`

SEEPAGE AND SLOPE STABILITY ANALYSIS OF EARTH FILL DAM USING NUMERICAL MODELING, THE CASE STUDY OF DORA DAM, NORTHERN TIGRAY, ETHIOPIA

MSc. THESIS IN DAM ENGINEERING

BY

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A THESIS SUBMITTED TO THE SCHOOL OF WATER RESOURCE ENGINEERING, HAWASSA UNIVERSITY INSTITUTE OF TECHNOLGY, SCHOOL OF GRADUATE STUDIES HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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EXAMINERS' APPROVAL SHEET-1

(Submission Sheet-2)

We, the undersigned, members of the Board of Examiners of the final open defense by **BEYENE ENDALEW ABRHA** have read and evaluated his/her thesis entitled" **seepage and slope stability analysis earth fill dam using numerical model(case study Dora Dam)**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree master of science in dam engineering

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SGS Approval	**Signature**	**Date**

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DECLARATION

I hereby declare that this M.Sc. thesis is my original work and has not been presented for a degree in any other University and all sources of material used for this paper.

Name: Beyene Endalew Signature ----------------

Place: Hawassa University

Date of Submission --------------

LIST OF ACRONYMS

ASHTO	American society highway and transportation office
ICOLD	INTERNATIONAL COMMISSION ON LARGE DAM
USACE	united states army corps of engineers
USBR	united state bureau of reclamation
VWC	volumetric water conten
WWCE	water works construction enterprise
TWRB	Tigray water resource bureau
U/S	upstream
D/S	downstream
M.M.D	maximum dry density
O.M.C	Optimum moisture content

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ABSTRACT

Embankment dam failure may occur due to different reasons such as structural instability conditions, hydraulic conditions, seepage through the dam body and foundation. The determination of factor of safety for the dam slope stability, under different cases of operations, is vital to ascertain the dam overall safety. In this thesis, Finite Element modeling is employed for simulating seepage and slope stability analysis of earth dam problems using Geo Studio software. The model is verified, and then it is employed to analyze seepage and stability of Dora Dam. Three different cases of operation are considered; end of construction before filling the reservoir, steady state seepage and rapid drawdown of the reservoir. Four major analytical methods there used to assess the stability of the dam side slopes; Morgenstern-Price, Bishop, Janbu, and Spencer. The results of the analysis presented in this paper confirm the safety of Dora dam against combined seepage and slope stability under all cases of operation. The result of stability analysis performed showed that all parts of the dam are safe within the prescribed range of factors of safety for the possible all loading and operation cases. But all the critical slip surfaces pass through the dam shell which indicates that this zone represents the weaker zone and it needs spatial attention. The result of seep/w analysis showed that 0.01899853 m^3*/sec seepage loss during steady state case and 0.019846008* m^3*/se seepage loss through the dam during transient analysis. The factory of safety of 1.795 for downstream slope during steady state, and 1.433 during sudden draw down are among the results found. The analysis result shows that the water tightness of the dam, stability of the embankment slope has in this thesis performed in a better way for some loading conditions like downstream slope at steady state, construction stage.*

Key words*: - seepage and slope stability analysis, numerical modeling, limit equilibrium method, finite element method, geo-slope software.*

1. INTRODUCTION

1.1 BACKGROUND

Dams are constructed regarding numerous purposes just like overflow command, course-plotting, water solutions, sport, power creation and as well as for irrigation purposes. Soil dams have invariably been associated with seepage while they impound water it. Water looks for routes regarding the very least opposition from the dam and it is groundwork. Seepage could be the main problem also it travels from the dam product looked after transporting dam supplies. Seepage must be handled just to save the actual erosion regarding embankment or even their groundwork.

Embankment dam are common in different other style of dams on account of numerous cause just like the employment of everyday development technologies method while using cheap fresh land product as well as subsurface supplies, absolutely no require of the distinct area condition. One of several important factor producing failing regarding embankment dam by simply seepage and hence seepage analysis regarding embankment dam is usually larger significance.

It is well known fact that excessive seepage in any type of a dam is one the root cause to destabilize the dam structure and thereby bring economic havoc, Arshad et.al. (2014). consequently, pragmatic efforts are employed to carryout in depth study of the seepage analysis through and beneath the body of a dam. Generally, designers by employing different techniques augment safety of a dam and mark errors in computation due to maintenance of water storage, especially by focusing on hydro structure of the dam. The seepage control of any dam may be analyzed by virtue of various available methods (UACE, 1986).

Porous environment can be investigated by using numerical techniques framed in the form of a software solution, i.e. computer program Kamanbedast et al. (2012).

The seepage analysis of a dam is essential for evaluation of its safety and stability especially by using numerical techniques; by doing this one can analyze seepage field and make its comparison under different conditions.

In the effect of core and other assorted factors can also be investigated Quanshu et al. (2010). Much computer software has come in general use, and any hard computations and simulation can be carried out through them by giving them appropriate inputs and data. This results in less error frequency and more detailed analysis when compared with field observations. The numerical model SEEP/W can be employed to carry out simulation of seepage and phreatic surface.

The SEEP/W and SLOPE/W computer software can be used to estimate the flow of water through the soil and failure due to slope analysis of Gopal et al. (2014). This software solves the underground water problems for stable, unstable, wet and dry conditions. This software not only have the superiority to the graphic method and manual calculations, but also regarding the time, we can gain good results. This software has many applications which helps designers in best designing of dams and analyzing the weak or strength points of dams and also designing of the construction which dealing with the seepage problems.

The FEM is also being successfully applied to analyze the Saturated-Unsaturated seepage through porous media. The Seepage Analysis of embankment Dam in largest the world in terms of Embankment Volume, was carried out using Saturated-Unsaturated Seepage Finite Element Method Naushahi, and Parvez, (1992).

Due to transient characteristic of the system, it provides a window of opportunity for the research to analyze such problems; for instance migration of a wetting front and dissipation of excessive pore water pressures Geo-Slope International (2007). In view of all above facts, the present research work was designed to model seepage and slope stability analysis of an earthen dam by using Geo-studio.

1.2 Statement of the problem

Dams are constructed for various purpose like the one flood control, navigation, water sources, recreation, power generation and irrigation, but earth dams have always been associated with seepage as they impound water (Novak, 2007).

Embankment dam are more common than any other type of dams because of various reason like the use of ordinary technology as construction method, utilizing cheap raw soil material and subsurface materials, no need of a particular valley shape. However, the disadvantage of embankment dam is highly affected by seepage problem (ICOLD, 1995). As see in figure 3.3 Dora embankment dam also seepage is major problem. To minimize the amount of water seeping through the body of dam and foundation good construction control in dams particularly embankment dams is vital for the increasing future safety of the dam. Quality control activities of embankment dams using geo-studio software was not a common methodology in the past, and this is one of the problem statements but in the future, it will be one of the best alternative methods for analysis safety of dams.

Thus, slope failure of an earthen dam can causes catastrophic failure for human beings and properties downstream of the dam. The long-term solution it Starts construction of one dam with its infrastructure around the proposed Tabia. Moreover, the dam is at the verge to complete especially the headwork of the dam meanwhile, the dam has a seepage problem at the left abutment and at the central body of the dam. Therefore, to identify the seepage and slope problem of the dam a study of seepage analysis with SEEP/W and SLOPE/W Geo studio model of the dam is important.

1.3 Research questions

- ➢ Is Dora dam performing as expected in design?
- ➢ What is the main cause of seepage of Dora embankment dam?
- ➢ At what phase and in which zone (body of the dam, foundation, and both abutments) the dam shows seepage/ leakage first?

1.4 Objective of the study

The main objective of this research work is to investigate the seepage and slope stability behavior of Dora embankment dam.

1.3.1 Specific objectives

- To check factor of safety of the dam that of slope and seepage analysis at different critical conditions.
- To quantify seepage through the main body of the dam and foundation
- To determine the pore water distribution for parent analysis of SLOPE/W /slope stability analysis

1.5 Scope of the study and limitation

This thesis work is concerned with seepage quantification, and slope stability analysis using SEEP/W and SLOPE/W only.

1.6 Significant of the study

The significant of the study is to checking the applicability of the Geo-studio on quantifying seepage and determination of slope stability on Dora dam. Geo-studio software was not a common methodology in the past, and this research is increase the construction evaluation of embankment dam using geo-studio software one-step forward especially during construction time.

2. LITERATURE REVIEW

2.1 Review on major causes of dam failure and their statistics

2.1.1 General about dams

There are numerous type of dam and the classified usually in terms of materials used for their construction and their form. Among these the Common types are homogeneous or zoned earth fills; rock fills with earth core or concrete face; and concrete dams that depend on gravity, arch, or buttress resistance.

Some dams are composites of various materials, including earth fill, rock fill, masonry, and concrete. A few have timber, asphaltic, or synthetic members. Topography, geology and availability of construction materials and technology are primary factors in weighing the comparative merits of dam types. Novak et al. (2007) give an initial broad classification of dams into two generic groups based on the principal construction material employed.

Table 2.1 types of dams and register statistics (ICOLD, 1988a, in NOVAK et al., 2007)

	Type	Total number of constructed large dams
Embankment dam	Earth fill and rock fill	82.90%
Concrete dam (including masonry)	Gravity	11.30%
	Arch	4.40%
	Buttress	1%
	Multiple arch	0.40%
Total large dams(ICOLD 1988)		36235

Large dams are defined by ICOLD as dams exceeding 15 m in height or, storage volume in excess of $1x10^6$ m^3 or a flood discharge capacity of over 2000 m^3 s-1.Based on this definition the case study dam in this research is categorized as a large dam with its height of 43 m.

2.1.2 Causes of dam failure and their statistics

There are varying statistics on causes of dam failure, for example statistics given by International Commission on Large Dams (ICOLD, 1995), US Army Corps of Engineers (USACE, 2003) and Novak et.al. (2007). Many attempts have been made at compiling and assessing statistics on dam failure. Main attempts on worldwide scale have been made by International Commission on Large Dams (ICOLD) in 1974, 1983 and 1995. ICOLD (1995) states that foundation problems are the most common causes of failure in concrete dams, with internal erosion and foundation shear strength each contributing for 21% of failure. In case of earth and rock fill dams, the most common cause of failures overtopping (31% as primary cause and 18% as secondary cause) followed by internal erosion in the body of the dam (15% as primary cause 13% as secondary cause),and in the foundation (12% as primary cause and 5% as secondary).

Table 2.**2**: statistics on causes of dam failure

Source	Over Toping	Foundation	Internal Erosion	Other
USACE(2006)	34%	30%	20%	6%
Novak et.al.(2009)	30-35%	No data	30-35%	No data
ICOLD	31%primary Cause 18% secondary cause	No data	27% primary cause 18% Secondary	No data

Source: Novak 4th edition (2007)

2.1.3 Embankment dams failure mechanisms and design practices

It is comprehensible that the degree of importance of different causes of dam failure varies with dam type. Dam design principles and considerations evolved with the identification of major causes of dam failure and the progressive understanding of their mechanisms. In addition, knowledge on causes of dam failure is crucial for dam safety evaluation, dam monitoring and rehabilitation decisions. The following paragraphs provide a summary on the major causes of dam failure and their mechanisms.

Earth fill embankments may be damaged by distortions at critical points. Differential settlement may be severe at steep abutments and at structural interfaces where effective compaction is difficult to obtain. At these locations, deformation of the fill may open dangerous paths of seepage.

For this reason, there have been many failures along outlet conduits. Although properly constructed embankments are able to accommodate substantial movement, they have relatively poor resistance to overflow; so their freeboard and associated spillway capacity must be determined conservatively.

In contrast, most concrete dams can withstand overtopping for at least several hours. The key to their safety may be the resistance of the foundation to impact of the spill. Essential criteria governing the structural competence of concrete dams are the margin of safety against overall structural stability (this includes safety against rotation and tipping of the dam; and translation and sliding of the dam body and natural rock foundation) in relation to all probable conditions of loading including empty reservoir condition. Moreover, there should not be overstress and material failure in the dam concrete and the rock foundation.

Arch dams can carry large loads, but their integrity depends inherently on strength of the abutments. Failure may be caused by rock deteriorations or by shearing under water pressures. Weakening of arch support also may be triggered by foundation erosion. Gravity dams are noted for durability because of their large masses, they can survive considerable weathering and site deficiencies. However, sometimes some have failed where foundation elements were susceptible to sliding. A few buttressed dams also have shown this tendency. Novak et al. (2007) identifies the following principal defect mechanisms and failure modes for embankment dams:

2.1.3.1 External erosion

External erosion is caused by flow over embankment (overtopping). The overtopping situation is occurred when (Costal, 1988).

1. Insufficient capacity of spillway design

2. Partly or fully blocked spillway

3. Losses of storage capacity of the dam

4. Huge water displacement due to earthquake

In case of excess rainfall, the upstream water level increases instantly. When this level exceeds the maximum drainage capacity of the dam, water started to flow over embankment. This over flowing water causes the breaching followed by slide at downstream slope of the embankment as a consequence of external erosion (Kjaernsli, 1992).When an intense rainfall occurred, the spillway was partly blocked by suspended particles causes the increase of water level higher than the estimated probable maximum flood (PMF) level and overtopping.

2.1.3.2 Internal erosion

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

2.1.3.3 Piping

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

2.1.3.4 Instability

The embankment, including its foundation, must be stable under construction and under all conditions of reservoir operation. Instability might occur when downstream slope too high and/or too steep in relation to shear strength of the shoulder material or when there is rapid drawdown of water level or because of failure of downstream foundation due to overstress of soft, weak horizons. Therefore, Face slopes must, be sufficiently flat to ensure internal and foundation stress remains within acceptable limits under different conditions of loading.

2.2 Embankment Zoning

Large embankment dams should be zoned to use as much material as possible from required excavation and borrow areas with the shortest haul distances and the least waste. Embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells seepage control, and stability. The slopes of an embankment dam may vary widely depending on the characteristics of the materials available for construction, foundation conditions, and the height of the dam as well (USBR, 2012). According to Novak et al, (2007) zoned embankment dams led to more economical structures where there are a variety of soils readily available. Major advantages of using zoned embankments are:

- Steeper slopes may be used, with consequent reduction in total volume of embankment material
- A wide variety of materials available on site may be used
- Maximum use can be made of material excavated from the foundation, spillway, outlet works, and other appurtenant structures.

2.3 Filter and transition zones

Transition filter zones are required to separate zones of different permeability and compressibility properties within the embankment. These can be particularly critical on the downstream side, where they will act both as filters to retain the migration of the base material and as drainage layers to avoid pressures building up in the downstream shoulder. During designing of these zones, it is important to consider that the gradation of adjacent zones should not migrated into the voids of adjoining zones, during either steady-state seepage or rapid drawdown seepage force. Besides, transition materials should also serve as self-healing of cracks of the core material. According to USACE, (1993) transition zones can be provided on both upstream and downstream of the core and should have a width not less than 3m.

2.4 Seepage Control Measures

All embankment dams are subjected to some seepage passing through, under, and around them. If uncontrolled, seepage may be detrimental to the stability of the structure because of excessive internal pore water pressures or by piping. Seepage should be effectively controlled to preclude structural damage or interference with normal operations.

2.5 Cut-off Trenches

Seepage under and round the flank of a dam must be controlled. This is achieved by the construction of a cut-off trench below the structure, continued as necessary on either flank. Modern embankment cut-off trenches are generally formed from wide trenches backfilled with rolled clay, if impervious strata lie at moderate depths.

2.6 Internal drainage

Seepage is always present within the body of any dam. Seepage flows and their resultant internal pressures must be directed and controlled. Internal drainage systems for this purpose are therefore an essential and critical feature of all modern dams. In embankments, drainage is affected by suitably located pervious zones leading to horizontal blanket drains or outlets at base level.

2.7 Erosion Control Measures

Upstream and downstream slopes, the toe area, projection areas of the abutments, approach and discharge channels, and areas adjacent to concrete structures should be protected against excessive erosion from wave action, surface runoff, and encroaching currents. Inadequate erosion protection can result in slope instability. Some common types of protection used are riprap, gabions, paving (concrete or asphalt), and appropriate vegetative cover.

Care must be taken to ensure that outlet or other facilities constructed through the dam do not permit unobstructed passage of seepage water along their perimeters with risk of soil migration and piping.

2.8 Embankment Overtopping Potential

All embankment dams should be evaluated for overtopping potential under the most extreme conditions expected for which the dam is determined to be a hazard to life or property. The maximum reservoir elevation determined for the design flood and expected wave run-ups are conditions that should be considered. However, a less severe storm with lower reservoir elevation but greater wave propagation may result in conditions that are more critical than those produced by the design flood. In general, overtopping of an embankment is not acceptable.

2.9 Limit Equilibrium Methods (LEM)

Limit equilibrium types of analyses for analyzing stability of earth slopes have been in use in geotechnical engineering. The idea of discretizing a potential sliding mass into vertical slices was introduced early in the 20 century. In 1916, Petterson (1955) presented the stability analysis of the Stigberg Quay in Gothenberg, Sweden where the slip surface was taken to be circular and the sliding mass was divided into slices. Later, Fellenius (1936) introduced the Ordinary or Swedish method of slices. In the mid-1950s Janbu (1954) and Bishop (1955) developed advances in the limit equilibrium method (SLOPE/W, 2007).

Limit equilibrium method of analysis for static slopes stability is still the most widely used tool to analyze the stability of embankment slope. It is mainly based on the assumption that failure occurs through sliding of mass along a slip surface (Rickard and Sitar, 2012). This method has a drawback that it does not consider stress strain relationship; it is approximate and requires a number of basic assumptions. However, it has also advantages that is quite general and can be applied to walls, slopes or foundations, or to any combination of these. In addition, it can be adapted for cases where the soil has layers with different properties or irregularly shaped boundaries (Krahn, 2003 and Atkinson, 2007).

Different techniques of applying limit equilibrium have been introduced so far. Ordinary or Fellenius, Bishop’s Simplified, Janbu’s Simplified, Spencer’s method and Morgenstern-Price’s are which are formulated based on limit equilibrium concept . All these methods are very similar and the main differences between these methods are the equation of statics included, equations of equilibriums are satisfied and inter slice force considered (SLOPE/W, 2007).

Method available in limit equilibrium stability analysis, equations of statics satisfied, summary of the interslice forces included are presented in table 2.4 below.

Table 2.3: features and limitation for limit equilibrium methods (SLOPE/W, 2007)

Method	Features and Limitation
Slope Stability Charts (Janbu,1968 Duncan et al, 1987)	- Accurate enough for many purposes. -Faster than detailed computer analysis.
Ordinary Method of Slices (Fellenius, 1927)	- Only for circular slip surfaces - Satisfies moment equilibrium. - Does not satisfy horizontal or vertical force equilibrium.
Bishop's Modified Method (Bishop, 1955)	-Only for circular slip surfaces. -Satisfies moment equilibrium. -Satisfies vertical force equilibrium. -Does not satisfy horizontal force equilibrium
Janbu's Generalized Procedure of Slices (Janbu, 1968)	-Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force locations to be varied. - More frequent numerical problems than some other methods.
Morgenstern and Price's Method (Morgenstern and Price, 1965)	-Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force orientations to be varied.

Spencer’s Method (Spencer,1967)	-Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Side forces are assumed to be parallel.

2.10 Finite element method

With the development of modern personal computers, finite element method has been increasingly applied in slope stability analysis. The finite element method (FEM) is powerful alternative approach the analysis of slope stability. Zaman, et al., (2000) studied that FEM is accurate, versatile and does not demand many assumption and it is also powerful in solving problems with irregular bodies and complex variation of flow lines. The advantage of finite element over limit equilibrium method is that no assumption needed in advance about the shape and location of the failure surface and this method gives information about the deformation at working stress level Griffiths and Lane, (1999); Chollada and Tanan, (2013). In application of finite element methods, stress-stain relationship to be considered in addition to force and moment equilibrium conditions and this is lead to much realistic result (SLOPE/W, 2007).

2.11 Loading Conditions for Embankment Dams

Variations of the loads acting on embankment slopes and variations of shear strengths with time result in changes in the factors of safety of slopes. As a result, it is often necessary to perform stability analyses corresponding to several different conditions, referring different stages in the life of the dam. For embankment dams, it is necessary to examine the stability of both the upstream and the downstream slopes for the most adverse loading condition presented in USACE (USACE, 2003). The major loading conditions that embankment dams should be evaluated are:

2.12 End of Construction

The end-of-construction loading condition is usually analyzed for embankments that include fine-grained soils, and are constructed on fine-grained saturated foundations that may develop excess pore pressures from the loading of the embankment. The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of the overlying layers USACE, (2003); Stematiu, (2006).

2.13 Steady-State Seepage

After a prolonged storage of reservoir water, water percolating through an embankment dam will establish a steady-state condition of seepage. The upper surface of seepage is called the phreatic line. It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most (USACE, 2003).

2.14 Rapid (or Sudden) Drawdown

Sudden drawdown condition is a classical scenario in slope stability, which arises when totally or partially submerged slopes experience a reduction of the external water level. Rapid drawdown conditions have been extensively analyzed in the field of dam engineering because reservoir water levels fluctuate widely due to operational reasons or emergency condition.

When the reservoir level behind an embankment dam is lowered, the stabilizing effect of the water pressure on the upstream slope lost and when the water level dropped quickly, the pore pressures in the slope do not have time to reach equilibrium with the new reservoir water level, so that the slope would be less stable. For analysis purposes, it is assumed that drawdown is very fast, and no drainage occurs in materials, which have low permeability. If drawdown occurs during or immediately after construction, the undrained shear strength used in the drawdown analysis is the same as the undrained shear strength that applies to the end of construction condition Duncan and Wright, (2005) and Fell et al, (2005).

