EVALUATING THE STABILITY OF EMBANKMENT DAM BY INTRODUCING CONCRETE FACE ROCK FILL DAM (CASE OF LEGEMERA DAM SOUTH WOLLO, ETHIOPIA)

MSc. THESIS

BY

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HAWASSA, ETHIOPIA

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This is to certify that the thesis entitled "Evaluating the stability of embankment dam by introducing concrete face rock fill dam" is submitted in partial fulfillment of the requirements for the Degree of Masters of Science with Specialization in Dam Engineering, the Graduate Program of the Department of Hydraulic and Water Resources Engineering, and it has been carried out by Jemal Kassim Mohammed Id. No PGDEng 009/09 under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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As members of the Examining Board of the Final MSc Open defense, We certify that we have read and evaluated the thesis prepared by Jemal Kassim entitled as "Evaluating the stability of embankment dam by introducing concrete face rock fill dam" and we recommend that it can be accepted as fulfilling the thesis requirement for the degree of masters of science in Hydraulic and water resources engineering (Specialization in Dam Engineering).

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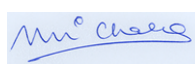
Michael M.(PhD)	 .	28 February 2019
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Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the school of graduates studies (SGS) through the school of graduate committee (SGC) of the candidate's department/school.

DEDICATION

To my father and mother, Kassim Mohammed and Kasanesh Mekonnen for your encouragement and support you provided me at my early age, and to my wife Fozia Musa Ahmed, for your deep love, affection and parenting us in a righteous way.

==

To a genius group of GEO-SLOPE International Ltd , Canada, where the Geo-Studio Software is developed for the first time, for their worldwide contribution of simplifying slope stability analysis. Indeed your contribution is great especially for the developing nation.
===

DECLARATION

I declare that this thesis is my original work. This thesis has not been presented for any other university, and that all sources of material used for the thesis have been cited and duly acknowledged.

Name: - Jemal Kassim

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LIST OF ACRONYMS AND SYMBOLS

ANRSWRDB	Amhara National Regional State Water Resource Development Bureau
AWWCE	Amhara Water Work Construction Enterprise
ACCRD	Asphalt Concrete Core Rock fill dam
ACFRD	Asphalt Concrete Face Rock fill dam
ATH	Acceleration Time History
DL	Dead Load
CFRD	Concrete Face Rock Fill Dam
ETB	Ethiopia Birr
FEM	Finite Element Methods
FDM	Finite Difference Method
ICOLD	International Commission on Large Dam
LEM	Limit Equilibrium Method
MDE	Maximum Design Earthquake
MPL	Maximum Pool Level
NPL	Normal Pool Level
OBE	Operating Base Earthquake
PGA	Peak Ground Acceleration
PVC	Poly Vinyl Chloride
RQD	Rock Quality Designation
USACE	United States Association of Civil Engineers
USBR	United States Bureau of Reclamation
USSD	United States Society on Dams
USSR	Union of Soviet Socialist Republic

ABSTRACT

Design of an embankment dam is a vital issue from the standpoint of safety, economy, controlling seepage, and speed of construction. Concrete face rock fill dams (CFRDs) virtually impervious, resistant to erosion and aging, workable, shorter period of construction and small volume of construction material. It is very well suited when there is no ample amount of impervious material within economically feasible hauling distance and severe weather conditions during construction. This study was aimed at addressing biggest challenge of hauling distance of the impervious clay core material by changing earth fill dam with central clay core to concrete face rock fill dam in case of Legemera micro earth dam, found in South Wollo. Universally recommended standard values beside to primary and secondary data essential for the work has been used. GeoStudio 2012 software was used for stability, seepage and deformation analysis of the dams. Consequently, by using this software good result of safety factor has been found for concrete face rock fill dam with regard to stability of the dam during maximum pool level, steady state condition and end of construction. During steady state condition, the upstream and downstream factor of safety computed for the dam is 1.954 and 1.544 respectively. Likewise, at the end construction, the upstream and downstream safety factor is 1.824 and 1.59 respectively. The seepage is also reduced to values of 2.42 $x10^{-6}$ m^3/sec by implementation of concrete face rock fill dam. Economic analysis result similarly shows that, concrete face rock fill dam is more economical by saving **22,441,387.75 ETB** due to fast rate construction of the dam independent of weather condition. Generally, application of CFRD can fulfill the basic requirement and minimum factor of safety under all loading condition, and dam of such types is seepage free.

