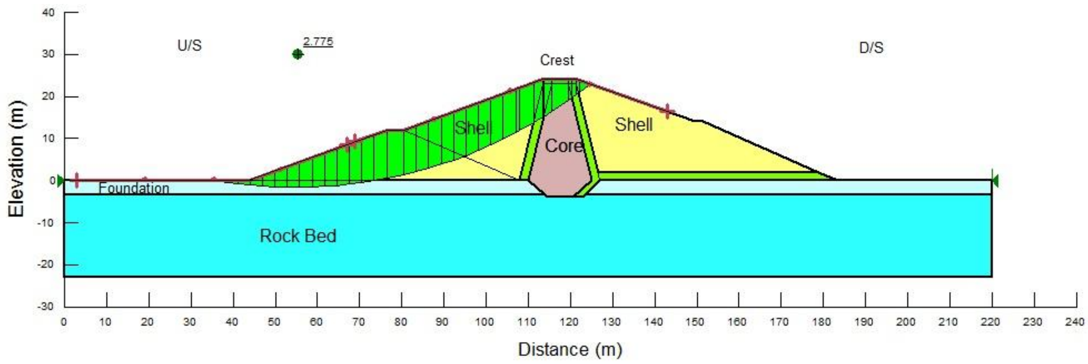

Figure 5.34. Slope stability U/S analysis in case of slow drawdown at 6hrs.

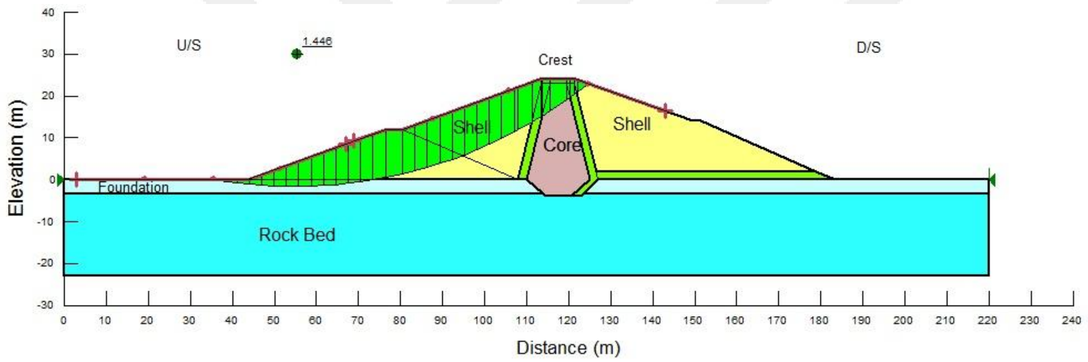

Figure 5.35. Slope stability U/S analysis in case of slow drawdown at 7th day.

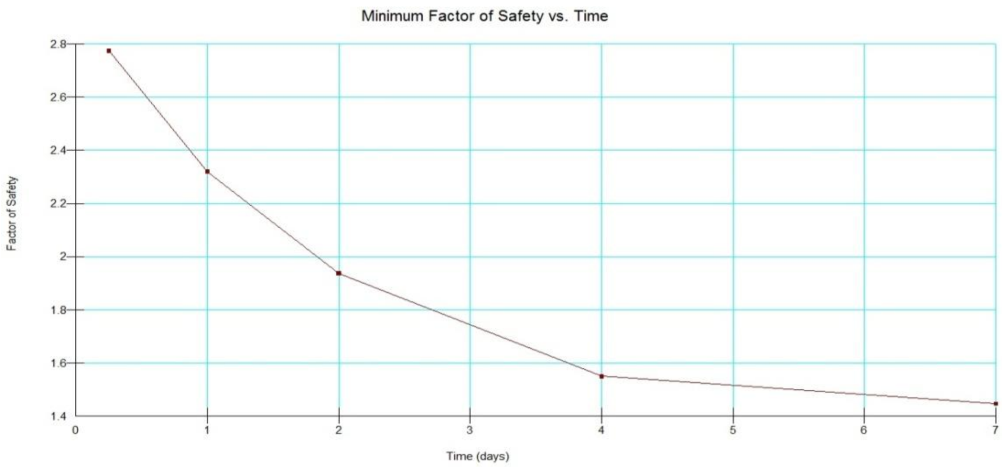

Figure 5.36. Minimum FoS vs. time steps in transient slow drawdown for 7days.

5.2.11 SLOW DRAWDOWN TRANSIENT ANALYSIS AT 30 DAYS

Slow drawdown transient analysis conducted for 30days drop from 22m to 0m reservoir level in steps time. Figure 5.37 shows phreaticlines for all time steps in transient slow drawdown in 30days. And figure 5.38 shows minimum FoS vs. time steps in transient slow drawdown for 30days.

The value of FoS reduces with time step for first 7days to 1.44 then increases with time to 1.61 at 30th day because the time step after 7days became wide steps and the time were available for sufficient somewhat dissipation of PWPfrom the pores of soil.

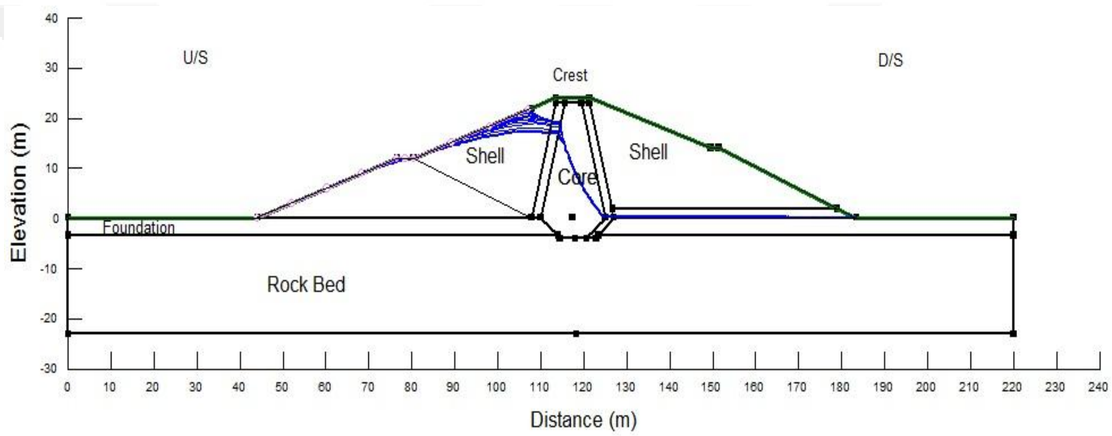

Figure 5.37. Phreaticlines for all time steps in transient slow drawdown in 30days.