3. MATERIALS AND METHODS

3.1 Description of Study Area

3.1.1 Location

Dora dam project is found in Southeast zone of Tigray region woreda Hintalo Wajrat Tabia Adi Keyh at site name Dora. Dora dam is located at GPS coordinate of 571990E, 1440455N (GPS location in UTM), and The road accessibility is 84 km road from Mekelle town to site is accessible by four wheel vehicle this includes 82km asphalt road to Adi Keyh and 2 km weathered road from Adi Keyh to project and as well as it is good catchment for the inflow of the dam

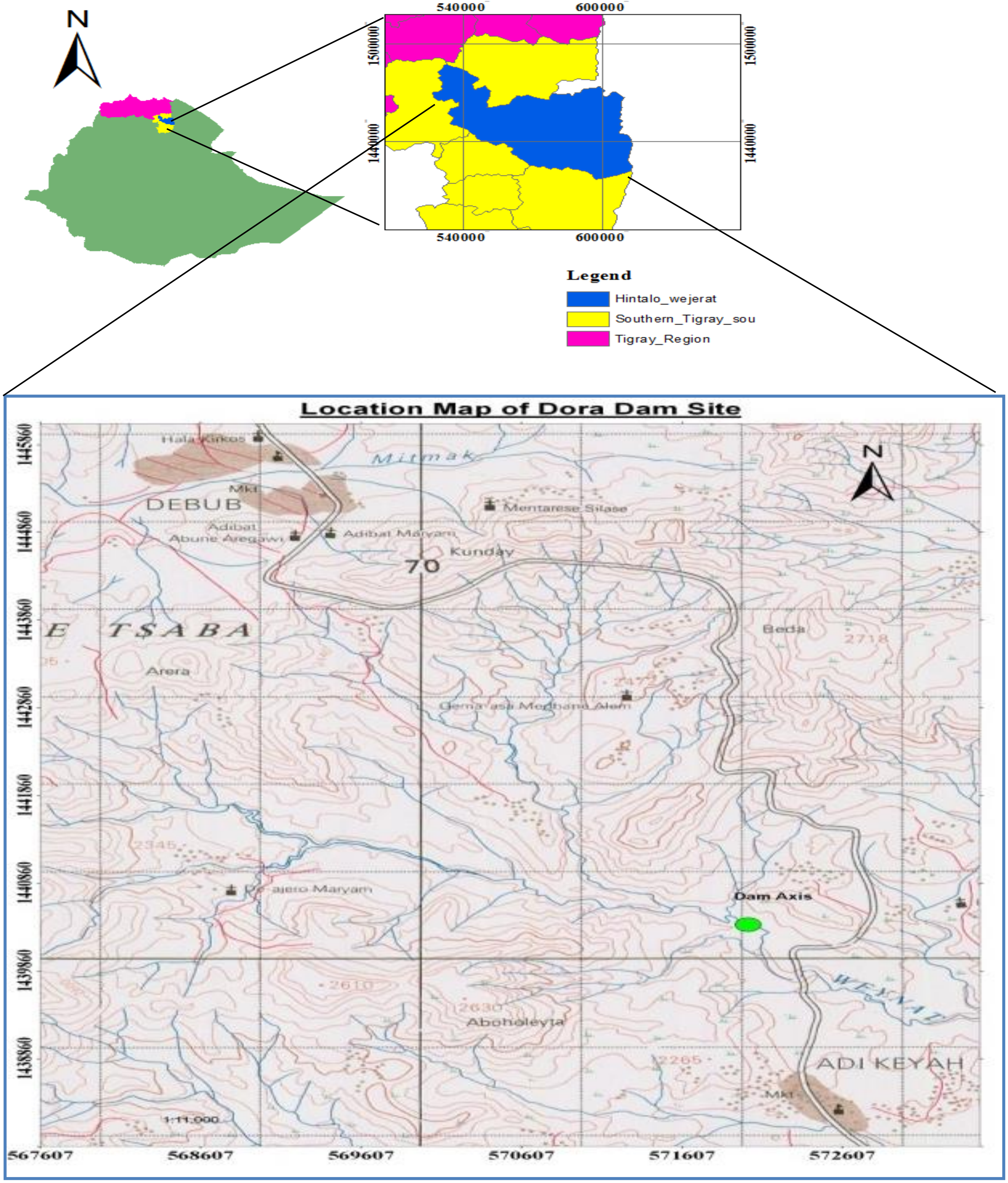

Figure 3.1: location map of Hintalo wejerat Adikey

3.1.2 Topography

The dominant topography in this watershed is sloping terrains having a slope range of 9-13. 89% and such topographic feature are commonly found at lower & middle part of the watershed as shown figure below. The steep landscapes are largely found at the middle, upper part of the catchment and the slope ranges 13.89-27.12% .The moderately steep topographies that are found at the middle, and some at lower of the watershed occupy 84.02% of the total area. The gentle sloping terrain covers 75.88% of the total area and is situated on middle & lower part of the watershed.

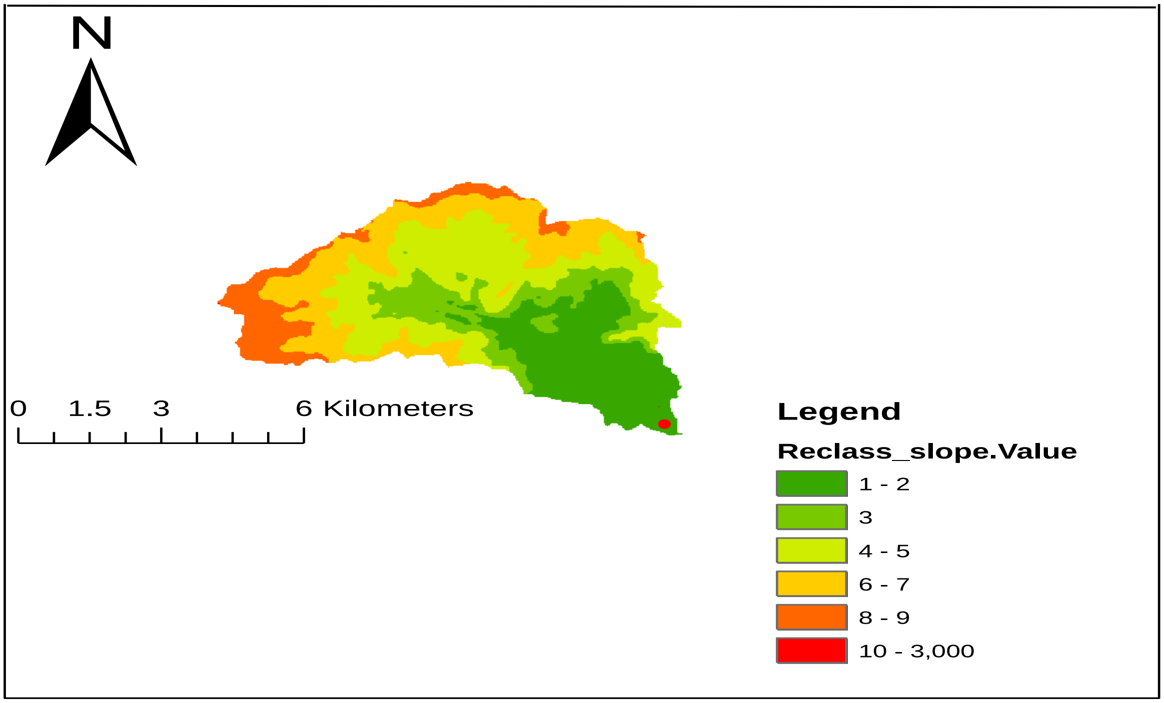

Figure 3.2: Dora Dam Catchment Area Slope

3.1.3 Some Components of Dora dam

Based on feasibility and detailed design of Dora Dam, the dam site is planned to harvest 17132955.6 m^3 of water from a catchment area of about 78.03 km^2 to introduce a modern irrigation practice downstream side of the dam (TWRB 2007). The reservoir of the project is shown in figure below.

Figure 3.3: Dora Dam Reservoir

Dora Dam, have about 43m height at the maximum site depth and crest width is adapted to be 7.0m, the side slopes have a 2.5H: 1V on the upstream side separated by 3m wide berms. The berms are also functionally required for various purposes. On downstream side, the side slopes are 2.0H: 1V above berm and 2.5H: 1V below berm. The geometry of the dam is shown in Figure 3.4 below.

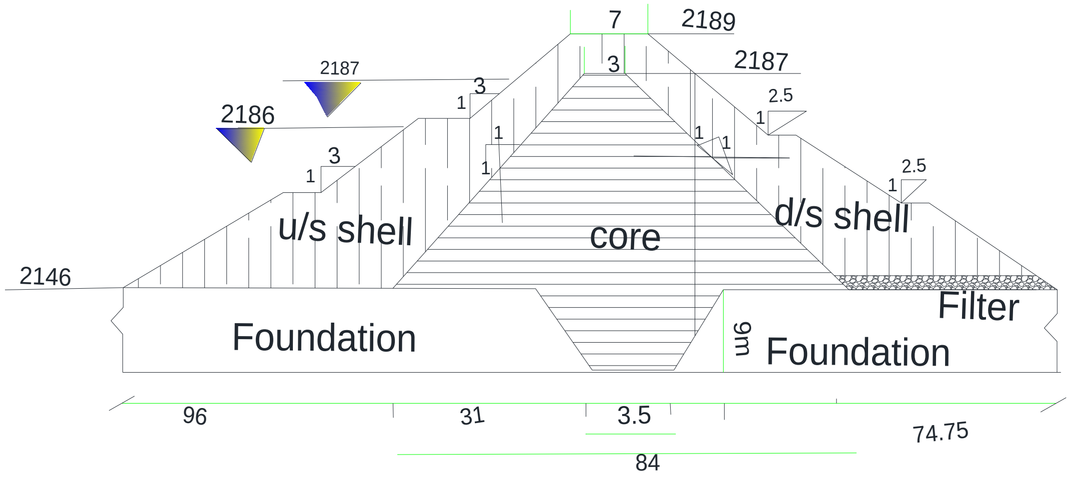

Figure 3.4: Dora Dam Cross Section

(Source: Feasibility and Detailed Design of Dora Earth Dam Dec, 2007)

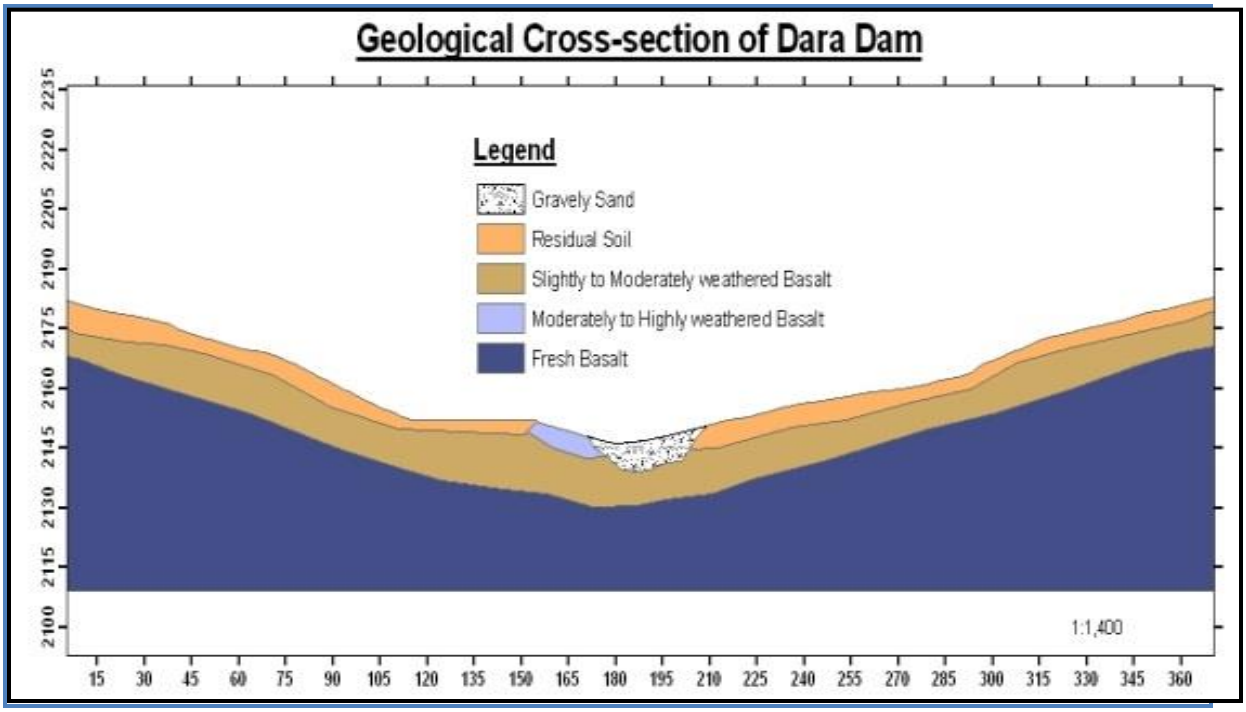

Figure 3.5: Longitudinal Profile along Dora Dam Axis

Source: Feasibility and geological investigation of Dora earth dam Asmelash (2015)

3.1.4 Climate

In the Dora earth fill, embankment dam there is no metrological station to record the rainfall data. According to this, the nearest station to the project area is machew and Adigudom metrological station. Based on the data transfer method the Machew metrological station is considered in case of the longer duration i.e. 56 years. Generally two rain fall season can be recognize, in northern Tigray; the 'belg' which can be gained small rain fall and from March to May, and 'kiremt' which is gained good rain fall and from July to September. The minimum average monthly rainfall is 1.696 mm at December and the maximum rainfall is 226.778mm at the august. That obtained from the Machew metrological station for 56 years of from (1954 to 2010). The maximum temperature of the Hintalo wejerat of Dora project area is between 23.10 ^{o}C February to 29.60 ^{o}C June and the Minimum Temperature ranges from 4.80 ^{O}C November to 12.8 ^{o}C July.

3.1.5 Geology

The regional geology of the area is characterized by rocks of varied composition such as Palaeozoic-Mesozoic sedimentary unit, Tertiary Volcanic and Quaternary deposits as well as the local geological condition dominates are Basalt and Enticho Sandstone rocks.

Central foundation: As exposed on the riverbed and observed from the test pit, the top 3.5m depth of part of the central foundation of the dam axis is covered by gravelly sand deposit Asmelash (2015). It is black in colour which indicates the source material mainly basalt that outcrops throughout the catchment of the site. It is loosely compacted, pervious and unstable during excavation. There is high flow of water until the observed depth of the test-pit. This deposit can extend to the depth of 6-7m as the thick accumulation of the deposit observed downstream of the proposed axis.

Underlying this highly to moderately weathered basalt can occur. This degree of weathering may extend to the depth of 9m. The coarse-grained nature of the weathering basalt will have pore spaces that can allow water to move freely Asmelash (2015).

Based on the present information the cut-off trench is proposed up to 9m at the river bed and 5m at the abutments depending on the slope; however this has to be verified using drilling especially on the centre of the river course or during the construction the cut off trench should reach up to the depth of the fresh and strong basaltic unit.

Abutments: both abutments are outcropped by similar geologic formation and their geotechnical properties are almost similar. The top of these abutments is outcropped by residual soil up to the depth of 0.7m. The depth of the residual deposit varies along the slopes of the ridge. In the middle of the ridge along the slope, the residual soil dips up to 1.50m. This soil is resulted from the highly to completely weathered basalt. It has a variable grain size on which the clay and silt grains are dominant with minor sand grains. The degree of weathering decreases with depth as a result the depth from1.0m to 5.50m is covered by moderately to slightly weathered rock unit. The trend of the joint is across the crest length of the proposed axis. A strong and a late coming basaltic dyke are observed on the river course of the reservoir that extends towards the right abutment of the dam Asmeash (2015).

3.2 Materials

3.2.1 Material Sources

Dora is an earth fill dam that is zoning type of dam made up of different materials. All of the materials are provided according to the design drawings and specification of the dam. The location details and other relevant information for identified potential material source sites are presented in figures below.

3.2.1.1 Impervious Core

The main objective of core is to protect seepage passing through the main body of the dam and to keep the phreatic line safe. The size and shape of the impervious core in a zoned dam depend on the availability of materials and their properties, especially hydraulic conductivity.

This function is only be performed effectively if the construction material, used for the core, has desirable engineering properties such as watertight; the material must not consolidate excessively under the weight of superimposed embankments and upon saturation.

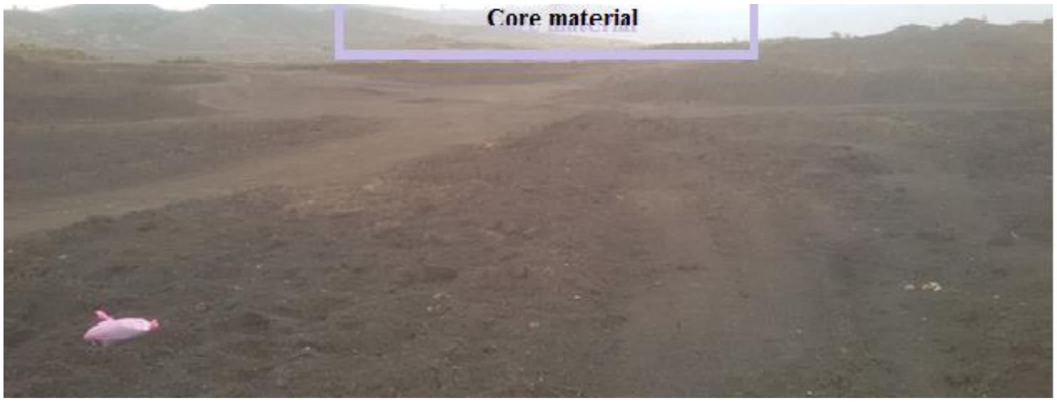

Figure 3.6: Photo of Clay Zone Taken From the Site

3.2.1.2 Filter Materials

Filter materials are Graded sand and gravel materials provided at the downstream horizontal part of dam. Filters are specified in terms of their particle size distribution. They are protecting, while being sufficiently coarser to allow drainage of seepage water. See the location and preparation of filter below.

Figure 3.7: photo of filter borrow zone

3.2.1.3 Shell Materials

Shell is a material constructed at the upstream and downstream part of the dam next to the coarse filter material to provide structural support for the core materials, to ensure good drainage and to distribute load over the foundation. The upstream shell affords stability against end of construction, rapid drawdown, earthquake, and other loading conditions and the downstream shell acts as a drain that controls the line of seepage and provides stability under high reservoir levels and during earthquakes. Therefore, Shell materials must exhibit adequate shear strength at economical slopes. The material proposed as shell material for Dora dam project was highly fractured Aphanatic basalt easily excavated by dozer and finer parts of blasted Agglomeratic basalt

Figure 3.8: highly fractured aphanatic basalt at quarry site and shell zone

3.2.1.4 Riprap Material

The upstream slopes of embankment dams are most probably exposed to dynamic and Mechanical attack by wave action and climatic weather conditions. Therefore, 1.5 m thick rock Riprap cover was provided on the upstream part of Dora dam to prevent from such damages. As shown on the figure 3.10 of geological map quarry site of Dora dam, the Agglomeratic Basalt is observed throughout the reservoir area interpolating with the Aphanitc basalt used as Riprap material. However, due to its smaller grading size as a result of closely to moderately Space fracturing Aphanatic basalt cannot be used as riprap independently but they can be used By mixing it with larger sized Agglomeratic basalt and /or ignimbrite.

Figure 3.9: photo of riprap zone taken from the site

3.3 Methods

- Digital camera to capture photographs of any flow on the body dam during site visit.
- World Wide Website to referring and gathering journals, reports, researches and any related works that was doing before.
- From the internet browser to brows Geo-studio software tutorials
- ArcGIS.10.3.Desktop software, to delineate geographical location of the study area.
- Geoslope_Geostudio_2007_version 7.10-build 4143 computer software to analyze seepage using SEEP/W model and slope/ W for slope stability analysis.
- Data collection (both primary and secondary data) and processing including inventories
- Document investigation relevant to the assessment of seepage and slope stability of the dam
- Discussions with relevant authorities working on the development of the dam

3.4 Data Collection

It will better making first visual inspection of a dam and its appurtenant structures to inspect any signs of over the dam second inspect a project's document files for clues to the probability of failure due to seepage and slope stability problems. Thirdly it may need to review background information on geologic characteristics, and records, and safety and inspection records to distinguish critical information relating to seepage. It includes both primary and secondary data’s.

3.4.1 Primary data collection

- To collect soil sample and conduct soil laboratory test
- To measure the downstream flow of the water that escape from the dam

Laboratory test is the basic part that needs strong attention and heartfelt responsibility. Because, every construction activity is led by the specifications that are done in laboratory works with related construction materials that we use in the construction site. If the materials that we use are not deeply investigated their chemical and physical properties like permeability, shear strength, and others, we cannot construct any structure.

Some of the tests that are done in this specific construction project at the time of construction those are: -

- ✓ Unit weight, Cohesion force, and friction angle of the embankment
- ✓ Hydraulic conductivity of the embankment
- ✓ In main lab (at office): - Procter test/optimum moisture content and maximum dry density determination,
- ✓ ATTERBERG limits (liquid limit, plastic limit, plasticity index), and gradation tests

3.4.2 Secondary data

Secondary data were collected from Tigray water resource bureau and Water Works Construction Enterprise as well as from Dora dam site. The datas are: -

- Design documents, which are useful to analyze design procedures, dimensions and cross sections of dam, properties of dam materials, geology of dam, etc.
- Construction reports, logs, records (including construction inspector's daily report), photographs, and as-built drawings about earthen dam of Dora

Any special reports prepared for the project or from the project related to seepage and slope stability case.