Key word: Cay core, Concrete face rock fill dam, Economy, Embankment dam, Geo-studio

1. INTRODUCTION

1.1. Background

A dam is a barrier constructed across a river or a natural stream to create a reservoir for impounding water (for irrigation, water supply, flood protection), or to facilitate diversion of water from the river, or to retain debris flowing in the river along with water. There are a large number of dams and many of them are high with large reservoirs. Large dams are defined by International Commission on Large Dam (ICOLD) as dams exceeding 15 m in height or, in the case of dams of 10-15m height, satisfying one of certain other criteria, e.g. a storage volume in excess of $1x10^6 m^3$ or a flood discharge capacity of over $2000m^3/s$ etc. ICOLD divide dams into embankment dams (about 82.9% of the total number), concrete gravity dam 11.3%, arch dam 4.4 %, buttress dam 1% and multiple arch 0.4% dams (ICOLD, 1998)

According to United States Committee on Large Dams (USCLD, 1993) embankment dam can be designed as earth fill dam and rock fill dam based on available construction material, foundation condition of the site and total cost. Earth fill dams are the most common type of dam, principally because their construction involves the use of materials from required excavations and the use of locally available natural materials requiring a minimum of processing (USACE, 2004). Rock fill dams use an impervious membrane to provide water tightness and rock of all sizes to provide stability by resisting shear stresses resulting from externally applied loads, such as reservoir load, earthquake loads, and from internal forces caused by the weight of the soil and the embankment slopes. Based on the type and location of the membrane rock fill dam can be designed as concrete face rock fill dam (CFRFD), asphalt concrete face rock fill dam (ACFRD), Geo-membrane faced rock fill dam (GMF) and Asphalt concrete core rock fill dam (ACCD) (Wangxijiong , 2000).

The first embankment dam was built around 2000 B.C in Mesopotamia close to Baghdad. Because of lack of adequate design knowledge, construction equipment, new materials like cement and concrete and technology of construction, the establishment of large dam is a recent history. In addition, economic conditions and institutional capacity existing in countries that needed large dams did not enable them to take them up. The large dam construction became possible during the 20th century mainly because of advances made in Science and Technology, which enabled mechanization of construction processes and short period of construction time (USACE, 2004).

Since the development of construction technology rock fill dams with concrete facing adopted during the past 25-30 years have resulted in increased use of this type of rock fill dam and its adoption for higher dams. Concrete faced rock-fill dams, CFRD, is the term used to describe a certain type of dam that have a dam body of rock-fills or gravel that is compacted in layers. Moreover, there is an anti-seepage system using a concrete face slab. The concrete slab works like an impervious layer while the rock-fill body consists of granular material which has a high permeability and supports the concrete face slab by giving the dam stability.

Therefore, dam type selection, widely depends on geological condition of the site, foundation condition, economy, construction material availability and others. In view of the available construction materials, and the short dry season, a rock fill embankment with upstream concrete face is supposed optimum dam type. The most important advantage of proposing an earthen embankment for dam lies in the maximum use of locally available material with considerable low cost (Fell et al., 1992). However, the biggest challenge for Legemera dam design option is availability of core materials within economically justifiable hauling distance that meets the design requirement. So, identifying alternative solutions of construction materials comparing based on technical, economic, construction complexity, project schedule and purpose criterion would be the main finding of this thesis.

1.2. Statement of the Problem

From design report of Amhara National Regional State Water Resources Development Bureau 2007, Legemera Micro Earth Dam is one of the embankment dam types to be constructed at the selected dam site as an earth/rock fill with a clay core impervious membrane.

The nearest quarry area identified for dam core encountered social problems due to agricultural land of non-beneficiary kebeles from the project, safe water tightness for the reservoir area and few in quantity compared to the design requirement and also difficult for temporary route constructed during field work for transporting the drilling rig due to very steep slope topography. The location of suitable impervious clay which meets quantity and quality is reported to be greater than fifteen kilometer far from dam site which means expensive due to hauling. Rock fill materials burrow area for the casing/shell zone of the designed zoned dam is proposed just near to the quarry site for rock source area. This burrow area is found at the central portion of the mountain or a ridge located about 0.5 to 0.8 km North West of the dam site. Therefore, use of concrete face rock fill dam is an appropriate solution to avoid the abovementioned problems of unnecessary hauling cost of clay core.