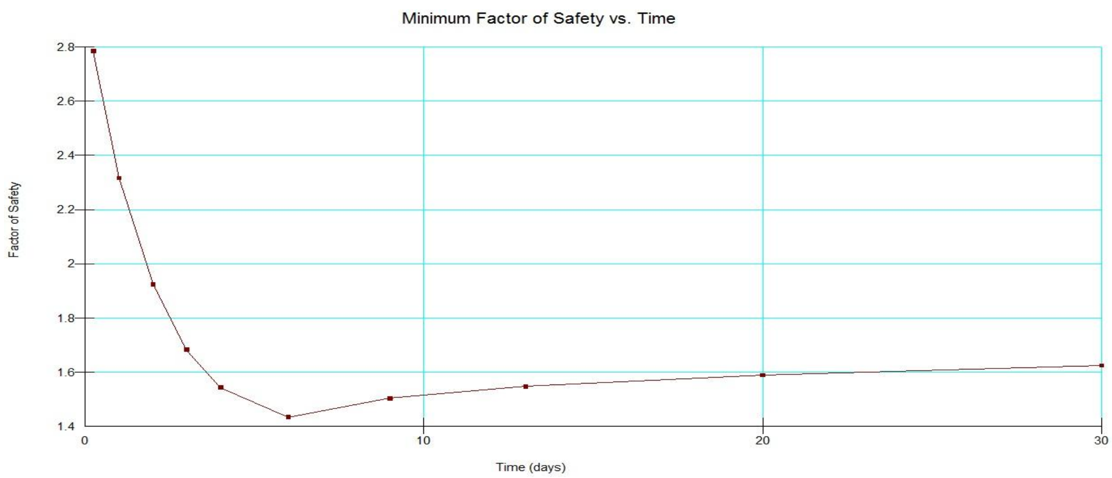

Figure 5.38. Minimum FoS vs. time steps in transient slow drawdown for 30days.

Summaryof factor of safety results for all cases is shown in the table 5.5.

Table 5.5. Summary of factor of safety resultsfor all cases.

Case at		Bishop	Ordinary	Janbu	Morgenstern – Price	QUAKE/W Stress	QUAKE/W Stress Newmark deformation
end of construction	D/S	2.327	2.280	2.273	2.323	---------	---------
	U/S	2.709	2.644	2.639	2.708	---------	---------
full reservoir	D/S	2.156	2.025	2.025	2.157	---------	---------
	U/S	2.999	2.745	2.914	3.002	---------	---------
rapid drawdown	D/S	---------	---------	--------	---------	---------	---------
	U/S	1.372	1.181	1.268	1.388		
dynamic analysis	D/S	1.203	1.257	1.322	1.353	1.015	1.887
	U/S	2.824	1.246	2.668	2.823	2.593	2.613

More details can be found in the appendix A.

CHAPTER SIX

CONCLUSIONS AND FUTURE WORKS

6.1 PREFACE

This chapter presents the most important conclusions from results, and introduces the scope for future works.

6.2 CONCLUSIONS

From the results the following conclusions can be considered as the main and important points that can bring out from the results of this thesis:

1. Shauger earth dam is in the safe case according to the slope stability for the cases: at the end of construction, full reservoir (maximum water level) and transient slow drawdown.
2. Shauger earth dam is not in the safe case according to the Ordinary and Janbo methods for U/S slope stability analysis for the case of rapid drawdown which was 1.181 and 1.268respectively, which is less than the minimum value 1.3 required according to the specifications andthere is a risk of U/S face failure. But if the results of other methods considered (see table 5.3), then the dam can be considered as safe for this case.
3. Shauger earth dam is not in safe case according toQUAKE/W Stress method for the slope stability for the case full reservoir (maximum water level) with earthquake. The factor of safety for D/S was1.015 in this case, which is less than the minimum value (1.1) required according to the specifications. Butif the results of other methods are considered (see table 5.4), then the dam can be considered as safe.
4. The total amount of the seepage through the Shauger earth dam in both cases without and with earthquake are 0.46060m^3/day and 0.3990m^3/day respectively. These values can be considered in permissible range.
5. The gradient values beneath the dam and at the toe of the dam are less than the critical value i.e. good evidence indicates thatpiping will not occur.

6.3 FUTURE WORKS

Future works may involves three dimension analysis for the earth dams, and/or taking the ageing factors in consideration, and/or taking in consideration the reservoir water waves effect during the earthquake besides the other factors.

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APPENDIX A

Reports aboutShauger earth dam analysis numerical model

1. Insitu

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

File Information

Title: Shauger earth dam

Comments: assessment of the Shauger earth dam safety 2015-2016.

Created By: Fathi Ghazi Fathi Bajlan

Revision Number: 321

Date: 1/5/2016

Time: 2:31:28 AM

File Name: Shauger dam analysis3333.gsz

Directory: C:\Users\acer\Desktop\Shauger Dam\

Project Settings

Length(L) Units: meters

Time(t) Units: Days

Force(F) Units: kN

Pressure(p) Units: kPa

Strength Units: kPa

Stiffness Units: kPa

Unit Weight of Water: 9.807 kN/m³

Air Pressure: 101.33 kPa

View: 2D

Analysis Settings

Insitu

Kind: SIGMA/W

Method: Insitu

Settings

Initial PWP: Water Table

Control

Apply Body Force in All Steps: Yes

Use Constant Stress: Yes

Convergence

Maximum Number of Iterations: 10

Displacement Norm Tolerance: 1 %

Equation Solver: Parallel Direct

Time

Starting Time: 0 days

Duration: 0 days

Ending Time: 0 days

Materials

Foundation (Quaternary) for instu analysis

Model: Linear Elastic

Stress Strain

Effective Young's Modulus (E'): 10000 kPa

Poisson's Ratio: 0.334

Unit Weight: 19 kN/m³

Rock bed for instu analysis

Model: Linear Elastic

Stress Strain

Effective Young's Modulus (E'): 10000 kPa

Poisson's Ratio: 0.334

Unit Weight: 20 kN/m³

Boundary Conditions

Fixed X

X: X-Displacement 0

Y: 0

Fixed X/Y

X: X-Displacement 0

Y: Y-Displacement 0

Initial Water Tables

Initial Water Table 1

Max. negative head: 5

Coordinates

Coordinate: (0, 0) m

Coordinate: (44, 0) m

Coordinate: (230, 0) m

Regions

	Points	Area (m^2)	Material
Region 1	1,2,3,4,5,34,6,23	260.5	
Region 2	1,7,8,2	46	
Region 3	5,9,24,10,11,12,13,14,3,4	163.75	
Region 4	15,16,17,30,31,18,19,20,21,12,13,14,3,2,8,7	1394.25	
Region 5	22,23,6,34,5,9,24,25,26,32,27	4285.4844	Rock bed for instu analysis
Region 6	28,15,7,1,23,22	391.89063	Foundation (Quaternary) for instu analysis
Region 7	25,29,11,10,24	331.625	Foundation (Quaternary) for instu analysis