3.4.3 Parameters used for seepage analysis and slope stability

- Coefficient of Permeability
- Volumetric water content (porosity)

Saturated volumetric water content (SVWC) values for different embankment materials are adopted from different literatures and SEEP/W manual (SEEP/W, 2007)

Table 3.1: volumetric water content of embankment material (SEEP/W, 2007)

Description	**SVWC**	**Remark**
Core	0.5	SEEP/W sample function
Fine filter	0.35	SEEP/W sample function
Coarse filter	0.28	SEEP/W sample function
Inner shell	0.27	SEEP/W sample function
Rock fill	0.25	SEEP/W sample function

Table 3.2: typical values of coefficient of permeability of soils and rocks (Das B. M, 2007)

Material	Coefficient of permeability(m/s)	**Drainage property**
Coarse	0.01 to 0.1	Very good
Fine gravel, coarse, and medium sand	10^{-5} to 0.01	Good
Fine sand, loose silt	10^{-7} to 10^{-5}	Fair
Dense silt, clayey silt	10-8 to10-7	Poor
Silty clay, clay	10^{-11}to10^{-8}	Very poor
Ignimbrite	5×10^{-10} to 2 x $^{10-7}$	Poor to Very poor
Dense basalt	$1*10^{-11}$ to 10^{-8}	Very poor
Fractured basalt	$1*10^{-9}$ to $1*10^{-5}$	Fair to Very poor

3.4.4 Laboratory Analysis

These tests are the specifications for the tests that has to be done in the site and to select the required material for construction in the quarry site depend on the design rule. For every test, it has its own guide value and this should compare with the specific value or range gives from the lab.

3.4.4.1 Proctor Test/Determination of Optimum Moisture Content and Maximum Dry Density

Optimum moisture content and maximum dry density is decided graphically as shown below on the graph, which is drawn dry density versus moisture contents. For the general procedure and detail calculation show at Appendix-A. Moisture content is the mass of water which can be removed from the soil and aggregate by heating (oven drying) at 105-110^{0}c expressed at a percentage of the dry mass. This test was done to determine the optimum moisture content (O.M.C) and maximum dry density (M.D.D), for detail show Appendix-A.

3.4.4.2 Compaction Test

In compaction test at lab for core, we have input data or specification given from laboratory result that we have seen in proctor test earlier. These data are O.M.C and M.D.D that have a calculated result of 13.8% and 1.83g/cm^3 respectively.

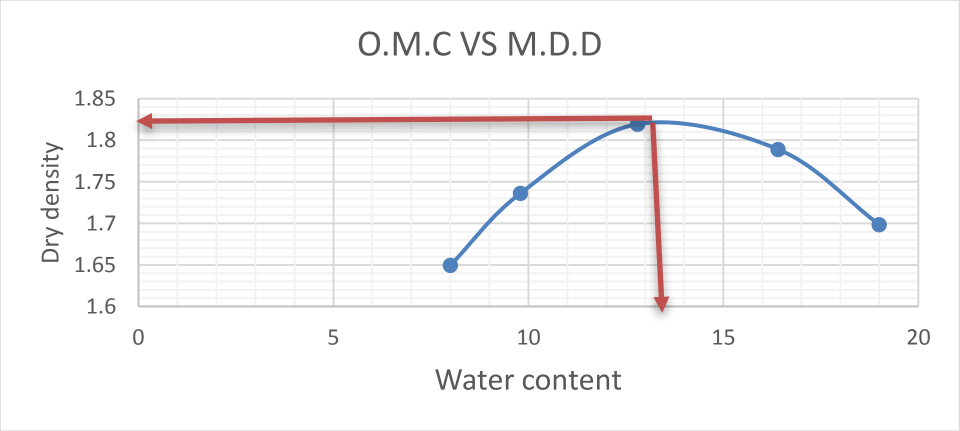

Figure 3.10:Graph of dry density versus moisture content of core (appendix-A, Table A.5)

From the above graph, M.D.D is 1.83 g/cm3 and O.M.C is 13.8%. Finally, this result is send to the field for compaction test.

In compaction test at lab for shell, we have input data or specification given from laboratory result that we have seen in proctor test earlier. These data are O.M.C and M.D.D that have a calculated result of 11.8% and 1.848g/cm3 respectively.

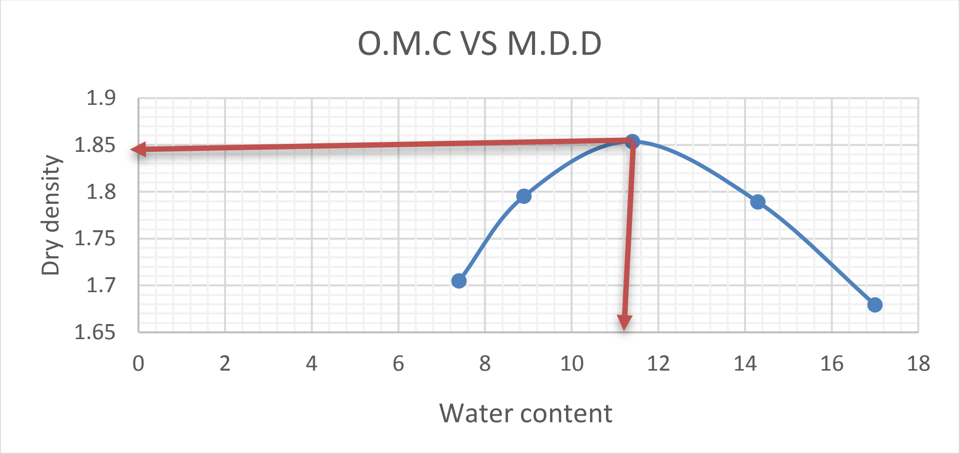

Figure 3.11:Graph of dry density versus moisture content of shell (appendix-A, table A.5). From the above graph, M.D.D is 1.848 g/cm3 and O.M.C is 11.8%. Finally, this result is send to the field for compaction test.

3.4.4.3 Liquid Limit Test

The liquid limit of a soil is the moisture (water) content at which soil passes from the plastic to liquid state as determined by the liquid limit test. For detail Procedures show Appendix-A. The water content of the soil can be expressed as the moisture content in percentage of the weight the oven dried mass and can be expressed as follows.

$$\text{M.C (\%)} = \frac{(Wt\ of\ wet\ sample + cont.) - (Wt\ of\ dry\ sample + cont.)}{(Wt\ of\ dry\ sample + cont.) - Wt\ of\ cont.} x100$$

Finally, draw graph by taking Moisture content vs. number of blows and read the value of liquid limit at 25 numbers of blows.

3.4.4.4 Plastic Limit Test

Plastic limit is the moisture content at which a mixture of soil passes from a liquid state to that of a semi-solid state. For detail Procedures show Appendix-A.

Calculation

O.M.C (%) $=\frac{(Wt\ of\ wet\ sample+cont.)-(Wt\ of\ dry\ sample+cont.)}{(Wt\ of\ dry\ sample+cont.)-Wt\ of\ cont.}x100$

Finally, calculate plastic limit. PL = average of the calculated moisture content.

3.4.4.5 Plasticity Index Test

It is simply the difference between the liquid limit and plastic limit.

PI=LL-PL Where: - PI- plasticity index, LL- liquid limit, and PL- plastic limit

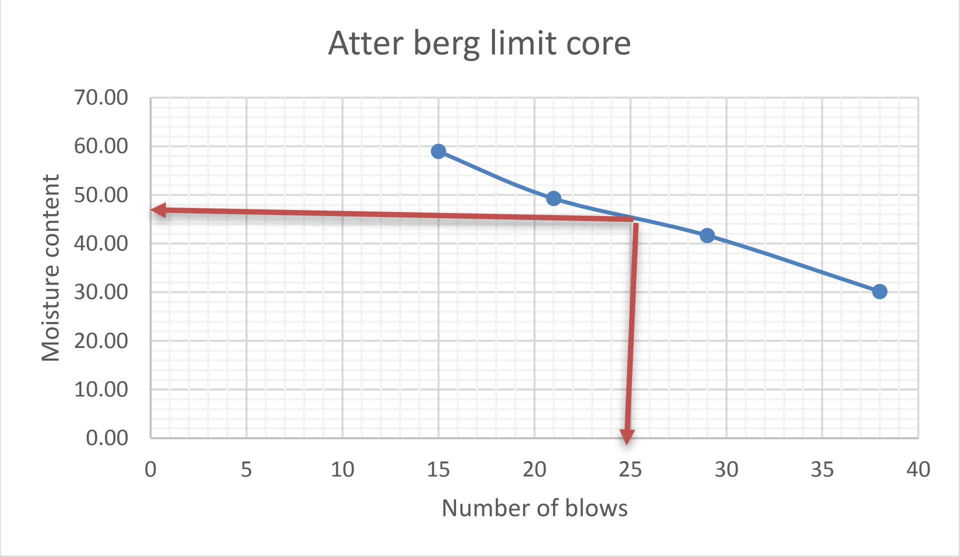

Figure 3. 12: Graph of moisture content versus number of blows of core

Finally, we can read liquid limit from the resulted graph and plastic limit by taking the average of the two trial values. Plasticity index is then, the difference of liquid limit and plasticity index.

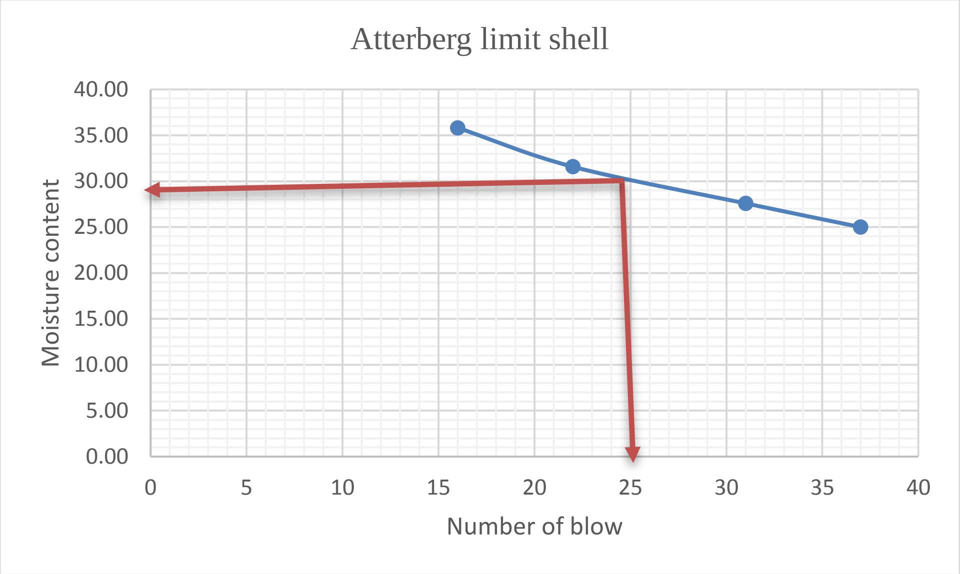

Figure 3.13: Graph of moisture content versus number of blows of shell

Finally, we can read liquid limit from the resulted graph and plastic limit by taking the average of the two trial values. Plasticity index is then, the difference of liquid limit and plasticity index.

3.4.4.6 Permeability

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

The coefficient of permeability depends on particle size and shape. Because the size of the pore space determines the quantity of permeability. Generally, smaller particles have low permeability because of the smaller pore space hence larger particles have higher permeability. However, presences of fine grains in a coarse-grained material pull down the permeability significantly because, the tiny particles considerably filling out the pore-spaces.

Laboratory test methods which is perform for coefficient of permeability and for detail Procedures show Appendix-A.

3.4.4.7 Cohesion

Cohesion is the force that holds together molecules or like particles within a soil. Cohesion, c, is usually determined in the laboratory from the direct Shear test.

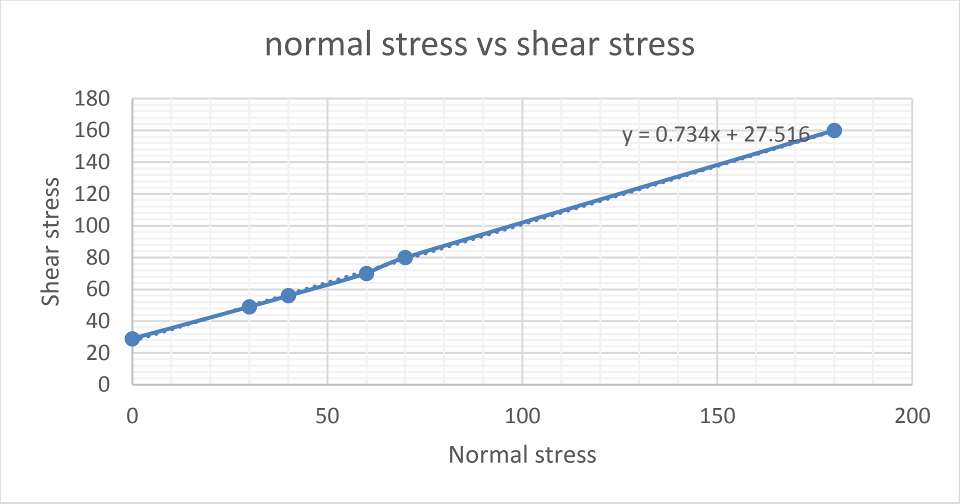

Figure 3.14:Plot of normal stress plot of normal stress versus shear stress for shell (Appendix-A, Table A.12)

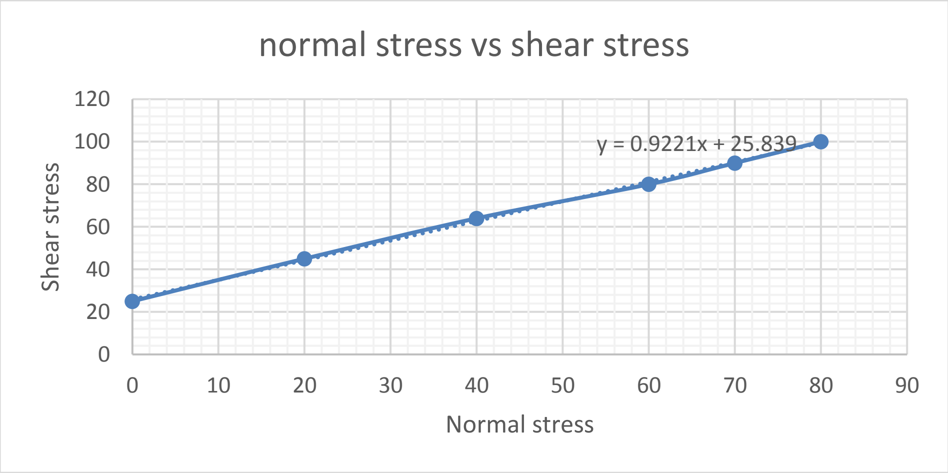

Figure 3.15: Plot of normal stress versus shear stress for core (Appendix-A, Table A.13)

3.4.4.8 Angle of internal friction

Angle of internal friction for a given soil is the angle of the shear stress and normal effective stress plot at which shear failure occurs. Angle of internal friction,

3.4.4.9 Unit Weight

Unit weight is determined in the laboratory by measuring the weight and volume of a soil sample obtained from the field. Table3.3 Summary of laboratory results of samples

Table 3.3: Summary of laboratory results of samples

Material Type	Atterberg limits			Grain size				
	LL [%]	PL [%]	PI [%]	Gravel[%]	Sand [%]	Silt[%]	Clay[%]	USCS
Core	45.9	25.2	20.7	0	18.8	21.6	59.6	CL-ML
Shell	30.8	15.6	15.2	49.4	42.7	6.6	1.3	ML or MH

The Soil (CL-ML) is poorly graded sand with clay (SM) and it is fair in shear strength, generally impervious and good in piping resistance, while soil (ML or MH) is poorly graded sand with silt (SM) and also it is good in shear strength (medium impervious) and excellent in resistance to piping.

Table 3.4: Average Engineering Parameter for the Fill Materials

Material Type	Geotechnical design parameter					
	Cohesion, C'[Kg/cm2]	Friction Angle[ϕ]	Unit-Weight γ (kN/m^3)	Optimum Moisture content. OMC[%]	Maximum dry Density, MDD[g/cm3]	Permeability, K[cm/sec]
Core	27.52	23.2	17.95	13.8	1.83	2.306x10^{-6}
Shell	25.84	29.36	18.13	11.8	1.848	2.912x10^{-5}

3.5 Seepage analysis

The phreatic surface of the seepage régime, i.e. the free surface, must be kept well clear of the downstream face to avoid high pore water pressures which may promote slope instability. In the extreme case of the seepage line emerging on the face, local softening and erosion will occur and may initiate sloughing as a prelude to instability.

Seepage pressures and velocities must also be controlled to prevent internal erosion and particle migration, particularly from the core. Seepage control is affected by the incorporation of vertical chimney drains and horizontal drainage layers, protected by suitable filters and transition layers.

In this investigation a basic knowledge of seepage theory and flow net construction will be assumed, including entry and exit conditions. A more exhaustive general discussion of flow nets and seepage analysis is provided in Cedergren (1977). Embankment flow nets in particular are considered in depth, with numerous illustrations in Cedergren (1973).Embankment seepage control is achieved in Volpe and Kelly (1985). The fundamental relationships applicable to flow nets for two-dimensional seepage are summarized below.

The seepage flow, q is defined by

$$q= \frac{K'.H.Nf}{Nd} \quad \text{……………………………………………………………… …...Equation 3.1}$$

Where H is the head differential and the ratio Nf/Nd is the flow net shape factor, i.e. the number of flow channels, Nf, in relation to the number of decrements in potential,

SEEP/W Software Model

SEEP/W is a finite element based GEO-STUDIO component, used for seepage analysis. It is applied for modeling of movement of water and estimation of pore water distribution through a pores media of a soil and rocks. Its comprehensive formulation makes it possible to analyze both simple and complex seepage problems.

$$Q= kiA \quad \text{………………………………………………………………………………3.2}$$

Where k is hydraulic conductivity, I is hydraulic gradient, A is cross sectional area

This tool has great application in the analysis of geotechnical, civil, hydrological and mining engineering projects (SLOPE/W, 2007).

There are two fundamental types of seepage analysis: steady state seepage analysis and transient seepage analysis. A steady-state seepage analysis is an analysis type where water pressures and water flow rates do not change with time. Since steady-state analyses ignore the time domain, it greatly simplifies the equations being solved.

A transient analysis, on the other hand, has pressure conditions that change with time.

This research is carry out both steady and transient seepage analyses to determine the amount of water flows (flux) passing through the embankment, to calculate the pore water pressure inside the embankment which is used as an input for the stability analysis of different loading conditions. For any seepage analyses, three finite element components are keys: Geometry, Material property and boundary condition (BC).

Geometry

Once dam dimension has been fixed based on guideline and standard, the geometry has been drawn on the SEEP/W window and will be discretized to small elements as finite element numerical models are based on the concepts of subdividing (discretize) a continuum into smaller pieces, describing the behavior or action of the individual pieces and the reconnecting all the pieces to represent the behavior of the continuum as a whole (SEEP/W, 2007).

Material Property and Model

SEEP/W has three material models: none model (used for removed part of a model), saturated/unsaturated and saturated model and for this study the latter two material models has been used. A summary of these models and the required soil properties are given below and a discussion of the individual parameters and functions are provided as follow (SEEP/W, 2007).

Saturated / Unsaturated model

During application of saturated unsaturated type of model material properties like hydraulic conductivity function, ratio, direction, and volumetric water content function are required.

This type of model is used for all embankment material as these materials are exposed to saturated and unsaturated condition due to the fluctuation of reservoir water (Broaddus, 1990)

Saturated only model

The required material properties are hydraulic saturated conductivity (Ksat), ratio and direction and saturated volumetric water content. The foundation materials are attached to this model as the foundation materials are expected to be saturated all the time regardless of reservoir water fluctuation (Broaddus, 1990).

Boundary condition (BC)

To analyze seepage problems on SEEP/W boundary conditions has to be set as solutions are dependent on the boundary conditions assigned to a problem. The equation of finite element equation for seepage analysis would be given by (SEEP/W, 2007):

$[K]\{H\}=Q$……………………………………………………………………Equation 3.3

Where

[K]= a matrix of coefficients related to geometry and materials properties

{H}=a vector of the total hydraulic heads at the nodes, and

Q=a vector of the flow quantities at the node.

Boundary conditions can be one of two fundamental options either can be specified with H (head) or Q (flux). For simplicity purpose, there are of boundary conditions, which can be assigned: potential seepage face, zero pressure and total head pressure. Therefore, for this study these boundary conditions have been applied depending up on the analysis type. The total head boundary condition in SEEP/W has been assigned which is expressed in the form of:

$H=\frac{U}{\gamma w}+y$……………………………………………………………………….Equation 3.4

Where:

H = the total head (in meters)

u = the pore-water pressure (kPa)

γw= the unit weight of water (kN/m3)

y = the elevation (meters)

Sudden draw down

In addition to what explained above, basically, two sets of boundary conditions are required to perform a transient seepage analysis: an initial boundary condition and a transient boundary condition (Huzjak et al, 2009). Initial boundary condition is required to define pore water pressures throughout the model at the beginning of the transient analysis. For this analysis, the initial boundary condition is taken from SEEP/W result which is a total head on the upstream embankment slope equal to the reservoir level at the initiation of drawdown.

Once the initial boundary condition has been set, transient boundary conditions should also be provided to simulate the analysis and these boundary condition case can be assigned in two ways:- as instantaneously drawdown or drawdown as a function time.

Instantaneous drawdown condition: - during instantaneous draw dawn analysis, it is assumed that the water in the reservoir will be withdrawn instantaneously but dissipation of pore water pressure may take longer time so that the analysis can be carried out for any time preference. This scenario is so conservative (USBR, 2012) that the water in the reservoir is assumed to be depleted instantaneously which may not happen in real condition.

Draw down with time:-in the case of draw down with time the water in the reservoir is drawn as a function of time and these head functions can be developed from the reservoir's elevation capacity curve and the design flow rates of the outlet works.

In this study, the first drawdown scenario (Instantaneous drawdown condition) is used assuming that the water in the reservoir has been drawn instantaneously to the minimum draw down level and result has been analyzed for the next 5 days after drawdown.

Procedure

- The maximum dam cross-section will be drawn on SEEP/W window

- The respective materials will be assigned to their regions
- The dam cross section will be discretized to small elements to apply FEM
- Material property and material model will be assigned to the respective region
- Type of analysis will be fixed (steady state or transient case)
- Boundary conditions will be applied depending op on analysis type
- Then the problem will be solved and the results will be analyzed and discussed

At the end of seepage analysis, the amount of flux thorough the dam body, through the dam foundation would be found. In addition the pore water pressure distribution in dam body and foundation, which is key parameter in the analysis of transient seepage and slope stability analysis, would be determined.

3.6 Slope stability analysis

For slope stability, analysis SLOPE/W, which is a component of GEO-SLOPE, has been used which is designed and developed for analysis of stability of slopes. The package is based on limit equilibrium (LE) principle and it consists of many different methods inside. Among the different methods incorporated in this package, Morgenstern-Price method is used for this thesis as this method satisfies both force equilibrium and moment equilibrium. The package requires three input data depending up on the problem. These are geometry, soil property and model, pore water pressure. Providing all the necessary parameters limit equilibrium will compute a factor of safety with the equation Duncan and Wright, (2005) below:

$$Fs=\frac{\text{Shear strength }(\tau)}{\text{Equilibrium shear stress}(s)} \quad \text{……………………………..............................Equation 3.5}$$

Where F.S = factor of safety,τ is the available shear strength and

S is the equilibrium shear stress.

The shear strength can be expressed by the Mohr–Coulomb equation and expressed by equation (Duncan and Wright, 2005):

S= $C+(\sigma n-u)\tan\phi$……………………………………………………..…………Equation 3.6

Where,

C and ϕ are the Mohr-coulomb cohesion and friction angle for the soil respectively,

σn is the total normal stress on the shear plane

σn-u effective normal stress

U- Pore water pressure

Geometry

Similar geometry used in SEEP/W has been used for slope stability analysis

Material property and model

There are different models incorporated in this package and Mohr-Coulomb is one of them, which are chosen for analysis. Material properties like unit weight, angle of friction and cohesion will be used as input parameters for the analysis.

Procedure:-

- The maximum dam cross section has been drawn on SLOPE/W window
- The respective materials has been assigned to their region
- Material property and material model will be assigned to each region
- Method of analysis has been selected (Morgenstern-Price is used for this thesis)
- Pore-water has been taken from seep result in this analysis
- Then the problem get solved and the results are analyzed and discussed

Results to Obtain

At the end of this analysis, the state of condition of embankment slope stability has been determined through a single number called factor of safety and interpretation and discussion has been made for all results computed during different loading conditions based on the criteria presented in Table 3 below. According to USACE, (2003) the minimum factor of safety with respect to loading condition has been summarized as follow.

Table 3.1: Minimum safety versus loading condition

Slope	**Loading condition**	**Reservoir characteristics**	**Minimum factor of safety**
Upstream and downstream	End of construction	Reservoir empty	1.3
Downstream	Steady State seepage	Reservoir at normal maximum operating level (full supply level)	1.5
Downstream	Maximum flood	Reservoir at maximum flood level	1.5
Upstream	Drawdown	Rapid drawdown to critical level	1.3

4.0 RESULTS AND DISCUSSIONS

4.1 Dam Zoning and Geometry

It has been discuss earlier in this document that Dora dam for irrigation purpose originally being designed an earth fill dam with central clay core, which requires a special treatment for the dam body (TWRB, 2008). To solve this problem the upstream side blanket will be constructed.

The dam cross section has been done based on the basic criterion listed in section 3.1.3. 43m height at the maximum site depth and crest width is adapted to be 7.0m; the side slopes have a 2.5H: 1V on the upstream side separated by 3m wide berms. The berms are also functionally required for various purposes. On downstream side, the side slopes are 2.0H: 1V above berm and 2.5H: 1V below berm.

A core top width of 3m has been provided for utilization of construction equipment, and considering the actual site condition regarding to the location of the core material zone and economic consideration, a clay core slope of 1V: 1H and 1V: 1H upstream and downstream respectively have been selected. Transition filter zones are required to separate zones of different permeability and compressibility properties within the embankment.

For prevention of internal erosion, piping and development of excessive pore pressure inside the dam a vertical transition and filter material of appropriate gradation is provided at both sides of the clay core.

Finite element method (FEM) is a numerical technique for finding good approximate solutions to boundary value problems for partial differential equations by dividing very complicated problem into smaller, simpler parts called finite elements.

Laboratory test is the basic part that needs strong attention and heartfelt responsibility describing at section 3.4.1. Because, every construction activity is led by the specifications that are done in laboratory works with related construction materials that we use in the construction site. If the materials that we use are not deeply investigated their chemical and physical properties like permeability, shear strength, and others, we cannot construct any structure.

4.2 Seepage Analysis

The seepage analysis has been done and discretization of the whole domain in to finite elements was the first step in finite element method. A combination of quadrilaterals and triangular polygons are used, as these polygons are best mesh shape for two-dimensional analyses. The number of finite elements has been adjusted based on the recommendation given by SEEP/W, (2007) engineering book where each discretized elements are clearly visible on 100% zoom extent as shown in the figure below. In addition to domain discretization, boundary condition has been applied for upstream total head, downstream total head and zero pressure lines on downstream filter.

The flux through the dam body and the foundation has been computed and based on the computation, the total flux through the dam body and foundation is estimated to be 9.4993x10^{-5} m3/sec/m. Among the total flux found, a flux of 1.1535x10^{-6}m3/sec/m is through the dam foundation (Fig.4.3).and the flux through the dam body and the foundation has been computed and based on the computation, during the transient case the total flux through the dam body and foundation is estimated to be 9.4992x10^{-5} m3/sec/m. Among the total flux found, a flux of 1.1532x10^{-6} m3/sec/m is through the dam foundation (Fig.4.4).

Considering the crest length of Dora embankment dam to be 400 m, the total volume of water seeps through the dam body and the foundation during the case of steady state and transient case is estimated to be 0.0379436 m3/sec and 0.0342572 m3/se respectively. The rate flow of water through dam body and foundation by itself does not lead to a conclusion regarding to dam safety problem. However, the quantity of seepage through the embankment should be controlled to required level. A leakage of 0.03m3/sec through the embankment and dam foundation is generally acceptable if proper filter material, drainage system and relief wells are incorporated (Jansen, 1988). In comparison to this recommendation, the quantity of seepage loss through the dam body and foundation calculated in this research lies above the allowable range.

4.3 Slope analysis

The Janbu's Simplified method, (1) considers normal interslice forces, but ignores interslice shear forces, and (2) satisfies over all horizontal force equilibrium, but not over all moment equilibrium. The Janbu's method is similar to the Bishop's Simplified method except that the Janbu's method satisfies only overall horizontal force equilibrium, but not overall moment equilibrium.

The Spencer method, for example, uses a constant function, which infers that the ratio of shear to normal is a constant between all slices. We do not need to select the function; it is fixed to be a constant function in the software when the Spencer method is selected.

Only the Morgenstern-Price and GLE methods allow for user-specified interslice functions. Some of the functions available are the constant, half-sine, clipped-sine, trapezoidal and data-point specified. The most commonly used functions are the constant and half-sine functions. A Morgenstern-Price or GLE analysis with a constant function is the same as a Spencer analysis. SLOPE/W by default uses the half-sine function for the M-P method. The half-sine function tends to concentrate the interslice shear forces towards the middle of the sliding mass and diminishes the interslice shear in the crest and toe areas.

The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and the factor of safety of downstream slope during steady state condition; end of construction and during construction stages are computed, 1.795, 2.599 and 1.604 respectively.

The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and the factor of safety of upstream slope during steady state, sudden drawdown; end of construction and during stage of construction, computed, 2.525, 1.432, 2.980 and 2.599 respectively.

Software results computed below

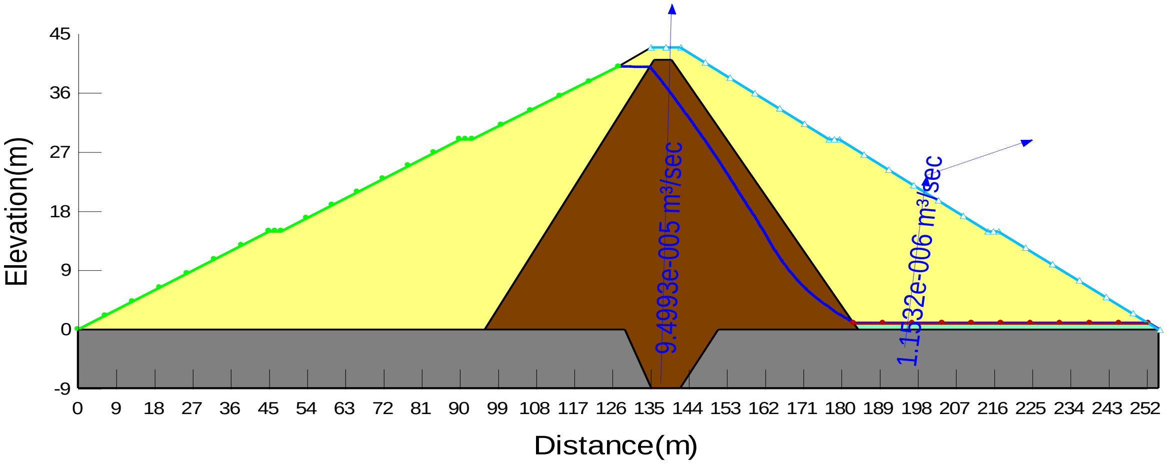

Figure 4.3: Steady state seepage quantification

Seepage Losses-The annual amount of water loss through the dam body and foundation is: -

Q = q*B ……………………………………………………………………...Equation 4.1

Where q = seepage loss rate (m3/y/m) and B = Total crest length of dam B = 400m),

q= 9.4992 $x10^{-5}$ m3/sec/m or 2991.473424 m^3/yr/m.

Q = q*B = 2995.667712 m^3/yr/m *400m = 1198267.0848m^3/yr

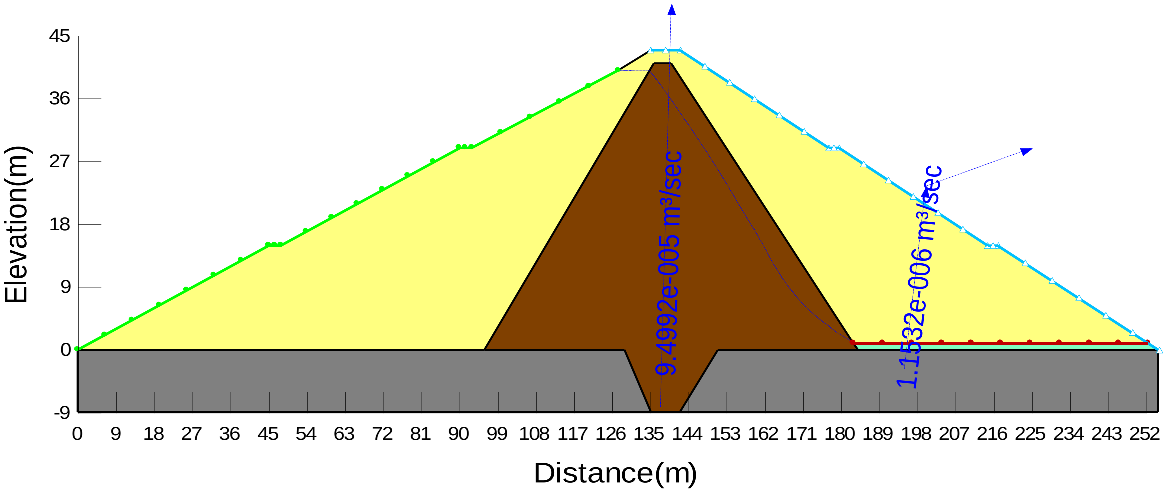

Figure 4.4: Transient Seepage Analysis quantification

Seepage Losses-The annual amount of water loss through the dam body and foundation is determined by (Q = q*B) formula.

Where q = seepage loss rate (m3/y/m) and B = Total crest length of dam, B=400m,

q= 8.5643×10^{-5} m^3 /s/m or 2700.837648 m^3/yr. /m.

Q = q*B = 2700.838 m3 /yr. /m *400 m = 1080335.0592 m^3 /yr.

Table 4.1: Results of Dora dam seepage analysis by geo-studio

Software result using secondary data (m3/se)		Software result using primary data(m3/se)		Actual measurement(m3/se)			Standard(Jansen, 1988) (m3/se)
Steady state case	Transient case	Steady state case	Transient case	Trial-1	Trial-2	Trial-3	
0.058308	0.057724	0.0379436	0.0342572	0.034987767	0.04554062	0.46814967	0.03

The line of zero pressure and contours of total head has been shown in figure 4.5. As it has been shown in figure that the line of zero pressure is horizontal until it reached to clay core because all the materials upstream of the core are assumed free drain, no energy is dissipated, and as it gets in to the clay core, it is rapidly drawn. The distribution of total head has also been presented in the figure 4.5 where this total head is a function of pore pressure and elevation head.

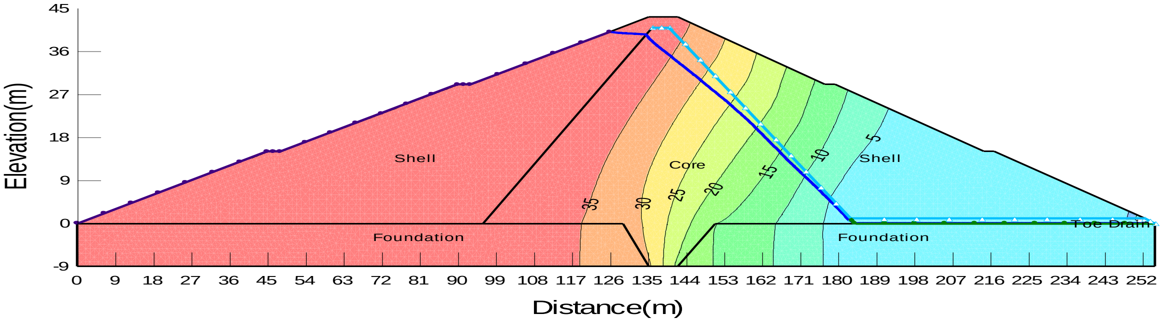

Figure 4.5: Line of zero pressure and contours of equipotential lines (total head)

The distribution of pore water pressure through the dam body and the foundation at steady state condition is shown in Figure 4.6 and this result is using as an initial parent analysis in SEEP/W, SLOPE/W and for transient analysis during sudden drawdown and for downstream slope stability analysis.

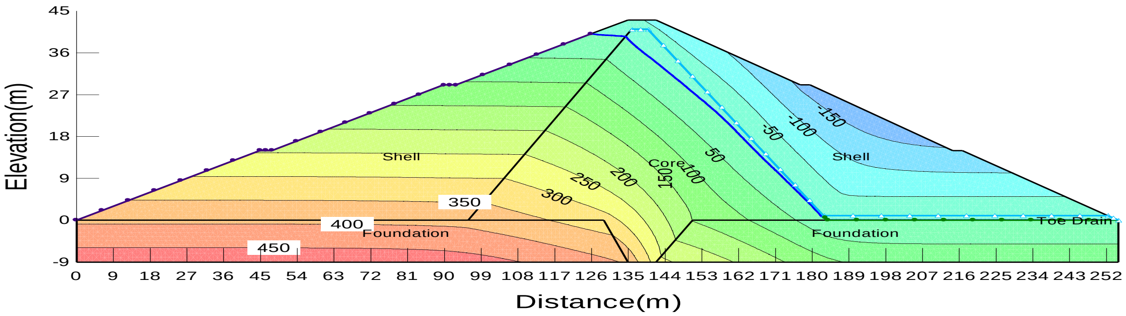

Figure 4.6: Contours of pore water pressure

The hydraulic gradient inside the dam body is associated with the possibility of material erosion and piping so that the hydraulic gradient at the contact of core to foundation contact and core to shell material has to be controlled. The hydraulic gradients are computed in the dam body and foundation and the result varies in the range of 0.2 to 1.6 as shown in the (fig.4.7) below.

For embankment dams, it is recommended that the hydraulic gradient should not exceed 4 (Kutzner, 1997). Therefore, the computed hydraulic gradient in this work less than the requirement of standard.

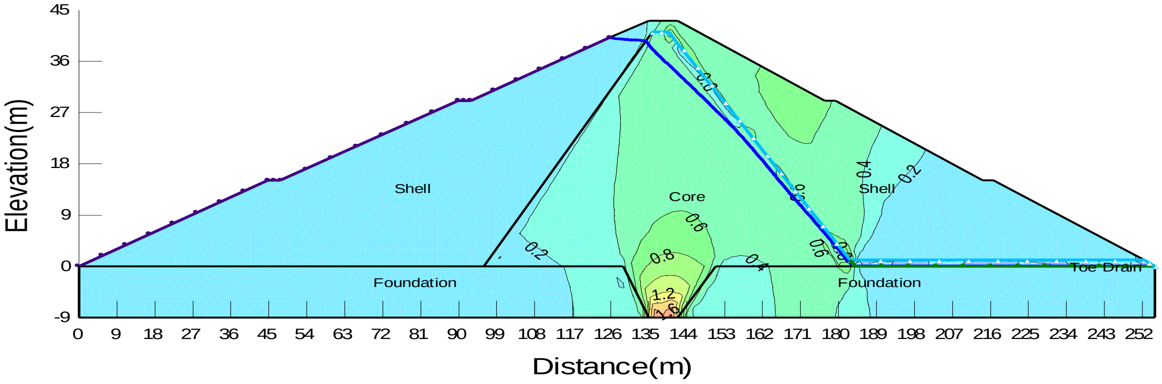

Figure 4. 7: Contours of horizontal gradient through dam body and foundation

4.3 Static Slope Analysis

Slope stability analysis has been done based on the respective shear strength parameters of each material. Based on the analysis of the dam that satisfies the minimum requirements in all loading condition. The results and discussion of stability analysis has been presented as follows based on each loading conditions.

Table 3.5: Minimum required factor of safety versus loading condition (USACE, 2011)

Slope	Loading condition	Reservoir characteristics	Minimum factor of safety
upstream and downstream	End of construction	Reservoir Empty	1.3
downstream	Steady state seepage	Reservoir at normal maximum (FSL)	1.5
downstream	Maximum flood	Reservoir at maximum flood level	1.5
upstream	Drawdown	Rapid drawdown to critical level	1.5

4.3.1 Steady state condition

The downstream embankment slope has been analyzed for steady state condition when the reservoir is at maximum pool level and a minimum factor of safety of 1.795 has been found (Fig.4.14).Besides the mass that intended to fail at this scenario is so thin so it will not compromise the stability of the embankment. The minimum factor of safety required for this loading condition is 1.5, so that the embankment is safe during steady state condition.

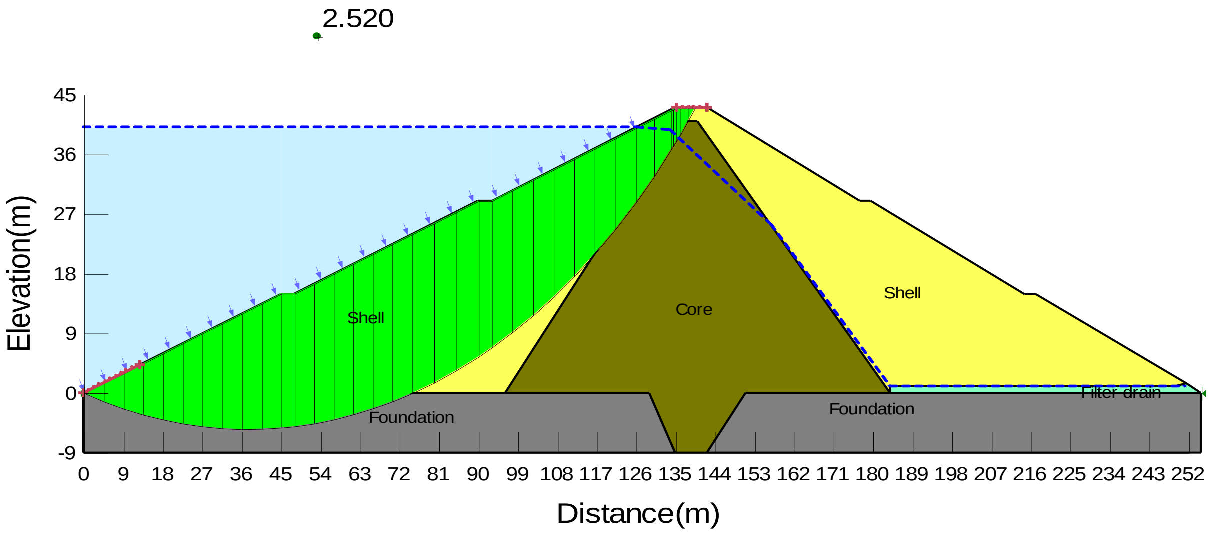

Figure 4.8: Factor of safety of upstream slope by bishop method during steady state condition

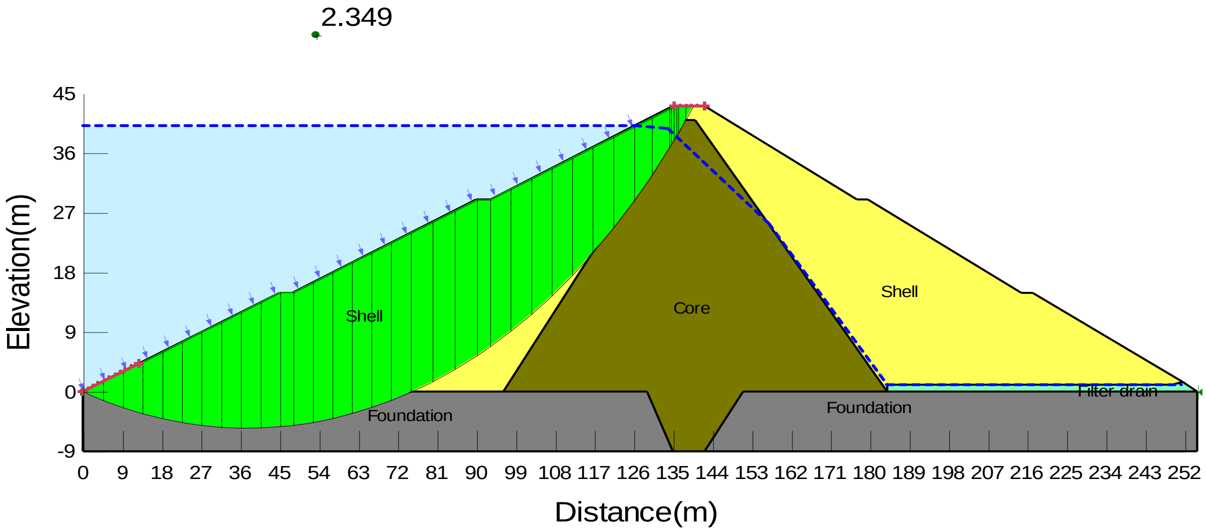

Figure 4.9: Factor of safety of upstream slope by Janbu method during steady state condition