Lines

	Start Point	End Point	Stress/Strain Boundary
Line 1	1	2	
Line 2	2	3	

Line 3	3	4	
Line 4	4	5	
Line 5	1	7	
Line 6	7	8	
Line 7	8	2	
Line 8	5	9	
Line 9	10	11	
Line 10	11	12	
Line 11	12	13	
Line 12	13	14	
Line 13	14	3	
Line 14	15	16	
Line 15	16	17	
Line 16	18	19	
Line 17	19	20	
Line 18	20	21	
Line 19	21	12	
Line 20	7	15	
Line 21	6	23	
Line 22	23	1	
Line 23	9	24	
Line 24	24	10	
Line 25	22	23	
Line 26	24	25	
Line 27	25	26	Fixed X
Line 28	27	22	Fixed X
Line 29	28	15	
Line 30	22	28	Fixed X
Line 31	25	29	Fixed X
Line 32	29	11	
Line 33	17	30	
Line 34	30	31	
Line 35	31	18	
Line 36	26	32	Fixed X/Y
Line 37	32	27	Fixed X/Y
Line 38	5	34	
Line 39	34	6	

Points

	X (m)	Y (m)
Point 1	110	0
Point 2	115.5	23

Point 3	119.5	23
Point 4	125	0
Point 5	120.5	-4
Point 6	114.5	-4
Point 7	108	0
Point 8	113.5	23
Point 9	123	-4
Point 10	127	0
Point 11	183.5	0
Point 12	179	2
Point 13	126.5	2
Point 14	121.5	23
Point 15	44	0
Point 16	76.5	12
Point 17	80.5	12
Point 18	113.5	24
Point 19	121.5	24
Point 20	149.5	14
Point 21	151.5	14
Point 22	0	-3.5
Point 23	113.9375	-3.5
Point 24	123.5	-3.5
Point 25	220	-3.5
Point 26	220	-23
Point 27	0	-23
Point 28	0	0
Point 29	220	0
Point 30	107	21.636364
Point 31	108	22
Point 32	118.5	-23
Point 33	117.5	0
Point 34	118	-4

2. Dam stress

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Dam stress

Kind: SIGMA/W

Parent: Insitu

Method: Load/Deformation

Settings

Initial Stress: Parent Analysis

Initial PWP: (none)

Exclude cumulative values: No

Control

Apply Body Force in All Steps: Yes

Use Constant Stress: Yes

Adjust Fill: No

Convergence

Maximum Number of Iterations: 10

Displacement Norm Tolerance: 1 %

Equation Solver: Parallel Direct

Time

Starting Time: 0 days

Duration: 1 days

# of Steps: 1

Save Steps Every: 1

Use Adaptive Time Stepping: No

Materials

Core

Model: Linear Elastic

Stress Strain

Young's Modulus (E): 10000 kPa

Unit Weight: 19.5 kN/m³

Poisson's Ratio: 0.334

Shell

Model: Linear Elastic

Stress Strain

Young's Modulus (E): 10000 kPa

Unit Weight: 20 kN/m³

Poisson's Ratio: 0.334

Filter

Model: Linear Elastic

Stress Strain

Young's Modulus (E): 10000 kPa

Unit Weight: 20 kN/m³

Poisson's Ratio: 0.334

Foundation (Quaternary) loading phase

Model: Elastic-Plastic

Stress Strain

Total E-Modulus Function: Foundation stiffness

Poisson's Ratio: 0.49

Total Cohesion Function: undraind strength C

Phi: 0 °

Unit Weight: 19 kN/m³

Dilation Angle: 0 °

Rock bed loading phase

Model: Elastic-Plastic

Stress Strain

Total E-Modulus Function: Rock bed stiffness

Poisson's Ratio: 0.49

Total Cohesion Function: Rock bed undraind strength C

Phi: 0 °

Unit Weight: 20 kN/m³

Dilation Angle: 0 °

Boundary Conditions

Fixed X

X: X-Displacement 0 Y: 0

Fixed X/Y

X: X-Displacement 0

Y: Y-Displacement 0

Initial Water Tables

Initial Water Table 1

Max. negative head: 5

Coordinates

Coordinate: (0, 0) m

Coordinate: (44, 0) m

Coordinate: (230, 0) m

Total C Functions

undraind strength C

Model: Spline Data Point Function

Function: Total Cohesion vs. Y-Total Stress

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Y-Intercept: 0

Data Points: Y-Total Stress (kPa), Total Cohesion (kPa)

Data Point: (-3, 0)

Data Point: (0, 0)

Rock bed undraind strength C

Model: Spline Data Point Function

Function: Total Cohesion vs. Y-Total Stress

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Y-Intercept: 4000

Data Points: Y-Total Stress (kPa), Total Cohesion (kPa)

Data Point: (-23, 5000)

Data Point: (-3, 4000)

Total E-Modulus Functions

Foundation stiffness

Model: Spline Data Point Function

Function: Total E-Modulus vs. Y-Total Stress

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Y-Intercept: 5000

Data Points: Y-Total Stress (kPa), Total E-Modulus (kPa)

Data Point: (-3, 6000)

Data Point: (0, 5000)

Estimation Properties

Depth: 1 m

K-Modifier: 1

N-Exponent: 0.5

K0: 1

Rock bed stiffness
Model: Spline Data Point Function

Function: Total E-Modulus vs. Y-Total Stress

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Y-Intercept: 6000

Data Points: Y-Total Stress (kPa), Total E-Modulus (kPa)

Data Point: (-23, 12000)

Data Point: (-3, 6000)

Estimation Properties

Depth: 1 m

K-Modifier: 1

N-Exponent: 0.5

K0: 1

3. SLOPE/W Analysis D/S

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

SLOPE/W Analysis D/S
Kind: SLOPE/W
Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: (none)
SlipSurface
Direction of movement: Left to Right
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 m
Minimum Slice Width: 0.1 m
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1

Materials

Core

Model: Mohr-Coulomb
Unit Weight: 19.5 kN/m³
Cohesion: 70 kPa
Phi: 0 °
Phi-B: 0 °

Shell

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 20 kPa
Phi: 35 °
Phi-B: 0 °

Filter

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 0 kPa
Phi: 33 °
Phi-B: 0 °

Foundation (Quaternary)

Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Phi-B: 0 °

Rock bed

Model: Mohr-Coulomb
Unit Weight: 2 kN/m³
Cohesion: 4000 kPa
Phi: 0 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (102.00977, 19.821735) m
Left-Zone Right Coordinate: (136.9, 18.5) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (141.16008, 16.978543) m
Right-Zone Right Coordinate: (199.5, 0) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	67	2.327	(182.062, 93.293)	93.315	(119.563, 24)	(184.054, 0)