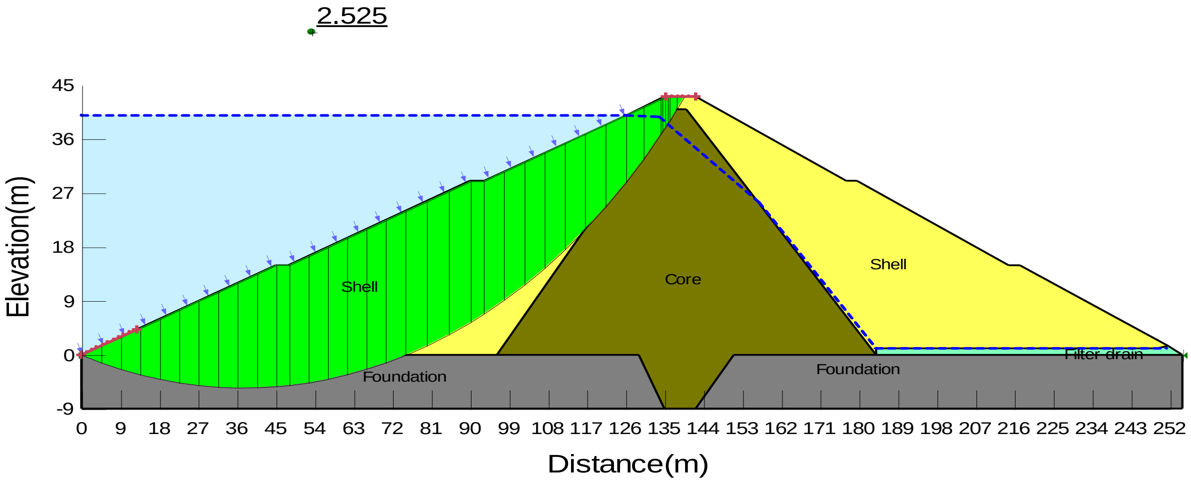

Figure 4.10: Factor of safety of upstream slope by Morgenstern- prince method during steady state

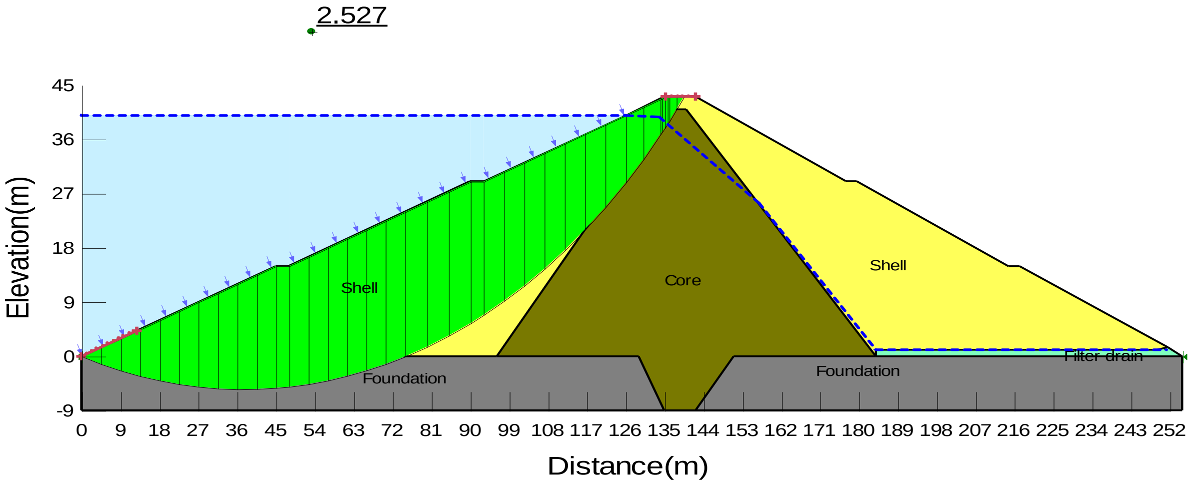

Figure 4.11: Factor of safety of upstream slope by spencer method during steady state condition

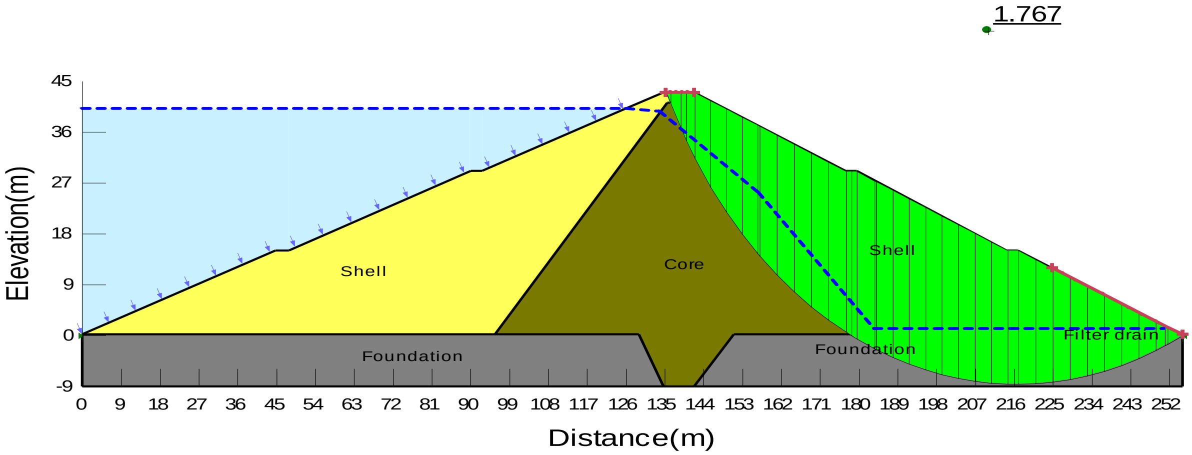

Figure 4.12: Factor of safety of downstream slope by bishop method during steady state condition

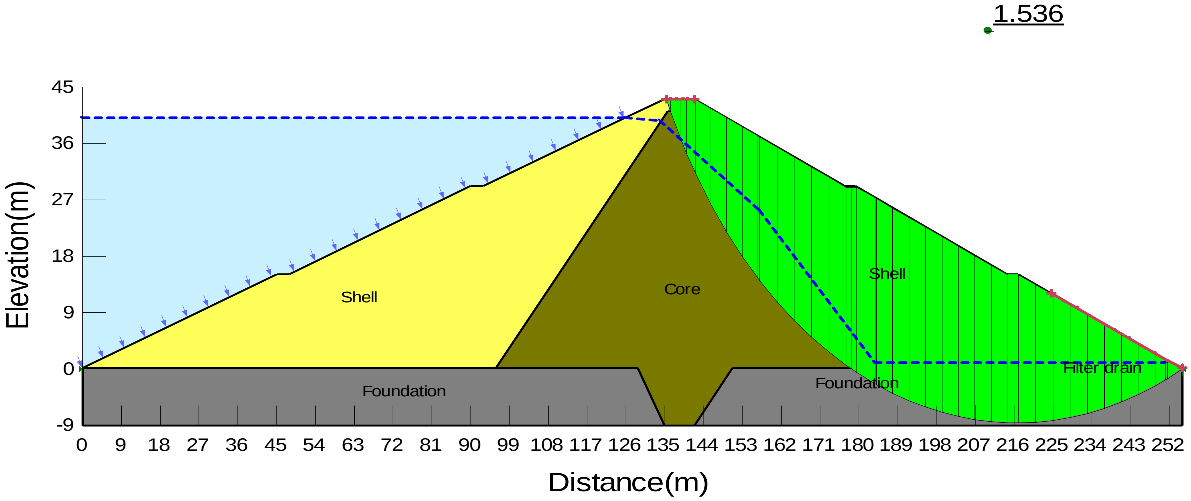

Figure 4.13: Factor of safety of downstream slope by janbu method during steady state condition

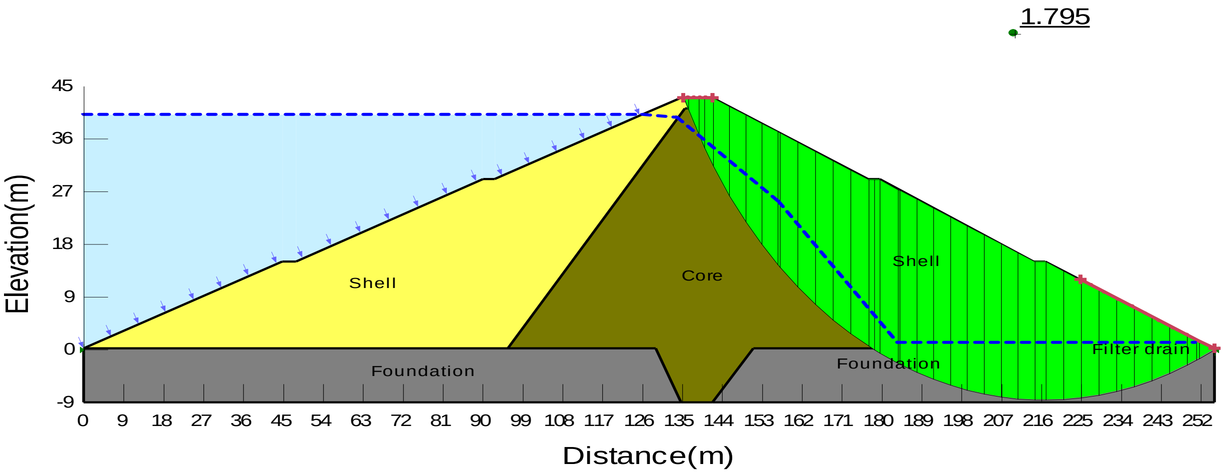

Figure 4.14: Factor of safety of downstream slope by Morgenstern- prince during steady state condition

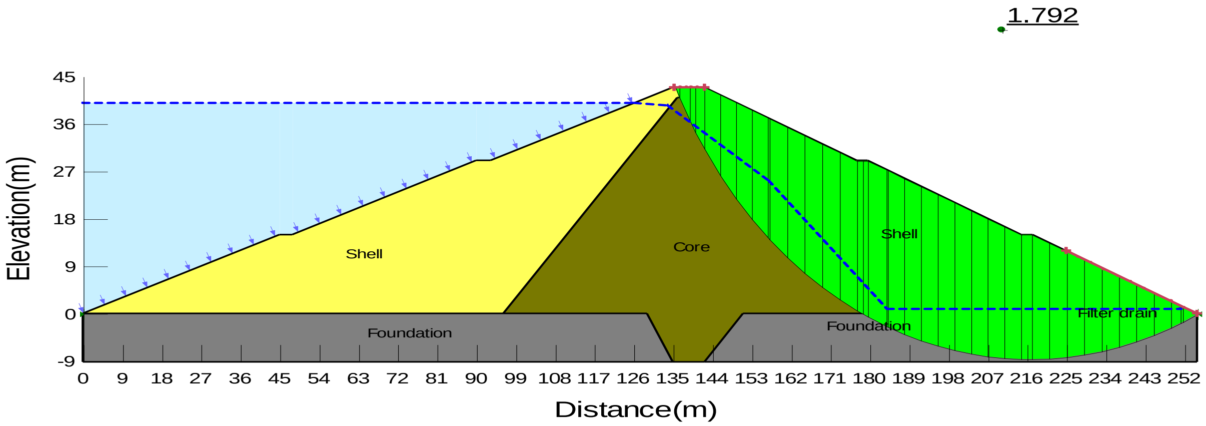

Figure 4.15: Factor of safety of downstream slope by spencer method during steady state condition

4.3.2 Sudden draw down

During sudden drawdown, the stabilizing effect of the water on the upstream face is lost suddenly, but the pore water pressures within the embankment may remain high. As a result, the stability of the upstream face of the dam will significantly reduce. During sudden drawdown, condition materials with high hydraulic conductivity drain quickly, but materials with low hydraulic conductivity takes a long time to drain so a transient (time dependent) analysis is required. For this study, it is assumed that the core material will be of slow drain while other embankment materials are of free drain Duncan and Wright, (2005) and Fell et al, (2005).

Basically, two sets of boundary conditions are required to perform a transient seepage analysis: an initial boundary condition and a transient boundary condition (Huzjak et al, 2009). Initial boundary condition is required to define pore water pressures throughout the model at the beginning of the transient analysis. For this analysis, the initial boundary condition is taken from SEEP/W result. Which is a total head on the upstream embankment slope equal to the reservoir level at the initiation of drawdown.

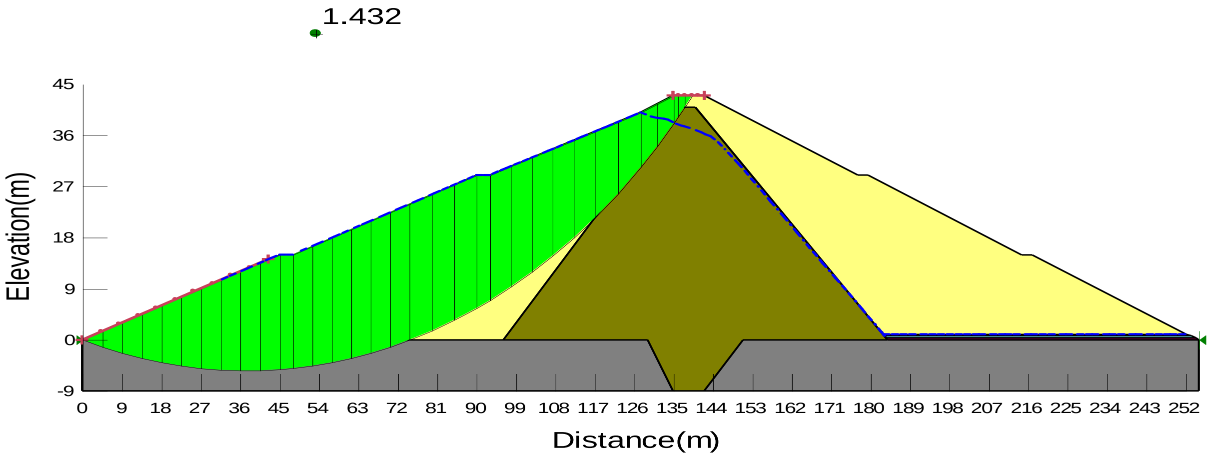

Figure 4. 16: Factor of safety for upstream slope by Janbu's method for rapid drawdown

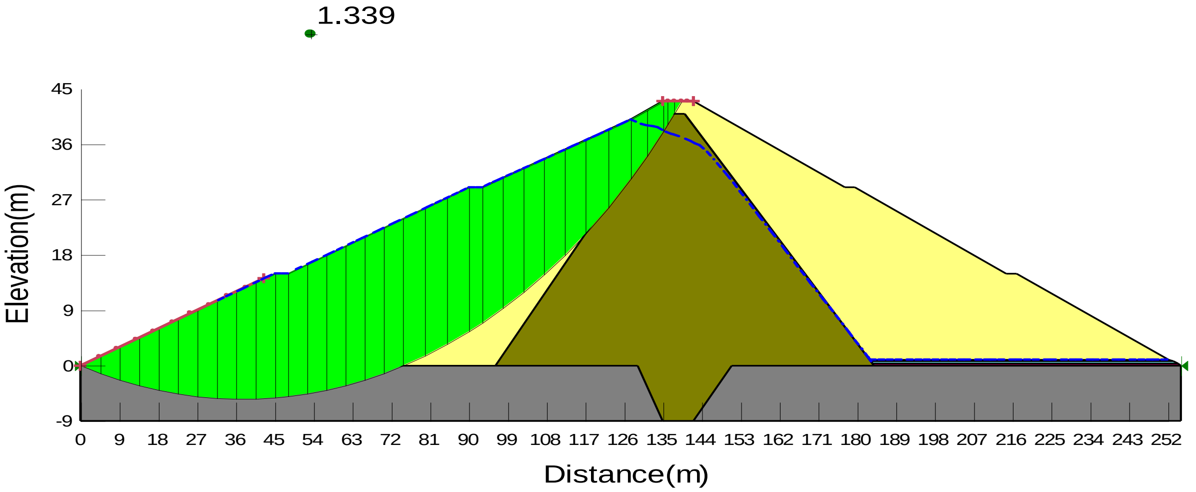

Figure 4. 17: Factor of safety for upstream slope by bishop’s method for rapid drawdown

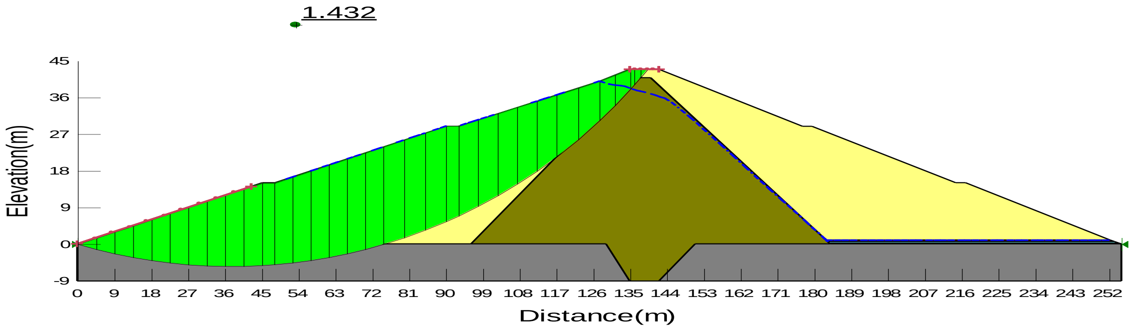

Figure 4.18: Factor of Safety for Upstream Slope by Morgenstern-Price's Method for Rapid Drawdown

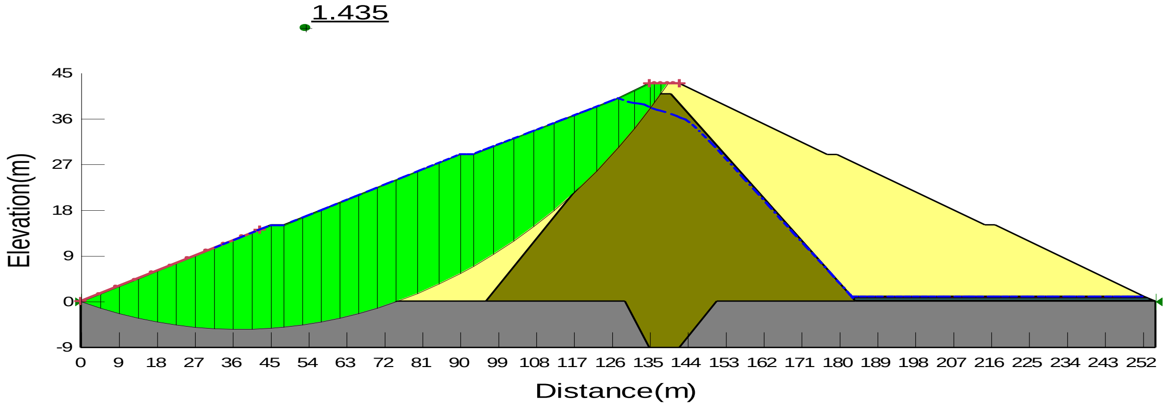

Figure 4.19: Factor of safety for upstream slope by spencer's method for rapid drawdown

4.3.3 End of construction

The stability of both upstream and downstream slopes during end of construction has been analyzed and a minimum factor of safety of 2.980 (Fig. 4.22) and 2.599 (Fig. 4.26) found for upstream and downstream slopes respectively. However, it satisfies the minimum requirement that the factor of safety during end of construction should not less than 1.30 (USACE, 2003).

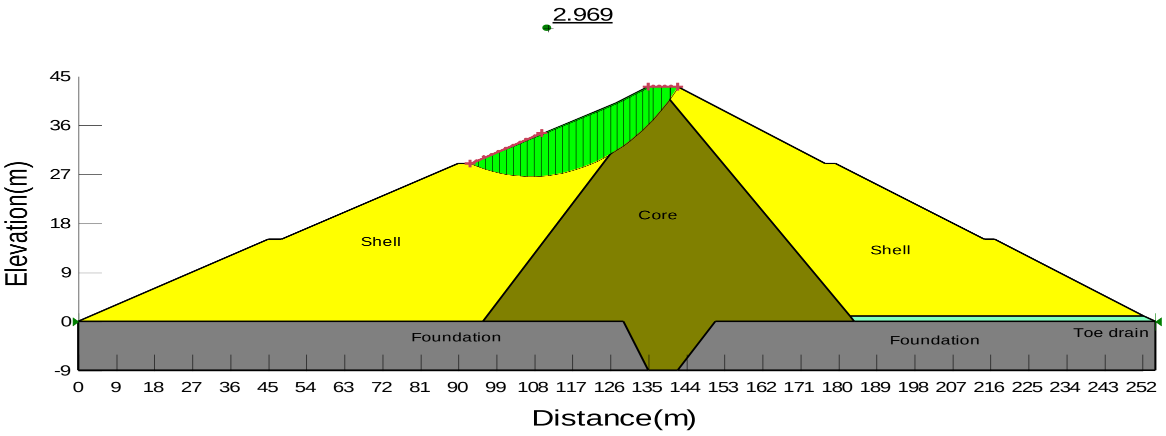

Figure 4.20: Factor of safety for upstream slope by bishop method at end of construction

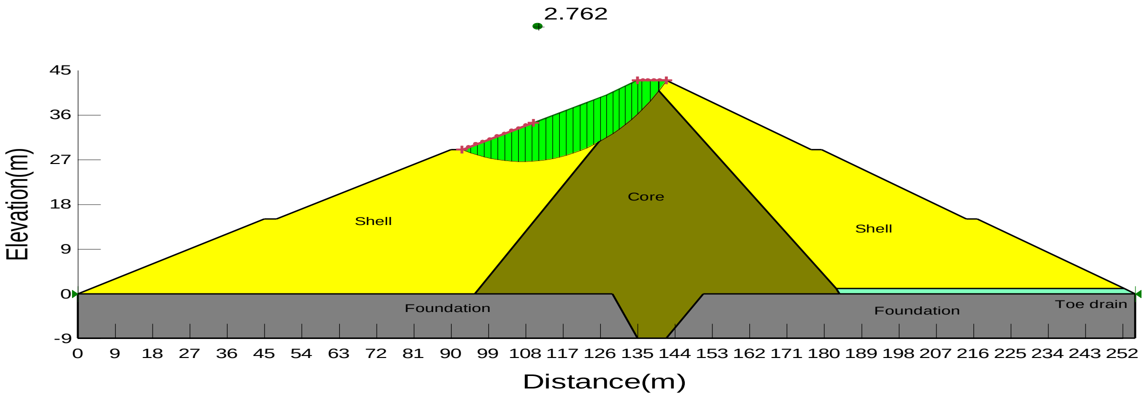

Figure 4.21: Factor of safety for upstream slope by Janbu method at end of construction

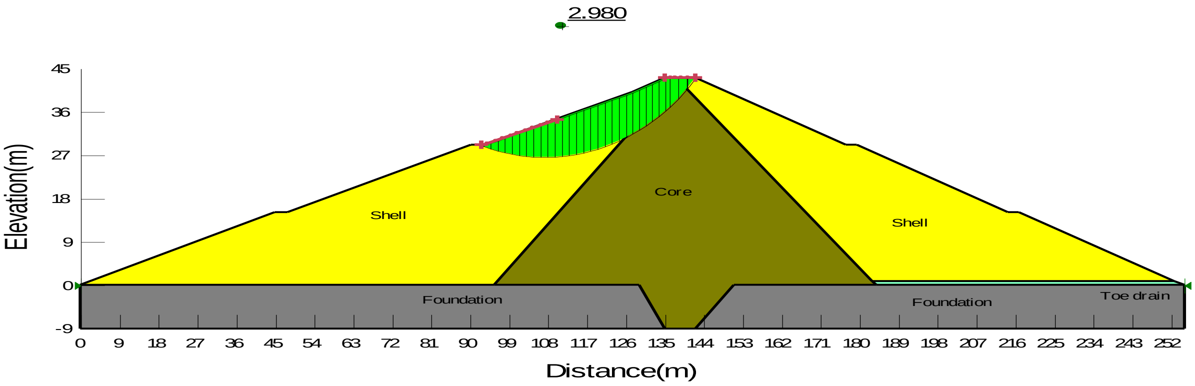

Figure 4.22: Factor of safety for upstream slope by Morgenstern-price’s method at end of construction

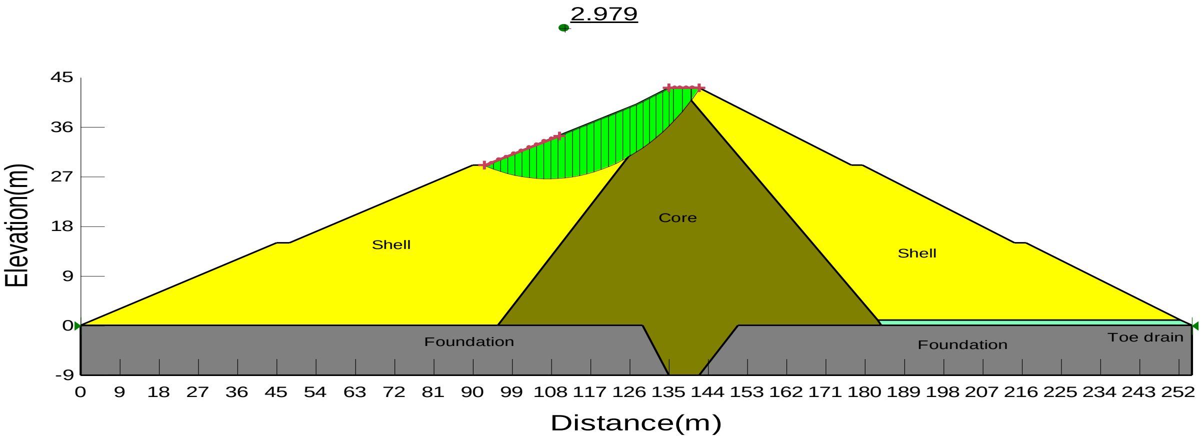

Figure 4.23: Factor of Safety for Upstream Slope by spencer's method at End of Construction

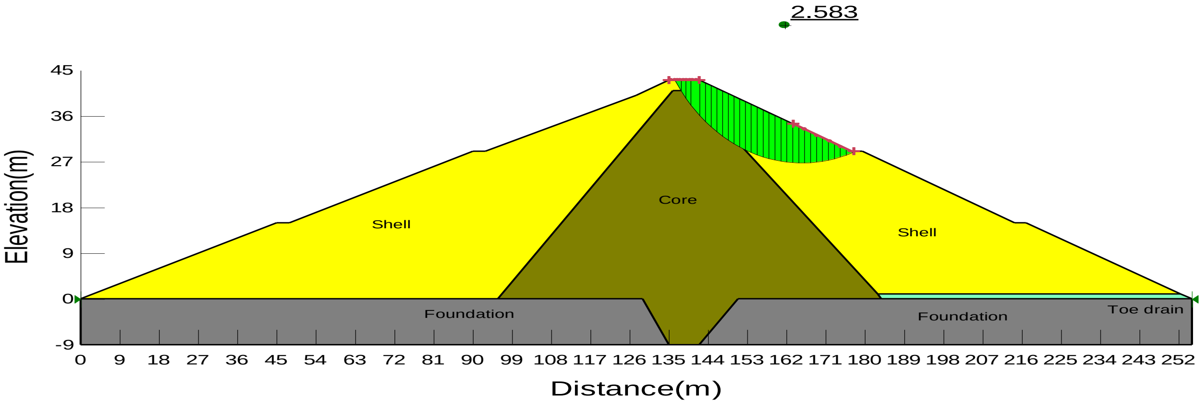

Figure 4.24: Factor of safety for downstream slope by Bishop's method at end of construction

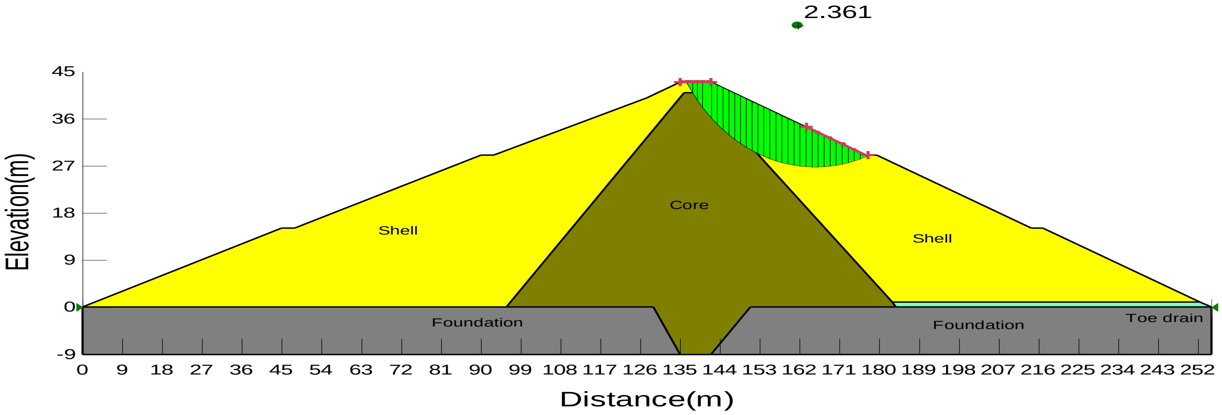

Figure 4.25: Factor of safety for downstream slope by Janbu's method at end of construction

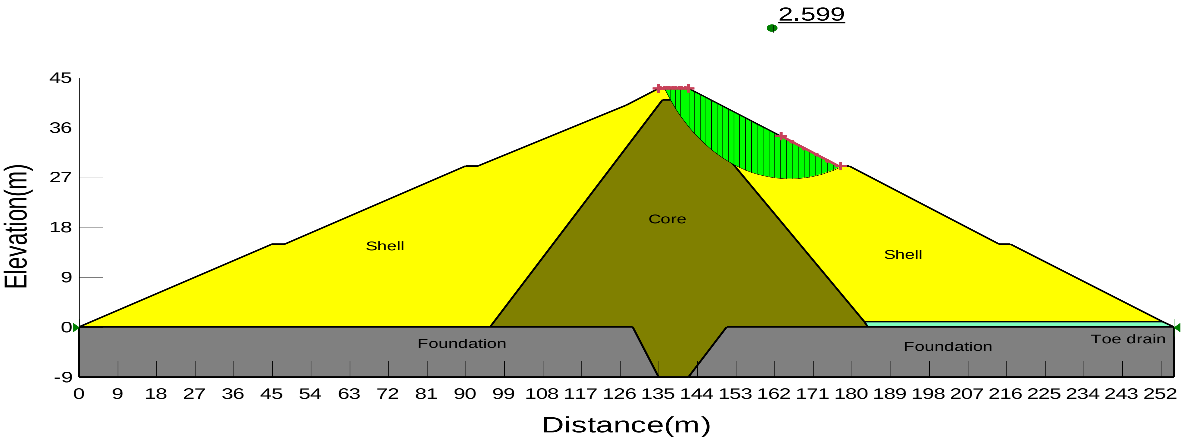

Figure 4.26: Factor of safety for downstream slope by Morgenstern-price's method at end of construction

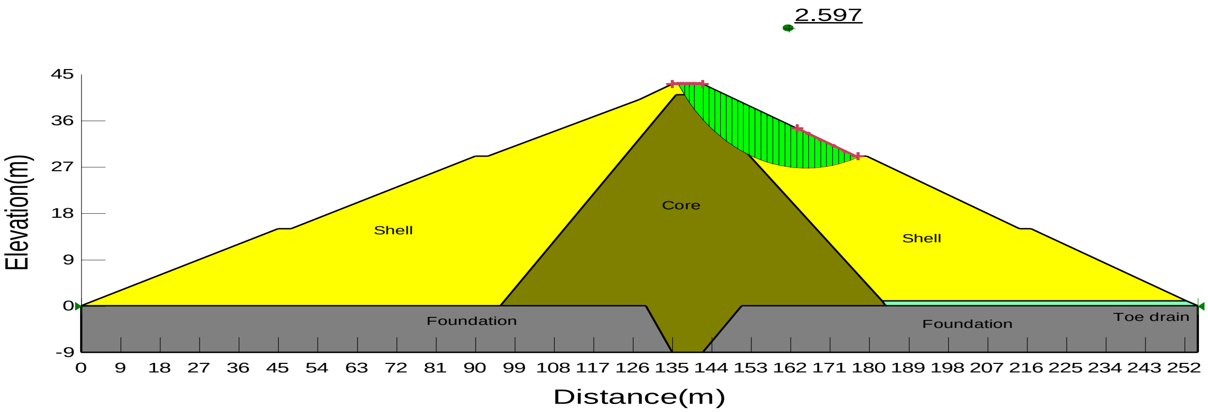

Figure 4.27: Factor of safety for downstream slope by spencer method at end of construction

4.3.4 Construction stage

The stability of the embankment has also been analyzed during construction stages and the respective factor of safety has been computed. In this study for slope stability computation, it has assumed that the embankment to be constructed in two stages and the upstream and downstream slopes has been analyzed for both conditions. Based on the computation, the minimum factor of safety during construction stage has been found to be 1.696and 1.604 (Fig4.14 and Fig 4.15) at upstream and the downstream respectively. According to USACE, (2003), the minimum required factor of safety during construction stage is 1.3. Hence, the revised section of the dam will be safe against structural failure during construction. The results computed for construction condition has been presented through Figure A. 6 to Fig. A. 8 of Appendix A for further reference

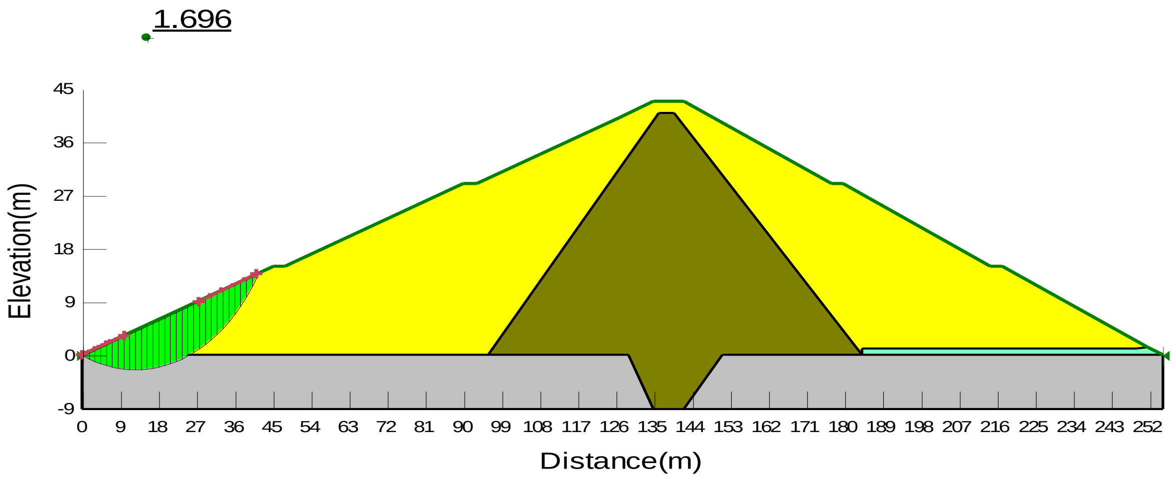

Figure 4.28: Factor of safety upstream slope during first stage construction

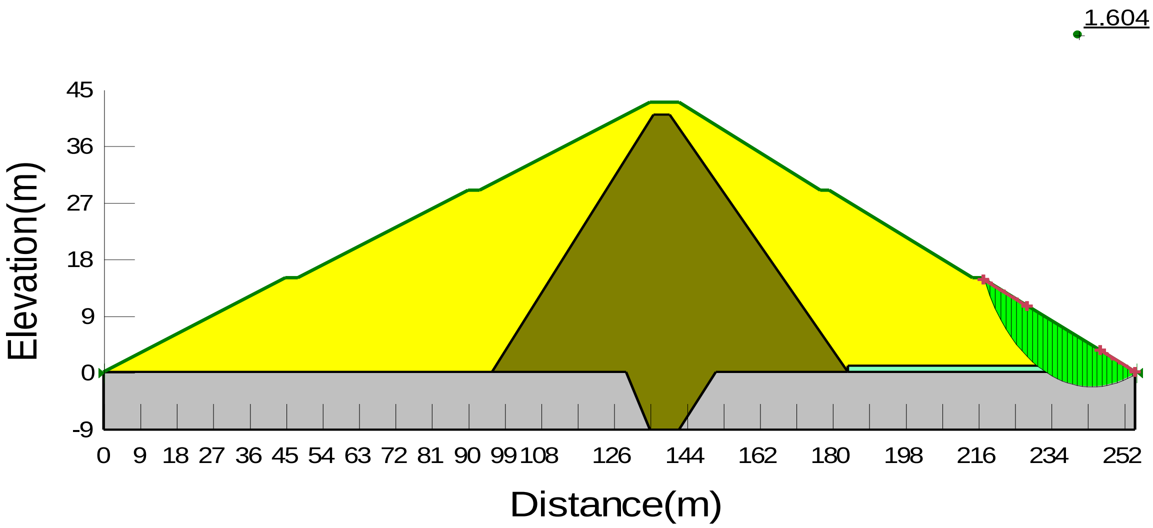

Figure 4.19: Factor of safety downstream slope during first stage construction

Table 4.2: Results of Dora dam stability analysis by geo-studio with limits

Critical stability Condition	Steady state		End of construction		Rapid drawdown
	Upstream	Downstream	Upstream	Downstream	Upstream
USACE(2003)	1.5	1.5	1.3	1.3	1.3
BDS (1994)	(1.5-1.3)		(1.5-1.3)		(1.3-1.2)
Bishop's	2.520	1.767	2.969	2.583	1.432
Janbu's	2.349	1.536	2.762	2.361	1.339
Morgenstern-Price	2.525	1.795	2.980	2.599	1.432
Spencer	2.527	1.792	2.979	2.579	1.435
Remark	Stable	Stable	Stable	Stable	Stable

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This research paper shows, the seepage and slope stability analysis of embankment dam via using the state of the art Finite Element Method based computer program via Geo- studio (2007) software, case of Dora high dam the following has been made.

- The quantity of seepage through dam body and dam foundation is found to be significance as compare to standards 0.03 (Jansen, 1988).
- The downstream slope has been found stable for all loading conditions i.e. steady state condition; end of construction and during construction stages.
- The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and the factor of safety of downstream slope during steady state condition; end of construction and during construction stages are computed, 1.795, 2.599 and 1.604 respectively.
- Similarly, the upstream slope has been found stable for all loading conditions i.e. sudden drawdown; end of construction and during construction.
- The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and the factor of safety of upstream slope during steady state, sudden drawdown; end of construction and during stage of construction, computed, 2.525, 1.432, 2.980 and 2.599 respectively.

The Dora dam project planned to harvest 17,132,955.6 m3 of water from a catchment area of about 78.03 km2 to introduce a modern irrigation practice downstream side of the dam.

water loss by seepage through embankment and dam foundation

The result of seep/w analysis showed that 1196589.3696 m3/yr. seepage loss during steady state case and 1080335.0592m3/yr. seepage loss through the dam during transient state analysis.

5.2 Recommendations

The seepage is appeared significance when it compare to the standard that is (0.0379436) software simulated, (0.034987767 trial-1, 0.04554062 trial-2 and 0.046814967 trial-3) d/s measured and 0.03 standard, (Jansen, 1988) based on this I recommended that at u/s face clay soil will be covered and at the same u/s blanket also construct if the dam is significance important? Compare to the cost maintenance.

The stability analysis performed showed that all the critical slip surfaces pass through the dam shell, which indicates that this zone represents the weaker zone within the dam, which has to be especially treated. Downstream of the dam is still not covered, it is recommended giving quick response to cover downstream of the dam by or tree and grass.

During slope stability analysis different methods are performed (Bishop's, Janbu's, Spencer, Morgenstern-price), Bishop's and Janbu's only consider Vertical force and Moment not considered for horizontal force, but Morgenstern-price consider all cases then as recommendation future researcher to choose this method for analysis.

The dam Dora is constructed for the purpose of an irrigation and no longer use for the purpose now. However, for the minimizing of seepage not any maintenance works are carried out during the past, Continuous analysis and maintenance will be strongly recommended to protect the nearest people from the flood hazard.

Finally for designers of embankment dam it is recommend to use geo-studio software for detecting amount of seepage through the body of dam and foundation, and for checking of slope stability as best option.

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7. APPENDIXES

7.1 Appendix-A

A Moisture Content Test determination

Procedure:

A. Clean and dry the moisture can (container). Make sure that all are marked the same reference no. or letter

b) Weigh each container and record.

c) Prepare five or more different soil samples which have 2000g mass and shake with water properly for all soil samples using deferent amount of water.

d) Fill the soil sample which are shake with water in the Mould and compact by Hammer(Mortar) 25 times without apply any force (simply dropping the Hammer).

e) With spatula (knife), the soil should be levelled or cut the excess soil from mould and clean using Brush.

f) Record weight of the compacted wet soil with Mould.

g) Take wet sample from compacted soil for determination of moisture content.

h) Record weight of wet sample + container.

i) Place the wet sample + container in the oven. Maintain in the required temperature normally 105-110^{0C} for 12- 24 hours.

j) Remove the sample from the oven and allow in the air to cool at least 10-15 min

k) Take weight of dry sample + container.

Calculation

The moisture content of a soil or aggregate is expressed as a percentage of its dry mass.

M.C (%) =$\frac{(Wt\ of\ wet\ sample+cont.)-(Wt\ of\ dry\ sample+cont.)}{(Wt\ of\ dry\ sample+cont.)-Wt\ of\ cont.}\,x100$

Wet density (g/cm3) =$\frac{(Wt\ of\ wet\ sample)}{volume\ of\ mould}x100$

Dry density (g/cm3) = $\frac{(\text{Wet density})}{moisture\ content+100}x100$

Finally, draw graph by taking Dry density vs. Moisture content.