Slices of Slip Surface: 67

	Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	67	120.12665	23.5	0	1.8791231	1.3157762	20
2	67	121.09505	22.6505	0	21.75498	14.127849	0
3	67	121.60435	22.21225	0	28.299311	18.377788	0
4	67	122.7776	21.243985	0	31.209849	21.853372	20
5	67	124.9154	19.5363	0	47.227265	33.068887	20
6	67	127.0532	17.927955	0	61.923143	43.359051	20
7	67	129.191	16.412705	0	75.331998	52.748033	20
8	67	131.3288	14.985135	0	87.491255	61.262036	20
9	67	133.46655	13.64052	0	98.425307	68.918142	20
10	67	135.6043	12.374725	0	108.16334	75.736789	20
11	67	137.7421	11.1841	0	116.72771	81.733619	20
12	67	139.8799	10.065411	0	124.14251	86.925523	20
13	67	142.0177	9.015793	0	130.41878	91.320215	20
14	67	144.1555	8.0326945	0	135.58157	94.935238	20
15	67	146.2933	7.113835	0	139.6434	97.779358	20
16	67	148.4311	6.2571785	0	142.6108	99.857159	20
17	67	150.5	5.484725	0	150.91252	105.67008	20
18	67	152.62445	4.7514415	0	156.64716	109.68552	20
19	67	154.8734	4.035079	0	153.21407	107.28165	20
20	67	157.12235	3.3806675	0	148.55282	104.0178	20
21	67	159.3713	2.7868625	0	142.66299	99.893704	20

22	67	161.62025	2.252477	0	135.55275	94.915055	20
23	67	163.76065	1.796791	0	129.82118	84.306861	0
24	67	165.79255	1.413688	0	121.01657	78.589082	0
25	67	167.82445	1.0769425	0	111.20704	72.218694	0
26	67	169.8564	0.78604805	0	100.38448	65.190444	0
27	67	171.88835	0.5405733	0	88.547657	57.503521	0
28	67	173.92025	0.340158	0	75.683069	49.14916	0
29	67	175.95215	0.1845111	0	61.779293	40.119942	0
30	67	177.98405	0.07340829	0	46.827353	30.410039	0
31	67	179.5347	0.01448449	0	34.695152	22.531295	0
32	67	180.92705	-0.01042945	0	22.985223	16.094426	0
33	67	182.64235	-0.01552117	0	7.9336255	5.5551844	0
34	67	183.77695	-0.00509172	0	0.0972808	0.068116778	0

4. SLOPE/W Analysis U/S

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

SLOPE/W Analysis U/S

Description: empty
Kind: SLOPE/W
Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: (none)
SlipSurface
Direction of movement: Right to Left
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (32.305936, 0) m
Left-Zone Right Coordinate: (67.5, 8.6769231) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (72.038548, 10.352695) m
Right-Zone Right Coordinate: (118.5, 24) m

Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	22	2.709	(47.933, 110.656)	111.754	(118.5, 24)	(32.3059, 0)

5 Steady-State Seepage Shauger dam

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Steady-State Seepage Shauger dam

Description: Shauger dam
Kind: SEEP/W
Method: Steady-State
Settings
Include Air Flow: No
Control
Apply Runoff: Yes

Boundary Conditions

Potential Seepage Face

Review: true
Type: Total Flux (Q) 0

Zero potential line

Type: Pressure Head 0

Researvoire head=22m

Type: Head (H) 22

Flux Sections

Flux Section 1

Coordinates
Coordinate: (118.5, -25) m
Coordinate: (118, 39.5) m

6 SLOPE/W Analysis D/S (2)

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

1-SLOPE/W Analysis D/S (2)

Description: Stability analysis (D/S) with full reservoir=Max. Level water
Kind: SLOPE/W
Parent: Steady-State Seepage Shauger dam
Method: Bishop, Ordinary and Janbu
Settings
  PWP Conditions Source: Parent Analysis
SlipSurface
  Direction of movement: Left to Right
  Allow Passive Mode: No
  Slip Surface Option: Entry and Exit
  Critical slip surfaces saved: 1
  Optimize Critical Slip Surface Location: No
  Tension Crack
    Tension Crack Option: (none)
FOS Distribution
  FOS Calculation Option: Constant

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (110.43387, 22.885044) m
Left-Zone Right Coordinate: (138, 18.107143) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (154.34175, 12.759964) m
Right-Zone Right Coordinate: (199, 0) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	118	2.156	(173.988, 40.019)	42.134	(138, 18.1071)	(187.168, 0)

7 SLOPE/W Analysis U/S (2)

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

2-SLOPE/W AnalysisU/S (2)

Description: Stability analysis (U/S) with full reservoir=Max. Level water
Kind: SLOPE/W
Parent: Steady-State Seepage Shauger dam
Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: Parent Analysis
SlipSurface
Direction of movement: Right to Left
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (19.00685, 0) m
Left-Zone Right Coordinate: (75.14583, 11.499999) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (81.80465, 12.474418) m
Right-Zone Right Coordinate: (138.3, 18) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	42	2.999	(53.263, 112.653)	114.364	(124.276, 23.0085)	(33.5554, 0)

8 SLOPE/W AnalysisU/S Rapid

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

3-SLOPE/W AnalysisU/S Rapid (3)

Description: Rapid drawdown

Kind: SLOPE/W
Parent: Steady-State Seepage Shauger dam
Method: Bishop, Ordinary and Janbu
Settings
    Apply Phreatic Correction: No
    PWP Conditions Source: Piezometric Line
    Use Staged Rapid Drawdown: No
SlipSurface
    Direction of movement: Right to Left
    Allow Passive Mode: No
    Slip Surface Option: Entry and Exit
    Critical slip surfaces saved: 1
    Optimize Critical Slip Surface Location: No
    Tension CrackTension Crack Option: (none)
FOS DistributionFOS Calculation Option: Constant

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (0, 0) m
Left-Zone Right Coordinate: (74, 11.076923) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (78.5, 12) m
Right-Zone Right Coordinate: (143.3119, 16.210036) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) mRight Coordinate: (220, 0) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X (m)	Y (m)
	0	0
	44	0
	76.5	12
	80.5	12
	108	22
	112.78261	20
	114.42391	18.5
	118	9
	124.6413	1.5
	127	0
	193.5	0

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	53	1.372	(51.33, 29.344)	32.234	(78.5, 12)	(37.9898, 0)

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9 Earthquake initial case

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

4-Earthquake initial case

Description: Stability analysis steady state seepage with earthquake inital state-Shauger dam.
Kind: QUAKE/W
Parent: Steady-State Seepage Shauger dam
Method: Initial Static
Settings
  Initial PWP: Parent Analysis
Control
  Coefficient of Equivalent Shear Stress: 0.65
  Coefficient of Equivalent Shear Strain: 0.5
  Equivalent Number of Cycles: 10
  Liquefaction Stress Limit: 0
Convergence
  Maximum Number of Iterations: 10
  Displacement Norm Tolerance: 1 %
  Equation Solver: Parallel Direct
  RileyDamping: 3
Time
  Starting Time: 0 days
  Duration: 0 days
  Ending Time: 0 days