Date sampled: - may 9/2018 Type of test: - standard proctor

Test Date tested: - April 15/2018 Volume of mould: - 944cm3

Table A.1: recorded data's and calculation results of proctor test for core

test no.	1	2	3	4
can no.	8	21	25	27
mass of can,W1(g)	57.2	57.2	61.3	16.7
mass of can+moist soil, W2(g)	75	77.4	69.8	61.7
mass of can+dry soil, W3(g)	68.4	70.7	67.3	51.2
Moisture content, w (%) =$\frac{W2-W3}{W3-W1}X100$	58.93	49.26	41.67	30.17
number of blows, N	15	21	29	38

Table A. 2: plastic limit test determination of core

Can No.	103	
Tset no.	1	2
mass of can,W1(g)	17.9	18
mass of can+moist soil, W2(g)	23.4	22.9
mass of can+dry soil, W3(g)	22.2	22
Plastic limit, PL =$\frac{W2-W3}{W3-W1}X100$	27.9	22.5
Take the average	25.5	

To calculate the plastic index, PI

PI=Liquid limit-plastic limit=45.9-25.2=20.7

Table A. 3 recorded data's and calculation results of proctor test for shell

test no.	1	2	3	4
can no.	8	21	25	27
mass of can,W1(g)	56.7	58.1	60.6	58.9
mass of can+moist soil, W2(g)	65.8	70.6	75.4	67.9
mass of can+dry soil, W3(g)	63.4	67.6	72.2	66.1
Moisture content, w (%) =$\frac{W2-W3}{W3-W1}X100$	35.82	31.58	27.59	25.00
number of blows, N	16	22	31	37

Table A. 4: plastic limit test determination of shell

Can No.	103	
Test no.	1	3
mass of can,W1(g)	15.4	14.6
mass of can+moist soil, W2(g)	24.5	22.5
mass of can+dry soil, W3(g)	23.2	24.2
Plastic limit, PL =$\frac{W2-W3}{W3-W1}X100$	16.7	14.6
Take the average	15.65	

To calculate the plastic index, PI

PI=Liquid limit-plastic limit=30.8-15.65=15.2

B. Liquid Limit Test determination

Procedure

a. Adjustment of mechanical device (CASAGRANDE device):-

- ✓ The apparatus must be clean and the bowl must be dry and oil free.
- ✓ Check that the grooving tool is clean and dry, and conforms to the correct profile.
- ✓ The machine should be placed on a firm solid part of the bench so that it will not wobble.

b) Mixing: -

the soil sample shall be placed in the evaporating(mixing) dish and add sufficient distilled water and mix the soil sample in the mixing dish with the spatula for at least 10min.

c) Turn the crank handle of the machine at a steady rate of two revolutions per second, so that the bowl is lifted and dropped. Continue turning until the groove is closed along. The groove is closed when the two parts of the soil come in to contact at the bottom of the groove. Record the number of blows required to reach this condition. If there is a gap between two points of contact, continue until there is a length of continuous contact of 13mm, and record the number of blows.

d) Remove a slice of soil approximately the width of the spatula extending from edge to edge of the soil. Followed together shall be removed and placed in two suitable containers. The containers and samples are weighed and the weight recorded.

e) Wash, clean, and dry the cup and grooving tool in preparation for the next trials.

f) The foregoing operation shall be repeated for at least two additional portions of the samples to which sufficient water has been added to bring the soil to a more fluid condition. The object of this procedure is to obtain samples of such consistency that at least one determination will be made in each of the following number of blows; 1st 11, 2nd 12.6, 3rd 13.5, 4th 14.3 and 15.7.

g) Place all the weighed, recorded sample and container in the oven to dry.

Procedure: -

a) Preparation of a small portion of thoroughly mixed sample

b) Roll in to ball

c) Roll in to thread until crumbling occurs (3mm) and take six or eight pieces

d) Gather the pieces together after crumbling stage is reached. Divide in to two parts and place in a suitable moisture can (container). Then, take weight of container and wet soil. e) Place the moisture can and wet sample in the oven.

f) Maintain the required temperature normally 105-110^{0}c for 12-24 hours.

g) Remove the sample from oven and allow in the air for about 5-10 min. record the weight of dried sample and moisture can.

D. proctor Compaction Test of core materials

1) Put a metal plate having 15cm diameter on randomly selected compacted part for test and dig a hole up to 15cm depth;

2) Take the soil gained from the hole and record the weight;

3) Record the Wt. of sand + Jar before pouring;

4) Fill the hole slowly by the prepared fine sand found in the Jar which can replace the compacted soil;

5) Take the remain fine sand with Jar from the hole, and record i.e. Wt of sand + Jar after pouring;

6) Record weight of sand in the cone;

7) Calculate Wt. of sand in the hole;

8) Calculate bulk density: -

Bulk density = $\frac{Wt\ of\ wet\ soil\ from\ sample}{volume\ of\ water\ replace}$

9) Take a sample by moisture can (tin) and record Wt. of wet soil + tin;

10) Dry the soil by the help of fire and record Wt. of dry soil + tin;

11) Record weight of tin, calculate Wt. of wet soil and Wt. of dry soil;

12) calculate the values of moisture content, dry density, and degree of compaction;

E**. Generally proctor Compaction Test**

- General procedures in compaction test of shell material are: -
- Dig a hole approximately 30cm*30cm and
- Record weight of wet sample taken from the hole;
- Enter the plastic material on the hole;
- Replace water to the hole and record the volume of water it takes in litter;

Calculate bulk density:-bulk density=$\frac{\text{weight of wet sample}}{\text{volume of water replacement}}$

Take wet sample by can and record the weight (weight wet sample + can);

Dry it by fire and record the weight (weight of dry sample + can);

Take weight of can, calculate weight of water and weight of dry sample;

Calculate moisture content and dry density as below:-

M.C (%) = $\frac{\text{wt of water}}{\text{wt of dry samole}}$x100

Dry density= $\frac{\text{bulk density}}{\text{moisture content+100}}$x100

Table A. 5: Water Content and Dry Density of Core Material

Item	Test number				
	1	2	3	4	5
Volume of mould, v (mm3)	1000	1000	1000	1000	1000
Weight of mould,w1(g)	3895	3895	3895	3895	3895
Weight of mould+compacted soil w2+w1(g)=w2	5420	5595	5671	5682	5565
compacted soil w2-w1(g)=w	1525	1700	1776	1787	1670
Bulk density, γb =$\frac{W}{V}$ (g/cm^3)	1.62	1.8	1.88	1.89	1.77
Water content (%)	11	12.6	13.5	14.3	15.7
Dry density γd =$\frac{\gamma b}{1+W}$	1.650	1.736	1.819	1.789	1.698

Table A. 6: Water Content and Dry Density of Shell

Item	Test number				
	1	2	3	4	5
Volume of mould, v (mm3)	1000	1000	1000	1000	1000
Weight of mould,w1(g)	4810	4810	4810	4810	4810
Weight of mould+compacted soil w2+w1(g)=w2	5420	5595	5671	5682	5565
compacted soil w2-w1(g)=w	1525	1700	1776	1787	1670
Bulk density, γb = $\frac{W}{V}$ (g/cm^3)	1.62	1.8	1.88	1.89	1.77
Water content (%)	7.4	8.9	11.4	14.3	17
Dry density γd =$\frac{\gamma b}{1+W}$	1.705	1.795	1.854	1.789	1.680

G. Permeability

Permeability is one of the most important properties that Hydrologist, Geotechnical engineers and ground water professionals always deal with (Cedergren 1989). Naturally all the soil materials are *permeable*, means water can flow through the soil by the interconnected pore spaces in the soil. The quantity of permeability is always denoted by the term Coefficient of Permeability (k).

Procedure

1. Supply water using a plastic tube from the water inlet to the burette. The water will flow from the burette to the specimen and then to the funnel. Check to see that there is no leak. Remove all air bubbles.

2 Allow the water to flow for some time in order to saturate the specimen. When the funnel is full, water will flow out of it into the sink.

3. using the pinch cock, close the flow of water through. The specimen. The pinch cock is located on the plastic pipe connecting the bottom of the specimen to the funnel.

4. Measure the head difference, h1 (cm). Note: Do not add any more water to the burette.

5. Open the pinch cock. Water will flow through the burette to the specimen and then out of the funnel. Record time (t) with a stop watch until the head difference is equal to h2 (cm) . Close the flow of water through the specimen using the pinchcock.

Falling head permeability for core

Table A.7: falling head permeability test determination of coefficient of permeability core

Test No.	1	2	3
Diameter of sandpipe reading, d (cm2)	1.02	1.02	1.02
Diameter of specimen, D (cm)	10	10	10
Area of sandpipe reading a (cm2)	0.79	0.79	0.79
Area of sample A:(cm2)	78.53	31.67	31.67
Height of travel through sample, L: (cm)	12.73	12.75	12.75
Height of initial head, h1 (cm)	170	168.2	168.9
Height of final head, h2 (cm)	168.2	148	145
Time, t (second)	480	1500	640
K= 2.303.$\frac{axL}{Axt} logh1/h2$	2.84 E-06	1.95 E-06	2.130 E-06
Average, K=	**2.306x10-6**		

Shell material

Table A.8: falling head permeability test determination of coefficient of permeability shell

Test No.	1	2	3
Diameter of sandpipe reading, d (cm2)	1.02	1.02	1.02
Diameter of specimen, D (cm)	6.35	6.35	6.35
Area of sandpipe reading a (cm2)	0.79	0.79	0.79
Area of sample A:(cm2)	31.67	31.67	31.67
Height of travel through sample, L: (cm)	18.6	18.6	18.6
Height of initial head, h1 (cm)	174	168.2	170
Height of final head, h2 (cm)	140	142.4	156
Time, t (second)	240	245	280
K= 2.303.$\frac{axL}{Axt} logh1/h2$	4.1586x10^{-4}	3.154x10^{-4}	1.4244x10^{-4}
Average, K=	**2.912x10^{-4}**		

Filter material

Table A.9: Determination of coefficient permeability Filter material

Test No.	1	2	3
Diameter of specimen, D (cm)	6.35	6.35	6.35
Length of specimen, L (cm)	13.2	13.2	13.2
Area of specimen, A (cm2)	31.67	31.67	31.67
Beginning head difference, hi (cm)	85.0	76.0	65
Ending head difference, h2 (cm)	25	21	21
Test duration, t (s)	16.3	16.3	15.3
k = 2.303.$\frac{axL}{Axt} logh1/h2$	0.036	0.038	0.036
Average k (cm/se)	0.037		

H. Determination of Shearing Strength of Soil

In the direct shear test, a sample of soil is placed into the shear box. After the specimen is placed in the box, and all the other necessary adjustments are made, a known normal load is applied (Braja M. Das, 2002). Then a shearing force is applied. The normal load is held constant throughout the test but the shearing force is applied at a constant rate of strain will be explained later on).The shearing displacement is recorded by a dial gauge.

Normally, the plotted points of normal and shearing stresses at failure of the various specimens will approximate a straight line. However, it is the usual practice to draw the best straight line through the test points to establish the Coulomb Law. The slope of the line gives the angle of shearing resistance and the intercept on the ordinate gives the apparent cohesion**.**

Procedures

1. Place the shear box into the shearing device.

2. Apply a normal load to the specimen using the load transfer plate and the loading hanger.

3. Remove the alignment screws from the shear box.

4. Turn the gap screws a half turn clock wise and then tow turns counter clockwise. This should increase the gap between the two halves of the shear box.

5. Set the shearing device the advance at a rate of 0.50 mm/min.

6. Begin the data acquisition system. Record the input voltage output and voltage from the channels needed.

7. Once data acquisition has begun start the shearing device.

8. Record the transducer output every 30 seconds after it has begun to change from the initial reading. Continue recording the output until the output is constant or drops for three consecutive readings.

9. Stop the data acquisition system.

The dimension of the box width =5.08cm,length = 5.08cm, height, =3.33cm

Calculate the nominal shear stress, acting on the specimen as follows:

$\tau \quad = \frac{F}{A}$

Where:

τ = nominal shear stress

F = shearing force

A = initial cross sectional area of the specimen.

Calculate the normal stress action on the specimen

$\sigma_n \quad = \frac{N}{A}$ where:

σ n = normal shear stress

N = normal vertical force acting on the specimen.

Estimate the friction angle, ϕ, for the specimen by assuming the shear stress at failure is the maximum shear stress.

Estimate the friction angle, ϕ, by assuming the horizontal plane is the failure plane

Table A.10: Shear strength test of core material

Normal stress	0	30	40	60	70	180
shear stress	29	49	56	70	80	160

Table A.11: Shear strength test of core material

Normal stress	0	20	40	60	70	80
shear stress	25	45	64	80	90	100

Discharge measurement

Velocity-Area Method

The most practical method of measuring the discharge of a stream is the velocity-area method. Discharge is computed as the product of the area and velocity. The measurement is made by subdividing a stream cross section into segments (some-times referred to as partial areas, sections, subareas, verticals, stations, profiles, panels, or ensembles), and by measuring the depth and velocity in a vertical within each segment. The total discharge is the summation of the products of the partial areas of the stream cross section and their respective average velocities Turnipseed, D.P., and Sauer, V.B. (2010).

Procedure

This method measures surface velocity. Mean velocity is obtained using a correction factor. The basic idea is to measure the time that it takes the object to float a specified distance downstream.

V surface = travel distance/ travel time = L/t

Because surface velocities are typically higher than mean or average velocities V mean = k V surface where k is a coefficient that generally ranges from 0.8 for rough beds to 0.9 for smooth beds (0.85 is a commonly used value)

Step 1. Choose a suitable straight reach with minimum turbulence (ideally at least 3 channel widths long)

Step 2. Mark the start and end-point of your reach

Step 3. If possible, travel time should exceed 20 seconds.

Step 4. Drop the object into the stream upstream marker.

Step 5. Start the watch when the object crosses the upstream marker and stop the watch when it crosses the downstream marker.

Step 6. It should repeat the measurement at least 3 times and use the average in further calculations.

Step 7. Measure the cross-sectional areas at the start and end-point of reach. Use the water width and multiply it by the average depth as computed from 3 or 4 depth measurements made across the channel.

Average the cross-sectional areas: Using the average area and corrected velocity, we can now compute discharge; Q. Correction factors to convert surface velocity to average velocity typically range from 0.8-0.9. Many times 0.85 is used.

The total discharge with the confidence interval of 85% is 0.032714 m3/sec

Table A.12: Area velocity method measuring and data recorded (date: 5/6//2010)

trial	Trial-1	Trial-2	Trial-3
Time(sec)	19	18	19
Average time	(19+18+19)/3		18.67 sec
Channel length (m)	7.8	Channel width(cm)	60
Velocity (m/sec)	L/tave	7.8/18.67 = 0.417783	

Table A.13: Area velocity method measuring of discharge (trial-1)

distance from initial point(m)	width of interval(m)	Depth (m)	Area(m2)	velocity	Discharge,Q (m^3/sec) interval
0.1	0.05	0	0	0	0
0.2	0.05	0.495	0.02475	0.45	0.0111375
0.3	0.025	0.495	0.012375	0.45	0.00556875
0.35	0.05	0.15972	0.007986	0.1452	0.001159567
0.45	0.005	0.1595	0.0007975	0.145	0.000115638

0.46	0.005	0.45958	0.0022979	0.4178	0.000960063
0.47	0.005	0.55	0.00275	0.5	0.001375
0.48	0.005	0.55	0.00275	0.5	0.001375
0.49	0.005	0.55	0.00275	0.5	0.001375
0.5	0.005	0.55	0.00275	0.5	0.001375
0.51	0.005	0.55	0.00275	0.5	0.001375
0.52	0.005	0.55	0.00275	0.5	0.001375
0.53	0.005	0.495	0.002475	0.45	0.00111375
0.54	0.005	0.495	0.002475	0.45	0.00111375
0.55	0.005	0.495	0.002475	0.45	0.00111375
0.56	0.005	0.495	0.002475	0.45	0.00111375
0.57	0.005	0.495	0.002475	0.45	0.00111375
0.58	0.005	0.495	0.002475	0.45	0.00111375
0.59	0.005	0.495	0.002475	0.45	0.00111375
0.6	0.005	0	0	0	0
			0.0820314		0.034987767

Table A.14: Area velocity method measuring of discharge (date: 5/8/2010)

trial	Trial-1	Trial-2	Trial-3
Time(sec)	10	10	11
Average time	(19+18+19)/3		10.33
Channel length (m)	7.8	Channel width(cm)	60
Velocity (m/sec)	L/tave	0.75483871	

Table A. 15: Area velocity method measuring of discharge (at trial-2)

distance from initial point(m)	width of interval(m)	Depth (m)	Area(m2)	velocity	Discharge,Q (m^3/sec) interval
0.1	0.05	0	0	0	0
0.2	0.05	0.495	0.02475	0.45	0.0111375
0.3	0.025	0.495	0.012375	0.45	0.00556875
0.35	0.05	0.15972	0.007986	0.1452	0.001159567

0.45	0.005	0.1595	0.0007975	0.145	0.000115638
0.46	0.005	0.45958	0.0022979	0.4178	0.000960063
0.47	0.005	0.55	0.00275	0.75484	0.003133809
0.48	0.005	0.55	0.00275	0.75484	0.003133809
0.49	0.005	0.55	0.00275	0.75484	0.003133809
0.5	0.005	0.55	0.00275	0.75484	0.003133809
0.51	0.005	0.55	0.00275	0.75484	0.003133809
0.52	0.005	0.55	0.00275	0.75484	0.003133809
0.53	0.005	0.495	0.002475	0.45	0.00111375
0.54	0.005	0.495	0.002475	0.45	0.00111375
0.55	0.005	0.495	0.002475	0.45	0.00111375
0.56	0.005	0.495	0.002475	0.45	0.00111375
0.57	0.005	0.495	0.002475	0.45	0.00111375
0.58	0.005	0.495	0.002475	0.45	0.00111375
0.59	0.005	0.495	0.002475	0.45	0.00111375
0.6	0.005	0	0	0	0
			0.0820314		0.04554062

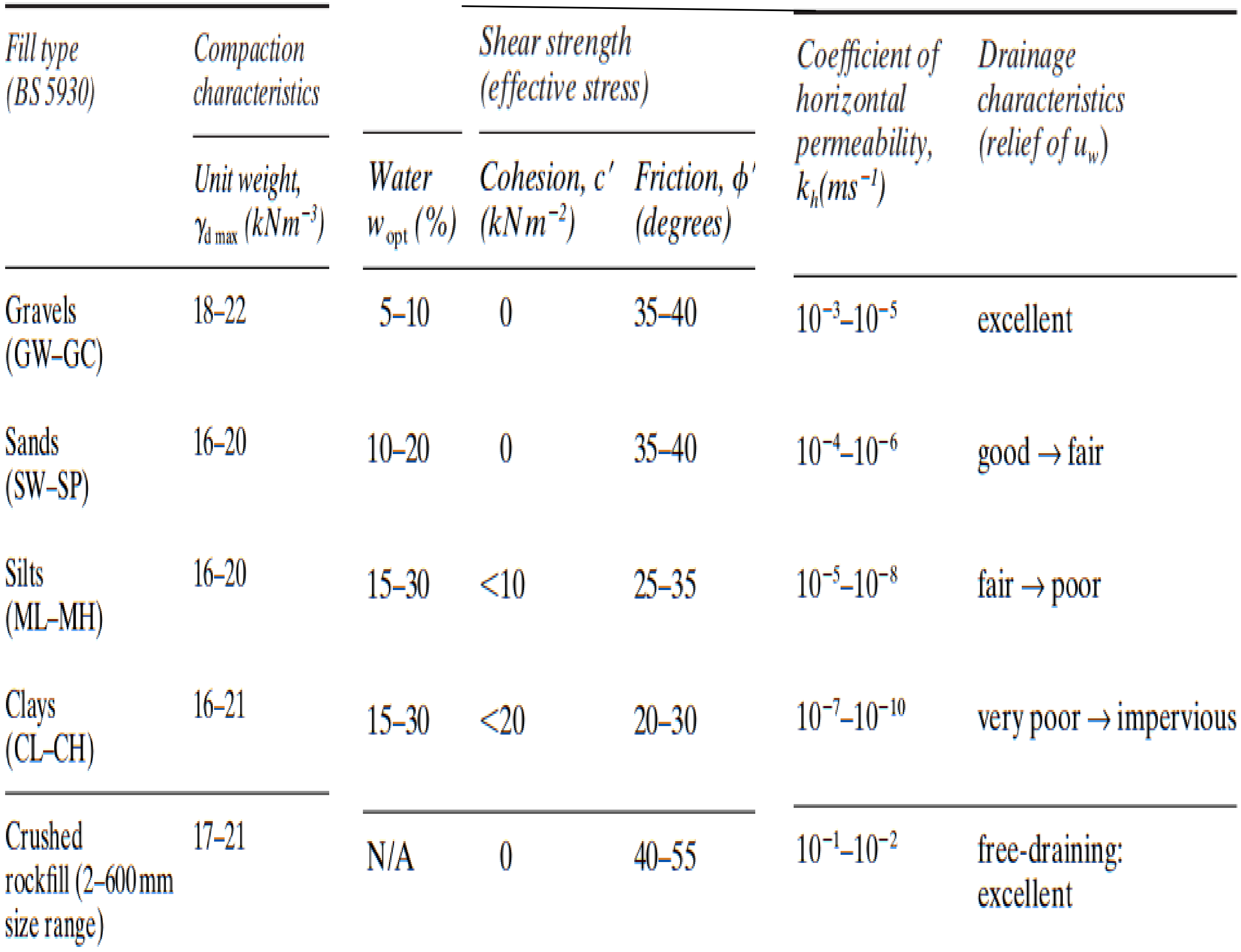
Table A. 16: Area velocity method measuring of discharge (date: 9/10/2010)

trial	Trial-1	Trial-2	Trial-3
Time(sec)	11	10	9
Average time	(19+18+19)/3		10.33
Channel length (m)	7.8	Channel width(cm)	60
Velocity (m/sec)	L/tave	0.78	

Table A. 17: Area velocity method measuring of discharge (at trial-3)

distance from initial point(m)	width of interval(m)	Depth (m)	Area(m2)	velocity	Discharge,Q (m^3/sec) interval
0.1	0.05	0	0	0	0
0.2	0.05	0.495	0.02475	0.45	0.0111375
0.3	0.025	0.495	0.012375	0.45	0.00556875
0.35	0.05	0.15972	0.007986	0.1452	0.001159567
0.45	0.005	0.1595	0.0007975	0.145	0.000115638
0.46	0.005	0.45958	0.0022979	0.4178	0.000960063
0.47	0.005	0.55	0.00275	0.5	0.0033462
0.48	0.005	0.55	0.00275	0.5	0.0033462
0.49	0.005	0.55	0.00275	0.5	0.0033462
0.5	0.005	0.55	0.00275	0.5	0.0033462
0.51	0.005	0.55	0.00275	0.5	0.0033462
0.52	0.005	0.55	0.00275	0.5	0.0033462
0.53	0.005	0.495	0.002475	0.45	0.00111375
0.54	0.005	0.495	0.002475	0.45	0.00111375
0.55	0.005	0.495	0.002475	0.45	0.00111375
0.56	0.005	0.495	0.002475	0.45	0.00111375
0.57	0.005	0.495	0.002475	0.45	0.00111375
0.58	0.005	0.495	0.002475	0.45	0.00111375
0.59	0.005	0.495	0.002475	0.45	0.00111375
0.6	0.005	0	0	0	0
			0.0820314		0.046814967