Materials

Core

Model: Linear Elastic
Stress Strain
  Unit Weight: 19.5 kN/m³
  Poisson's Ratio: 0.334
  Damping Ratio: 0.1
  Pore Water Pressure Function: PWP function
  Cyclic Function: Cyclic # function
  G Modulus: 5000 kPa

Shell

Model: Linear Elastic
Stress Strain
  Unit Weight: 20 kN/m³
  Poisson's Ratio: 0.334
  Damping Ratio: 0.1
  Pore Water Pressure Function: PWP function
  Cyclic Function: Cyclic # function
  G Modulus: 5000 kPa

Filter

Model: Linear Elastic
Stress Strain

Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Foundation (Quaternary) for instu analysis

Model: Linear Elastic
Stress Strain
Unit Weight: 19 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Rock bed for instu analysis

Model: Linear Elastic
Stress Strain
Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Boundary Conditions

Fixed X

X: X-Displacement 0
Y: 0

Fixed X/Y

X: X-Displacement 0
Y: Y-Displacement 0

Reservoir Pressure

X: Fluid Pressure
Elevation: 22
Gamma: 10
Y: Fluid Pressure
Elevation: 22
Gamma: 10

Initial Water Tables

Initial Water Table 1

Max. negative head: 5
Coordinates
Coordinate: (-0.5, 22) m
Coordinate: (108, 22) m
Coordinate: (113, 20.5) m
Coordinate: (114.5, 19.5) m
Coordinate: (121.5, 15.5) m

Coordinate: (124.5, 2) m
Coordinate: (127, 0) m
Coordinate: (201, 0) m

Cyclic Number Functions

Cyclic # function

Model: Spline Data Point Function
Function: Cyclic Number vs. Shear Stress Ratio
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 1000
Data Points: Shear Stress Ratio, Cyclic Number
Data Point: (0.11, 1000)
Data Point: (0.15, 100)
Data Point: (0.2, 10)
Data Point: (0.3, 1)
Estimation Properties
Sample Material: Loose Sand

Pore Pressure Functions

PWP function

Model: Spline Data Point Function
Function: PWP Ratio vs. Cyclic Number Ratio N/NL
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 0
Data Points: Cyclic Number Ratio N/NL, PWP Ratio
Data Point: (0, 0)
Data Point: (0.05, 0.075089716)
Data Point: (0.1, 0.12368875)
Data Point: (0.15, 0.16607762)
Data Point: (0.2, 0.20519259)
Data Point: (0.25, 0.24231186)
Data Point: (0.3, 0.2781656)
Data Point: (0.35, 0.31324362)
Data Point: (0.4, 0.3479165)
Data Point: (0.45, 0.38249492)
Data Point: (0.5, 0.41726502)
Data Point: (0.55, 0.45251442)
Data Point: (0.6, 0.48855688)
Data Point: (0.65, 0.52576183)
Data Point: (0.7, 0.56459752)
Data Point: (0.75, 0.60570358)
Data Point: (0.8, 0.65003056)
Data Point: (0.85, 0.69915001)
Data Point: (0.9, 0.75610124)
Data Point: (0.95, 0.82872015)
Data Point: (1, 1)
Estimation Properties
N-Exponent: 0.7

10 Stability analysis with earthquake Dynamic Analysis

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Stability analysis with earthquake Dynamic Analysis

Description: Stability analysis steady state seepage with earthquake Dynamic Analysis state-Shauger dam.
Kind: QUAKE/W
Parent: 4-Earthquake initial case
Method: Equivalent Linear Dynamic
Settings
- Initial Stress: Parent Analysis
- Initial PWP: Parent Analysis
- Exclude cumulative values: No

Control
- Coefficient of Equivalent Shear Stress: 0.65
- Coefficient of Equivalent Shear Strain: 0.5
- Equivalent Number of Cycles: 10
- Liquefaction Stress Limit: 0

Convergence
- Maximum Number of Iterations: 10
- Displacement Norm Tolerance: 1 %
- Equation Solver: Parallel Direct
- RileyDamping: 3

Time
- Starting Time: 0 days
- Duration: 10 days
- # of Steps: 500
- Save Steps Every: 10
- Steps Generated via Earthquake Records: Yes

Materials

Core

Model: Linear Elastic
Stress Strain
- Unit Weight: 19.5 kN/m³
- Poisson's Ratio: 0.334
- Damping Ratio: 0.1
- Pore Water Pressure Function: PWP function
- Cyclic Function: Cyclic # function
- G Modulus: 5000 kPa

Shell

Model: Linear Elastic
Stress Strain
- Unit Weight: 20 kN/m³
- Poisson's Ratio: 0.334
- Damping Ratio: 0.1
- Pore Water Pressure Function: PWP function
- Cyclic Function: Cyclic # function
- G Modulus: 5000 kPa

Filter

Model: Linear Elastic
Stress Strain
Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Foundation (Quaternary) for instu analysis

Model: Linear Elastic
Stress Strain
Unit Weight: 19 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Rock bed for instu analysis

Model: Linear Elastic
Stress Strain
Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334
Damping Ratio: 0.1
Pore Water Pressure Function: PWP function
Cyclic Function: Cyclic # function
G Modulus: 5000 kPa

Boundary Conditions

Fixed Y

X: 0
Y: Y-Displacement 0

Fixed X/Y

X: X-Displacement 0 Y: Y-Displacement 0

Earthquake Records

Horizontal Earthquake Records

Description: Example.acc
Use Baseline Correction: No
Modified Peak Acc.: 0.57401229
Modified Duration: 1

Records

	Time (days)	Acc (m/days²)
1	0.02	-0.0014275517
284	5.68	0.13062099

Initial Water Tables

Initial Water Table 1

Max. negative head: 5

Coordinates
Coordinate: (-0.5, 22) m
Coordinate: (108, 22) m
Coordinate: (113, 20.5) m
Coordinate: (114.5, 19.5) m
Coordinate: (121.5, 15.5) m
Coordinate: (124.5, 2) m
Coordinate: (127, 0) m
Coordinate: (201, 0) m

Cyclic Number Functions

Cyclic # function

Model: Spline Data Point Function
Function: Cyclic Number vs. Shear Stress Ratio
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 1000
Data Points: Shear Stress Ratio, Cyclic Number
Data Point: (0.11, 1000)
Data Point: (0.15, 100)
Data Point: (0.2, 10)
Data Point: (0.3, 1)
Estimation Properties
Sample Material: Loose Sand

Pore Pressure Functions

PWP function

Model: Spline Data Point Function
Function: PWP Ratio vs. Cyclic Number Ratio N/NL
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 0
Data Points: Cyclic Number Ratio N/NL, PWP Ratio
Data Point: (0, 0)
Data Point: (0.05, 0.075089716)
Estimation Properties
N-Exponent: 0.7

11 <u>Load/Deformation</u>

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Load/Deformation

Kind: SIGMA/W
Parent: Stability analysis with earthquake Dynamic Analysis
Method: Load/Deformation
Settings
Initial Stress: Parent Analysis
Initial PWP: (none)
Exclude cumulative values: No
Control

Apply Body Force in All Steps: No
Use Constant Stress: Yes
Adjust Fill: No
Convergence
Maximum Number of Iterations: 10
Displacement Norm Tolerance: 1 %
Equation Solver: Parallel Direct
Time
Starting Time: 10 days
Duration: 1 days
# of Steps: 1
Save Steps Every: 1
Use Adaptive Time Stepping: No

Materials

Core

Model: Linear Elastic
Stress Strain
Young's Modulus (E): 10000 kPa
Unit Weight: 19.5 kN/m³
Poisson's Ratio: 0.334

Shell

Model: Linear Elastic
Stress Strain
Young's Modulus (E): 10000 kPa
Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334

Filter

Model: Linear Elastic
Stress Strain
Young's Modulus (E): 10000 kPa
Unit Weight: 20 kN/m³
Poisson's Ratio: 0.334

Foundation (Quaternary) loading phase

Model: Elastic-Plastic
Stress Strain
Total E-Modulus Function: Foundation stiffness
Poisson's Ratio: 0.49
Total Cohesion Function: undraind strength C
Phi: 0 °
Unit Weight: 19 kN/m³
Dilation Angle: 0 °

Rock bed loading phase

Model: Elastic-Plastic
Stress Strain
Total E-Modulus Function: Rock bed stiffness
Poisson's Ratio: 0.49
Total Cohesion Function: Rock bed undraind strength C
Phi: 0 °

Unit Weight: 20 kN/m³
Dilation Angle: 0 °

Boundary Conditions

Fixed X

X: X-Displacement 0
Y: 0

Fixed X/Y

X: X-Displacement 0
Y: Y-Displacement 0

Reservoir Pressure

X: Fluid Pressure
Elevation: 22
Gamma: 10
Y: Fluid Pressure
Elevation: 22
Gamma: 10

Total C Functions

undraind strength C

Model: Spline Data Point Function
Function: Total Cohesion vs. Y-Total Stress
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 0
Data Points: Y-Total Stress (kPa), Total Cohesion (kPa)
Data Point: (-3, 0)
Data Point: (0, 0)

Rock bed undraind strength C

Model: Spline Data Point Function
Function: Total Cohesion vs. Y-Total Stress
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 4000
Data Points: Y-Total Stress (kPa), Total Cohesion (kPa)
Data Point: (-23, 5000)
Data Point: (-3, 4000)

Total E-Modulus Functions

Foundation stiffness

Model: Spline Data Point Function
Function: Total E-Modulus vs. Y-Total Stress
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 5000
Data Points: Y-Total Stress (kPa), Total E-Modulus (kPa)
Data Point: (-3, 6000)
Data Point: (0, 5000)
Estimation Properties

Depth: 1 m
K-Modifier: 1
N-Exponent: 0.5
K0: 1

Rock bed stiffness

Model: Spline Data Point Function
Function: Total E-Modulus vs. Y-Total Stress
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 6000
Data Points: Y-Total Stress (kPa), Total E-Modulus (kPa)
Data Point: (-23, 12000)
Data Point: (-3, 6000)
Estimation Properties
Depth: 1 m
K-Modifier: 1
N-Exponent: 0.5
K0: 1

12 Slope Stability D/S with earthquake

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Slope Stability

Kind: SLOPE/W
Parent: Stability analysis with earthquake Dynamic Analysis-Shauger dam (2)
Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: Parent Analysis
SlipSurface
Direction of movement: Left to Right
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 m
Minimum Slice Width: 0.1 m
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1

Materials

Core

Model: Mohr-Coulomb
Unit Weight: 19.5 kN/m³
Cohesion: 70 kPa
Phi: 0 °
Phi-B: 0 °

Shell

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 20 kPa
Phi: 35 °
Phi-B: 0 °

Filter

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 0 kPa
Phi: 33 °
Phi-B: 0 °

Foundation (Quaternary)

Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Phi-B: 0 °

Rock bed

Model: Mohr-Coulomb
Unit Weight: 2 kN/m³
Cohesion: 4000 kPa
Phi: 0 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (98.874157, 18.681512) m
Left-Zone Right Coordinate: (172.125, 5) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (173.94465, 4.2059695) m
Right-Zone Right Coordinate: (215, 0) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)

1	108	1.203	(181.25, 10.3)	10.553	(172.125, 5)	(183.545, 0)

13 Slope Stability U/S Dynamic analysis

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Slope Stability U/S Dynamic analysis

Kind: SLOPE/W
Parent: Stability analysis with earthquake Dynamic Analysis-Shauger dam (2)
Method: Bishop, Ordinary and Janbu
Settings
  PWP Conditions Source: Parent Analysis
SlipSurface
  Direction of movement: Right to Left
  Allow Passive Mode: No
  Slip Surface Option: Entry and Exit
  Critical slip surfaces saved: 1
  Optimize Critical Slip Surface Location: No
  Tension Crack
    Tension Crack Option: (none)
FOS Distribution
  FOS Calculation Option: Constant
Advanced
  Number of Slices: 30
  Optimization Tolerance: 0.01
  Minimum Slip Surface Depth: 0.1 m
  Minimum Slice Width: 0.1 m
  Optimization Maximum Iterations: 2000
  Optimization Convergence Tolerance: 1e-007
  Starting Optimization Points: 8
  Ending Optimization Points: 16
  Complete Passes per Insertion: 1

Materials

Core

Model: Mohr-Coulomb
Unit Weight: 19.5 kN/m³
Cohesion: 70 kPa
Phi: 0 °
Phi-B: 0 °

Shell

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 20 kPa
Phi: 35 °
Phi-B: 0 °

Filter

Model: Mohr-Coulomb

Unit Weight: 20 kN/m³
Cohesion: 0 kPa
Phi: 33 °
Phi-B: 0 °

Foundation (Quaternary) for instu analysis

Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Phi-B: 0 °

Rock bed for instu analysis

Model: Mohr-Coulomb
Unit Weight: 2 kN/m³
Cohesion: 4000 kPa
Phi: 0 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (2.39521, 0) m
Left-Zone Right Coordinate: (56.5, 4.6153847) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (62.36743, 6.7818204) m
Right-Zone Right Coordinate: (132, 20.25) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	83	2.824	(54.451, 27.792)	29.838	(79.767, 12)	(43.5924, 0)