Table A.18: typical values of engineering property of soils and rocks (Novak, 2007)



Fill type (BS 5930)	Compaction characteristics		Shear strength (effective stress)		Coefficient of horizontal permeability, $k_h(ms^{-1})$	Drainage characteristics (relief of u_w)
	Unit weight, $\gamma_{d\max}$ (kNm^{-3})	Water w_{opt} (%)	Cohesion, c' (kNm^{-2})	Friction, ϕ' (degrees)		
Gravels (GW–GC)	18–22	5–10	0	35–40	10^{-3}–10^{-5}	excellent
Sands (SW–SP)	16–20	10–20	0	35–40	10^{-4}–10^{-6}	good → fair
Silts (ML–MH)	16–20	15–30	<10	25–35	10^{-5}–10^{-8}	fair → poor
Clays (CL–CH)	16–21	15–30	<20	20–30	10^{-7}–10^{-10}	very poor → impervious
Crushed rockfill (2–600mm size range)	17–21	N/A	0	40–55	10^{-1}–10^{-2}	free-draining: excellent

7.2 Appendix-B

Result Reports Software analysis for all loading condition

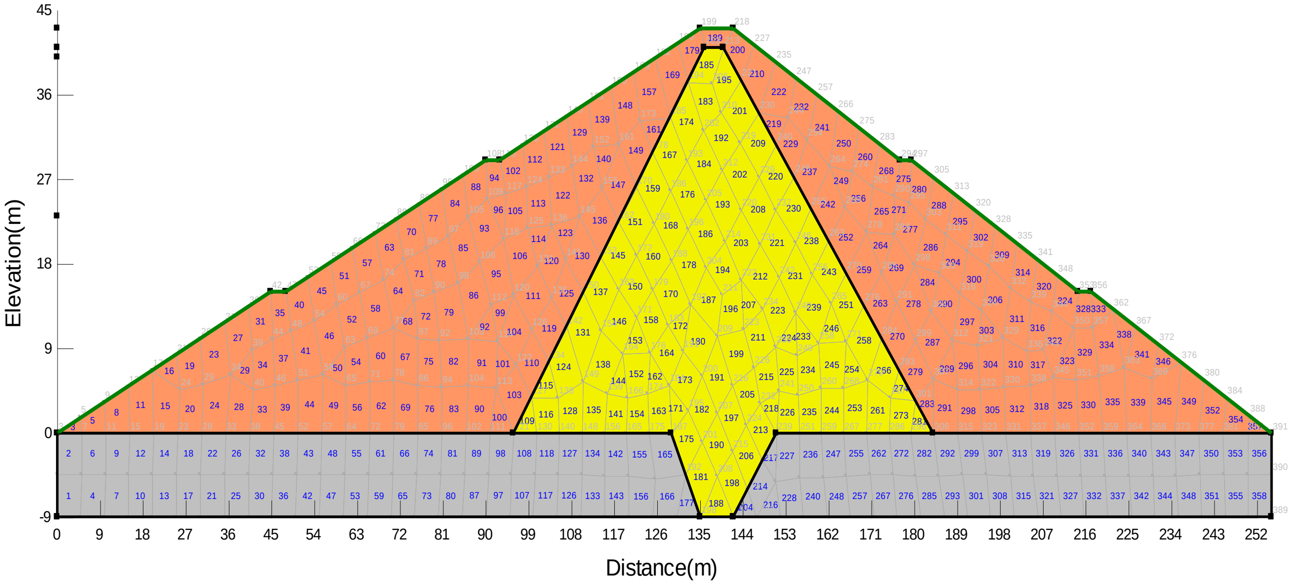


Figure B. 1: Plot of geometry model with significant nodes (2095 nods and 2031 elements)

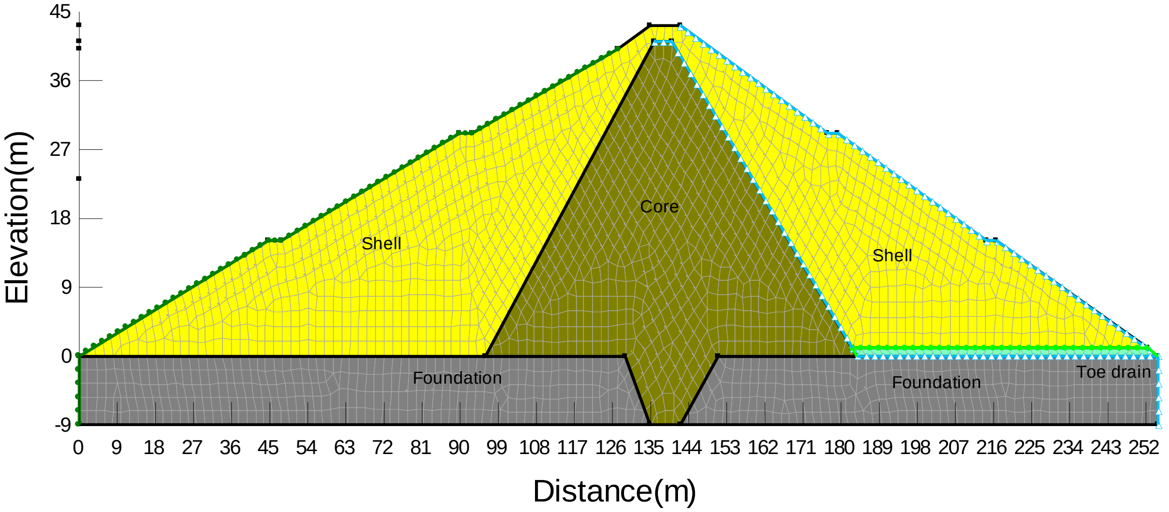


Figure B. 2: Finite element discretization and boundary condition applied for seepage analysis

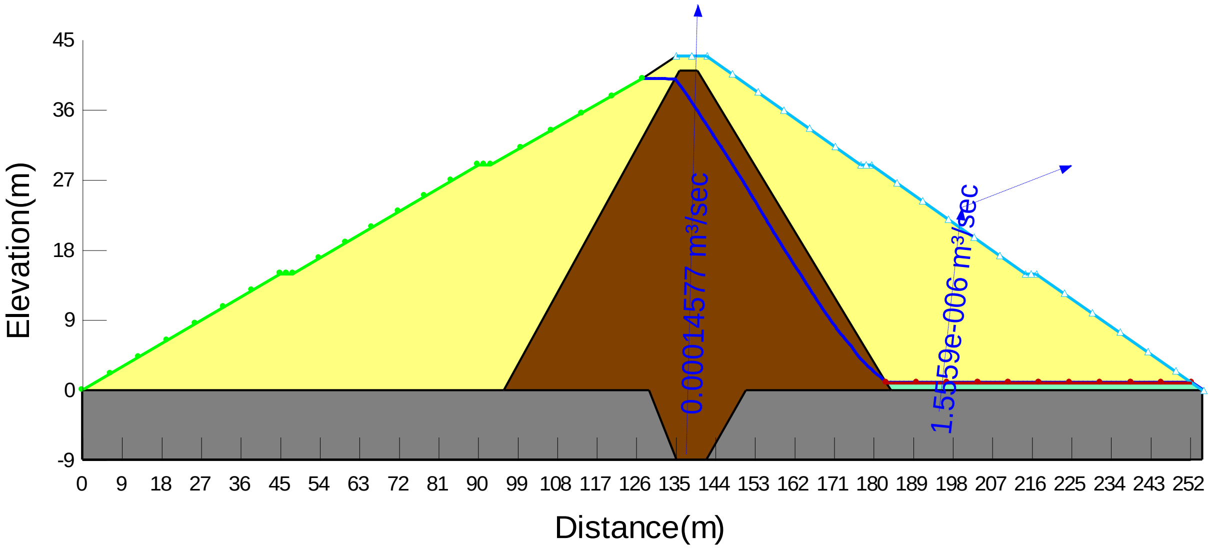


Figure B. 2: Steady state seepage quantification (using 2nd data)

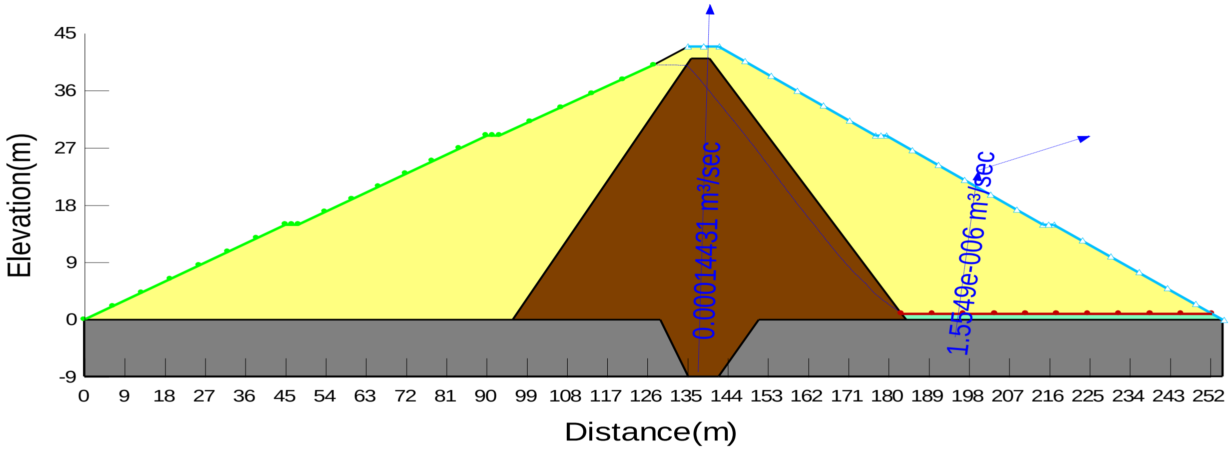


Figure 4.1: Transient seepage analysis (using 2nd data)

Table B. 1: Software Report for Regions

	Material	**Points**	**Area (m²)**
Region 1	shell	1,2,3,4,5,29,6,7,8,9,10,11,28,27,14,15,16	3664.7433
Region 2	core	16,15,14,27,13,20,19,18,17	2016.4512
Region 3	foundation	1,21,18,17,16	1188
Region 4	foundation	20,13,12,22,19	976.5
Region 5	filter	27,28,12,13	70.268653

Table B. 2: Slip slices steady state condition of upstream of the dam

Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	2.5352625	-1.200037	404.03832	430.27781	9.5503952	13
2	7.6057875	-3.4412165	426.01975	494.43248	24.900197	13
3	12.67631	-5.37447	444.96262	551.24539	38.683767	13
4	17.746835	-7.01747	461.06016	600.87965	50.890133	13
5	22.81736	-8.3841795	474.43783	643.45008	61.515428	13
6	27.808545	-9	480.41021	654.88643	63.504153	13
7	32.72039	-9	480.34913	669.42273	68.81716	13
8	37.632235	-9	480.22698	683.53148	73.996787	13
9	42.54408	-9	480.06411	697.17198	79.020803	13
10	46.5	-9	479.9	698.03333	79.39404	13
11	50.22097	-9	479.67812	708.94747	83.447218	13
12	54.662905	-9	479.36294	720.29387	87.691686	13
13	59.10484	-9	478.9352	731.28007	91.84602	13

14	63.54678	-9	478.3949	741.95109	95.926609	13
15	67.988715	-9	477.697	752.30693	99.94984	13
16	72.68347	-8.400687	470.89773	724.35982	92.252659	13
17	77.63105	-7.073594	456.63898	706.15195	90.815293	13
18	82.57863	-5.483901	439.50645	683.79546	88.913929	13
19	87.52621	-3.618792	419.33512	657.30424	86.613676	13
20	91.5	-1.9347845	401.17843	627.46165	82.360354	13
21	94.281155	-0.6269865	387.31809	607.18044	80.02335	13
22	95.99963	0.2239363	378.48925	582.53069	115.91167	29
23	98.655725	1.6652538	363.0446	567.48439	86.779542	25
24	103.0933	4.244067	334.61056	527.97055	82.076448	25
25	107.5309	7.1247035	302.85749	484.37117	77.047987	25
26	111.9685	10.338344	267.62323	436.15262	71.53648	25
27	116.40605	13.92511	228.726	382.62739	65.327264	25
28	120.8436	17.93789	185.96709	322.89686	58.12324	25
29	125.2812	22.448605	139.25528	255.81443	49.476423	25
30	129.375	27.11291	92.440526	193.13931	42.744096	25
31	133.125	31.949765	44.968097	133.91546	37.755916	25
32	135.5	35.254265	11.938733	89.431499	32.893727	25
33	137.55765	38.497975	-21.647162	37.120812	15.75685	25
34	139.68535	42	-60.309523	-7.8770735	-4.4748006	29

Table B. 3: Slip slices steady state condition of downstream case

Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	135.4456	41.944265	-42.141699	-19.250714	-10.935928	29
2	135.9456	40.764905	-36.607227	-14.451858	-6.1344497	25
3	138	36.68281	-9.408061	39.52964	16.779337	25
4	141	31.064975	17.610267	113.21402	40.581387	25
5	143.96805	26.562305	37.439596	163.31836	53.432366	25
6	147.9041	21.297095	58.10873	214.00637	66.174622	25
7	151.84015	16.773355	72.418267	255.2282	77.598213	25
8	155.77625	12.82685	81.832505	290.16831	88.433302	25
9	159.7123	9.3524665	87.671683	320.89276	98.996475	25
10	163.64835	6.2781135	90.749361	348.76346	109.52049	25
11	167.58445	3.5519075	91.208331	374.5969	120.29131	25
12	171.5205	1.1351705	89.143609	398.9741	131.51524	25
13	175.24425	-0.8997295	85.495841	425.5784	123.77993	13
14	178.25	-2.379381	81.609001	450.40795	134.23184	13
15	181.21225	-3.6763085	78.349848	474.44108	144.16542	13
16	183.46225	-4.6007405	78.240156	487.34195	148.90088	13
17	185.89995	-5.4756495	79.941245	498.51238	152.34744	13
18	189.69985	-6.712404	84.985186	513.48578	155.96146	13
19	193.49975	-7.7567575	90.918839	526.02824	158.36687	13
20	197.29965	-8.6159195	96.962112	535.52285	159.62306	13
21	201.11215	-9	99.448562	591.54117	179.10706	13
22	204.93725	-9	98.774069	565.24116	169.78014	13
23	208.76235	-9	98.428979	537.37256	159.7624	13

24	212.58745	-9	98.251206	508.11836	149.17945	13
25	215.75	-9	98.172	492.92	143.67652	13
26	219.04815	-9	98.122672	475.99041	137.53261	13
27	223.14445	-9	98.086054	442.10618	125.21309	13
28	227.24075	-9	98.0592	407.68488	112.69453	13
29	231.21995	-8.6088795	94.1921	404.18072	112.82663	13
30	235.0821	-7.732582	85.548705	352.11904	97.023668	13
31	238.94425	-6.6648035	74.993462	293.49902	79.52952	13
32	242.8064	-5.3979145	62.428996	229.41314	60.777257	13
33	246.66855	-3.9223245	47.752419	161.09224	41.25232	13
34	250.5307	-2.226066	30.570672	89.918431	21.600818	13
35	253.7309	-0.660555	11.949099	29.784331	6.4914937	13

Table B. 4: Slip slices sudden drawdown condition of upstream case

Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	2.25	-0.622849	22.521886	33.506552	5.5969671	25
2	6.75	-1.783673	48.26186	90.264591	21.40146	25
3	11.25	-2.7768685	69.986411	144.61651	38.025935	25
4	15.75	-3.606293	89.83934	195.61753	53.896679	25
5	20.25	-4.2750775	108.1129	242.55223	68.500261	25
6	24.75	-4.7856895	124.81269	284.8014	81.518319	25
7	29.25	-5.139979	139.84733	321.85957	92.739868	25
8	33.75	-5.3392115	153.13267	353.44802	102.06577	25
9	38.25	-5.384093	164.61135	379.34586	109.4127	25
10	42.75	-5.2747825	174.23499	399.61278	114.83572	25

11	46.5	-5.0764215	180.90944	402.97789	113.14953	25
12	50.218345	-4.752325	186.00132	402.42266	110.27218	25
13	54.655035	-4.2380015	190.45789	409.01774	111.3618	25
14	59.091725	-3.569795	192.96871	411.27685	111.23355	25
15	63.528415	-2.7453065	193.50392	409.56669	110.08948	25
16	67.965105	-1.7615075	191.99537	404.31267	108.18107	25
17	72.401795	-0.6146805	188.37663	395.90674	105.74187	25
18	77.18345	0.816272	183.1075	385.74186	112.32206	25
19	82.31007	2.5668275	176.37014	373.55201	109.2997	25
20	87.43669	4.558901	167.95763	358.65605	105.70586	25
21	91.5	6.2955715	160.81937	337.8127	98.109003	25
22	95.39311	8.1536525	153.55376	315.40255	89.714251	25
23	100.17931	10.636687	144.1596	295.14794	83.694205	25
24	104.9655	13.37776	133.34289	272.62876	77.207421	25
25	109.75175	16.39628	120.80124	247.51181	70.236814	25
26	114.538	19.715955	106.40432	219.34714	62.605226	25
27	119.5733	23.57589	87.596374	188.00057	42.619054	29
28	124.85775	28.058555	64.120455	145.53461	34.558258	29
29	129.375	32.26061	41.869769	105.92458	27.189654	29
30	133.125	36.09924	17.846291	68.97439	21.702591	29
31	135.5	38.658855	-4.3344242	40.070357	17.008857	29
32	136.7685	40.110385	-15.9107	20.829497	8.8415967	29
33	138.3685	42	-26.907492	-1.8800797	-1.0421452	25

Table B. 5: Slip slices end of construction condition of upstream case

Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	115	92.8263	28.942345	0	6.9484627	3.9088689
2	115	93.825	28.63181	0	22.261342	12.523154
3	115	95.475	28.15983	0	49.646906	27.928947
4	115	97.125	27.754415	0	76.04837	42.781133
5	115	98.775	27.41374	0	100.86189	56.74002
6	115	100.425	27.136325	0	123.4764	69.461849
7	115	102.075	26.921005	0	143.35482	80.644485
8	115	103.725	26.76689	0	160.08413	90.055584
9	115	105.375	26.673355	0	173.41765	97.55638
10	115	107.025	26.640025	0	183.28452	103.107
11	115	108.675	26.666765	0	189.80288	106.77392
12	115	110.325	26.75368	0	193.21729	108.6947
13	115	111.975	26.901125	0	193.90138	109.07953
14	115	113.625	27.109695	0	192.26709	108.16016
15	115	115.275	27.380245	0	188.76552	106.19035
16	115	116.925	27.71392	0	183.81559	103.40576
17	115	118.575	28.112155	0	177.78849	100.0152
18	115	120.225	28.57673	0	170.99844	96.195447
19	115	121.875	29.10981	0	163.69036	92.084275
20	115	123.525	29.713985	0	156.03442	87.777415
21	115	125.175	30.39236	0	148.13357	83.332779
22	115	126.90035	31.18709	0	139.93226	59.975044
23	115	128.70065	32.11073	0	130.82626	56.072207
24	115	130.5005	33.139455	0	121.36483	52.017031

25	115	132.3003	34.28213	0	111.2871	47.697711
26	115	134.1001	35.54952	0	100.23049	42.958845
27	115	135.5	36.617095	0	88.157918	37.784532
28	115	137	37.89477	0	68.919669	29.539008
29	115	139	39.7605	0	40.724749	17.45465
30	115	140.0629	40.816255	0	24.292234	10.411665
31	115	141.0629	41.941375	0	5.1573955	2.9013012

Table B.6: Slip slices end of construct condition of downstream case

Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	136.05895	42	0	3.0497638	1.7156496	27.84
2	137.686	40.203445	0	31.949611	25.84	27.52
3	139.22865	38.69304	0	54.073778	13.693621	27.52
4	141	37.15846	0	76.240169	23.176051	27.52
5	142.68335	35.836405	0	91.828042	32.676578	27.52
6	144.05	34.876955	0	98.506	39.357549	27.52
7	145.41665	34.00042	0	104.82917	42.219726	27.52
8	146.78335	33.200445	0	110.99541	44.929839	27.52
9	148.15	32.471785	0	117.14051	47.572692	27.52
10	149.51665	31.810085	0	123.3456	50.206486	27.52
11	150.8759	31.2146	0	130.49237	52.865994	25.84
12	152.22775	30.68163	0	136.85139	73.408694	25.84
13	153.5796	30.20507	0	143.06685	76.985968	25.84
14	154.93145	29.782785	0	148.9617	80.482487	25.84
15	156.2833	29.412975	0	154.29514	83.798644	25.84
16	157.63515	29.09412	0	158.76997	86.798983	25.84

17	158.987	28.82497	0	162.01131	89.316305	25.84
18	160.3388	28.604495	0	163.60513	91.139723	25.84
19	161.69065	28.43187	0	163.13743	92.036327	25.84
20	163.0425	28.306465	0	160.20707	91.773223	25.84
21	164.39435	28.22783	0	154.47406	90.124745	25.84
22	165.7462	28.19568	0	145.74572	86.89963	25.84
23	167.09805	28.2099	0	133.94333	81.989492	25.84
24	168.4499	28.27055	0	119.19264	75.350035	25.84
25	169.80175	28.37784	0	101.77838	67.052014	25.84
26	171.1536	28.53215	0	82.163188	57.255591	25.84
27	172.50545	28.734045	0	60.882165	46.221034	25.84
28	173.8573	28.984275	0	38.54786	34.24936	25.84
29	175.2091	29.28378	0	15.73846	21.685161	25.84