14 Steady-State Seepage dynamic analysis

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Steady-State Seepage

Kind: SEEP/W
Parent: Stability analysis with earthquake Dynamic Analysis-Shauger dam (2)
Method: Steady-State
Settings
Include Air Flow: No
Control
Apply Runoff: Yes
Convergence
Maximum Number of Iterations: 50
Tolerance: 0.1
Maximum Change in K: 1
Rate of Change in K: 1.1
Minimum Change in K: 0.0001
Equation Solver: Parallel Direct
Potential Seepage Max # of Reviews: 10
Time
Starting Time: 10 days
Duration: 10 days
Ending Time: 20 days

Materials

Core

Model: Saturated / Unsaturated
Hydraulic
K-Function: core
K-Ratio: 1
K-Direction: 0 °

Shell

Model: Saturated / Unsaturated
Hydraulic
K-Function: shell
K-Ratio: 1
K-Direction: 0 °

Filter

Model: Saturated / Unsaturated
Hydraulic
K-Function: filter
K-Ratio: 1
K-Direction: 0 °

Foundation (Quaternary) loading phase

Model: (none)

Rock bed loading phase

Model: Saturated / Unsaturated
Hydraulic
K-Function: rock bed
K-Ratio: 1
K-Direction: 0 °

Boundary Conditions

Potential Seepage Face

Review: true
Type: Total Flux (Q) 0

Zero potential line

Type: Pressure Head 0

Researvoire head=22m

Type: Head (H) 22

Flux Sections

Flux Section 1

Coordinates
Coordinate: (119, -22.5) m
Coordinate: (117, 36) m

K Functions

core

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 3e-009
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (2, 3e-009)
Data Point: (100, 3e-009)
Estimation Properties
Hydraulic K Sat: 0 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0 m^3/m^3

shell

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 1e-006
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (2, 1e-006)
Data Point: (100, 1e-006)
Estimation Properties
Hydraulic K Sat: 0 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0 m^3/m^3

filter

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 0.0005
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (2, 0.0005)
Data Point: (100, 0.0005)
Estimation Properties
Hydraulic K Sat: 0 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0 m³/m³

rock bed

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 5e-008
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (2, 5e-008)
Data Point: (100, 5e-008)
Estimation Properties
Hydraulic K Sat: 0 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0 m³/m³

15 Instataneous drawdown transient Seepage 7days

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

5-Instataneous drawdown transient Seepage 7days

Description: long term Instataneous drawdown transient Seepage Shauger dam 7days reservoir level drops from 22m to 0m.
Kind: SEEP/W
Parent: Steady-State Seepage Shauger dam
Method: Transient
Settings
Initial PWP: Parent Analysis

Include Air Flow: No
Exclude cumulative values: Yes
Control
Apply Runoff: Yes
Convergence
Maximum Number of Iterations: 50
Tolerance: 0.1
Maximum Change in K: 1
Rate of Change in K: 1.1
Minimum Change in K: 0.0001
Equation Solver: Parallel Direct
Potential Seepage Max # of Reviews: 10
Time
Starting Time: 0 days
Duration: 7 days
# of Steps: 5
Step Generation Method: Exponential
Initial Increment Size: 0.25 days
Save Steps Every: 1
Use Adaptive Time Stepping: No

Materials

Core

Model: Saturated Only
Hydraulic
K-Sat: 3e-009 m/days
Volumetric Water Content: 0.4 m^3/m^3
Mv: 0.001 /kPa
K-Ratio: 1
K-Direction: 0 °

Shell

Model: Saturated Only
Hydraulic
K-Sat: 1e-006 m/days
Volumetric Water Content: 0.4 m^3/m^3
Mv: 0.001 /kPa
K-Ratio: 1
K-Direction: 0 °

Filter

Model: Saturated Only
Hydraulic
K-Sat: 0.0005 m/days
Volumetric Water Content: 0.4 m^3/m^3
Mv: 0.001 /kPa
K-Ratio: 1
K-Direction: 0 °

Foundation (Quaternary) for instu analysis

Model: Saturated Only
Hydraulic
K-Sat: 2.5e-007 m/days

Volumetric Water Content: 0.4 m³/m³
Mv: 0.001 /kPa
K-Ratio: 1
K-Direction: 0 °

Rock bed for instu analysis

Model: Saturated Only
Hydraulic
K-Sat: 5e-008 m/days
Volumetric Water Content: 0.4 m³/m³
Mv: 0.001 /kPa
K-Ratio: 1
K-Direction: 0 °

Boundary Conditions

drawdown level

Type: Head (H) 0

upstream face

Review: true
Type: Total Flux (Q) 0

Flux Sections

Flux Section 1

Coordinates
Coordinate: (118.5, -25) m
Coordinate: (118, 39.5) m

16 Slope Stability U/S in istantaneous

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Slope Stability U/S

Description: Slope Stability U/S with instantaneous drawdown 7 days transient case.
Kind: SLOPE/W
Parent: 5-Instataneous drawdown transient Seepage 7days
Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: Parent Analysis
SlipSurface
Direction of movement: Right to Left
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack

Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 m
Minimum Slice Width: 0.1 m
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1

Materials

Core

Model: Mohr-Coulomb
Unit Weight: 19.5 kN/m³
Cohesion: 70 kPa
Phi: 0 °
Phi-B: 0 °

Shell

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 20 kPa
Phi: 35 °
Phi-B: 0 °

Filter

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 0 kPa
Phi: 33 °
Phi-B: 0 °

Foundation (Quaternary) for instu analysis

Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Phi-B: 0 °

Rock bed for instu analysis

Model: Mohr-Coulomb
Unit Weight: 2 kN/m³
Cohesion: 4000 kPa
Phi: 0 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (4.4898, 0) m
Left-Zone Right Coordinate: (64.3125, 7.5) m
Left-Zone Increment: 4
Right Projection: Range

Right-Zone Left Coordinate: (66.40449, 8.2724269) m
Right-Zone Right Coordinate: (143.5, 16.142857) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) mRight Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	91	0.221	(-49.44, 501.983)	509.57	(124.374, 22.9736)	(49.9683, 2.20367)

Slices of Slip Surface: 91

17 Slow drawdown transient 7days

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

6-Slow drawdown transient Seepage

Description: Slow drawdown transient Seepage Shauger dam for transient case
Kind: SEEP/W
Parent: Steady-State Seepage Shauger dam
Method: Transient
Settings
- Initial PWP: Parent Analysis
- Include Air Flow: No
- Exclude cumulative values: Yes

Control
- Apply Runoff: Yes

Convergence
- Maximum Number of Iterations: 50
- Tolerance: 0.1
- Maximum Change in K: 1
- Rate of Change in K: 1.1
- Minimum Change in K: 0.0001
- Equation Solver: Parallel Direct
- Potential Seepage Max # of Reviews: 10

Time
- Starting Time: 0 days
- Duration: 7 days
- # of Steps: 5
- Step Generation Method: Exponential
- Initial Increment Size: 0.25 days
- Save Steps Every: 1
- Use Adaptive Time Stepping: No

Materials

Core

Model: Saturated / Unsaturated
Hydraulic
K-Function: core conductivity function
Vol. WC. Function: core
K-Ratio: 1
K-Direction: 0 °

Shell

Model: Saturated / Unsaturated
Hydraulic
K-Function: shell conductivity function
Vol. WC. Function: shell
K-Ratio: 1
K-Direction: 0 °

Filter

Model: Saturated / Unsaturated
Hydraulic
K-Function: filter conductivity function
Vol. WC. Function: filter
K-Ratio: 1
K-Direction: 0 °

Foundation (Quaternary) for instu analysis

Model: Saturated / Unsaturated
Hydraulic
K-Function: foundation conductivity function
Vol. WC. Function: foundation
K-Ratio: 1
K-Direction: 0 °

Rock bed for instu analysis

Model: Saturated / Unsaturated
Hydraulic
K-Function: rock bed conductivity function
Vol. WC. Function: bed rock
K-Ratio: 1
K-Direction: 0 °

Boundary Conditions

slow drawdown

Function: Drawdown curve
Review: true
Type: H (Q=0)

Flux Sections

Flux Section 1

Coordinates
Coordinate: (118.5, -25) m
Coordinate: (118, 39.5) m

K Functions

core conductivity function

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 0.0002592
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Estimation Properties
Volume Water Content Function: core
Hydraulic K Sat: 0.0002592 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0.05 m^3/m^3

shell conductivity function

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 0.0864
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Estimation Properties
Volume Water Content Function: shell
Hydraulic K Sat: 0.0864 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0.05 m^3/m^3

filter conductivity function

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 43.2
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (0.01, 43.2)
Estimation Properties
Volume Water Content Function: filter
Hydraulic K Sat: 43.2 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0.05 m^3/m^3

foundation conductivity function

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 0.0216
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (0.01, 0.0216)
Estimation Properties
Volume Water Content Function: foundation
Hydraulic K Sat: 0.0216 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0.05 m^3/m^3

rock bed conductivity function

Model: Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
K-Saturation: 0.00432
Data Points: Matric Suction (kPa), X-Conductivity (m/days)
Data Point: (0.01, 0.00432)
Estimation Properties
Volume Water Content Function: bed rock
Hydraulic K Sat: 0.00432 m/days
Hyd. K-Function Estimation Method: Van Genuchten Function
Maximum: 1000
Minimum: 0.01
Num. Points: 20
Residual Water Content: 0.05 m^3/m^3

Hydraulic Boundary Functions

Drawdown curve

Model: Spline Data Point Function
Function: Total Head vs. Time
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Y-Intercept: 22
Data Points: Time (days), Total Head (m)
Data Point: (0, 22)
Data Point: (7, 0)

Vol. Water Content Functions

core

Model: Data Point Function
Function: Vol. Water Content vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Mv: 0.0001 /kPa
Saturated Water Content: 0 m^3/m^3
Porosity: 0.40000083
Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.39999949)
Estimation Properties
Vol. WC Estimation Method: Sample functions
Sample Material: Clay
Saturated Water Content: 0.4 m^3/m^3
Liquid Limit: 0 %
Diameter at 10% passing: 0
Diameter at 60% passing: 0
Maximum: 1000
Minimum: 0.01
Num. Points: 20

shell

Model: Data Point Function
Function: Vol. Water Content vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Mv: 0.0001 /kPa
Saturated Water Content: 0 m^3/m^3
Porosity: 0.39999594
Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)
Data Point: (0.01, 0.39999494)
Estimation Properties
Vol. WC Estimation Method: Sample functions
Sample Material: Silty Sand
Saturated Water Content: 0.4 m^3/m^3
Liquid Limit: 0 %
Diameter at 10% passing: 0
Diameter at 60% passing: 0
Maximum: 1000
Minimum: 0.01
Num. Points: 20

filter

Model: Data Point Function
Function: Vol. Water Content vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Mv: 0.0001 /kPa
Saturated Water Content: 0 m^3/m^3
Porosity: 0.39999526
Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)
Data Point: (0.01, 0.39999426)
Estimation Properties
Vol. WC Estimation Method: Sample functions
Sample Material: Sand
Saturated Water Content: 0.4 m^3/m^3
Liquid Limit: 0 %
Diameter at 10% passing: 0
Diameter at 60% passing: 0
Maximum: 1000
Minimum: 0.01
Num. Points: 20

foundation

Model: Data Point Function
Function: Vol. Water Content vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Mv: 0.0001 /kPa
Saturated Water Content: 0 m³/m³
Porosity: 0.39999594
Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)
Data Point: (0.01, 0.39999494)
Estimation Properties
Vol. WC Estimation Method: Sample functions
Sample Material: Silty Sand
Saturated Water Content: 0.4 m³/m³
Liquid Limit: 0 %
Diameter at 10% passing: 0
Diameter at 60% passing: 0
Maximum: 1000
Minimum: 0.01
Num. Points: 20

bed rock

Model: Data Point Function
Function: Vol. Water Content vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Mv: 0.0001 /kPa
Saturated Water Content: 0 m³/m³
Porosity: 0.40000083
Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)
Data Point: (0.01, 0.39999949)
Estimation Properties
Vol. WC Estimation Method: Sample functions
Sample Material: Clay
Saturated Water Content: 0.4 m³/m³
Liquid Limit: 0 %
Diameter at 10% passing: 0
Diameter at 60% passing: 0
Maximum: 1000
Minimum: 0.01
Num. Points: 20

18 Slope Stability U/S slow drawdown 7days

Report generated using GeoStudio 2012, GEO-SLOPE International Ltd.

Analysis Settings

Slope StabilityU/S slow drawdown

Description: Slope StabilityU/S slow drawdown
Kind: SLOPE/W
Parent: 6-Slow drawdown transient Seepage

Method: Bishop, Ordinary and Janbu
Settings
PWP Conditions Source: Parent Analysis
SlipSurface
Direction of movement: Right to Left
Allow Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 m
Minimum Slice Width: 0.1 m
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1

Materials

Core

Model: Mohr-Coulomb
Unit Weight: 19.5 kN/m³
Cohesion: 70 kPa
Phi: 0 °
Phi-B: 0 °

Shell

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 20 kPa
Phi: 35 °
Phi-B: 0 °

Filter

Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion: 0 kPa
Phi: 33 °
Phi-B: 0 °

Foundation (Quaternary) for instu analysis

Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Phi: 35 °
Phi-B: 0 °

Rock bed for instu analysis

Model: Mohr-Coulomb
Unit Weight: 2 kN/m³
Cohesion: 4000 kPa
Phi: 0 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (2.78912, 0) m
Left-Zone Right Coordinate: (67, 8.4923077) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (69.0045, 9.2324307) m
Right-Zone Right Coordinate: (143, 16.321429) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m
Right Coordinate: (220, 0) m

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	67	1.455	(54.556, 111.024)	112.622	(124.652, 22.8745)	(35.6534, 0)

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