Hence, this research considers the advantages of concrete face rock fill dam due to insufficient quantity and quality of natural clay core at an economically feasible distance for Legemera micro earth dam irrigation project.

1.3 Research Question

The main questions of this study were:

- Does the use of concrete face rock fill dam significantly reduce seepage flow through the dam body as compared to existing dam?
- Does concrete face rock fill dam is more stable than dam with clay core under different loading condition?
- Does concrete face rock fill dam is a better alternative over clay core under given conditions?

1.4 Research Objectives

1.4.1 General objectives

The overall objective of this research is to determine an appropriate solution to limited clay material/soil for impervious core in Legemera micro irrigation dam project by introducing concrete face rock fill dam.

1.4.2 Specific objectives

- To develop safe design of different sections of concrete face rock fill dam by fixing upstream and downstream slope with available material in a safe way.
- To assess the advantages of concrete face rock fill dam with regards to stability and seepage control.
- To assess the economic advantage of CFRD over the clay core (existing design).

1.5 Significance of the Study

Currently, in Ethiopia embankment dams have been constructed and design works have been done by the Federal as well as Regional Water Works Design and Supervision Enterprises for the success of second national growth and transformation plan program. However, shortage of construction material becomes significant in the design of embankment dams. This may be due to the inaccessibility of suitable construction material in an economic distance of the project site. So, the designer should make in depth analysis and select the appropriate construction materials in the design of embankment dams.

Therefore, the result and findings of this research work will provide an alternative insight of concrete face rock fill dam the design and construction of embankment dam. Furthermore, this study will serve as a guideline for designers and professionals of this area in which construction material assessment, design, analysis and supervision of embankment works takes place.

1.6 Scope of the Research

The scope of this study is limited to the evaluation of stability analysis, seepage analysis, deformation analysis and dynamic analysis of the dam under different loading condition and finally, the result of the analysis obtained for concrete face rock fill dam is evaluated against result computed by Amhara national regional state water resources development bureau the former dam, central clay core dam. In addition to this economic analysis of concrete face rock fill dam were done and compare to existing dam (central clay core).

2. LITERATURE REVIEW

2.1 Embankment Dam

Embankment dam are water impounding structures composed of natural fragmental materials (such as soil and rock) and consist of discrete particles which maintain their individual identities and have spaces between them. These materials derive strength from their position, internal friction, and mutual attraction of their particles. Unlike cemented materials, these fragmental materials form a relatively flexible structure which can deform slightly to conform to the foundation deflection without causing failure. These dams are most suited in areas where the foundation material is weak or soft bedrock, abutments of deep soil deposits or weak rock, availability of a suitable location for a spillway, and availability of sufficient and suitable soils from required excavation or nearby borrow areas (USBR, 2012).

According to ICOLD (2011) an embankment dam which is constructed primarily of compacted earth in either homogeneous or zoned areas containing more than 50% of earth embankment dam categorize into Earth fill dam whereas embankment dams mainly contain more than 50% compacted or damped pervious rock fill under categories of rock fill dam.

Development of soil mechanics, study of behavior of earth dams, and the development of better construction techniques have been helpful in creating confidence to build higher dams with improved designs and more ingenious details. The result is that the highest dam in the world today is an earth dam. The highest earth/rock fill dams in the world are Roguni U.S.S.R (335m) Nurek, U.S.S.R. (300m) Mica, Tehri India (260m) and Oroville, U.S.A. (235m).Thousands of embankment dams exceeding 20 meters in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction (USSD, 2011).

2.2 Embankment Design Alternative for Rock fill Dam

2.2.1 Rock fill dam with clay core

The central core provides imperviousness to the embankment and reduces the seepage. The maximum width of the impervious core will be governed by stability and seepage criteria and also by the availability of the material. An earth dam with a sufficiently thick impervious core of strong material with pervious outer shells can have relatively steeper embankment slopes limited only by the foundation and embankment characteristics (USBR, 1998).

The central clay core will act as imperious material when the quantity of material will be optimized together with the overall behavior of the dam structure (Foster et al., 1998). Typically earth-core rock fill sections using central and sloping impervious earth cores. Sloping cores of impervious earth materials are sometimes advantageous from a construction of materials stand-point. They have been constructed both externally and internally. However, they present stability and rupture problems that are less severe in a centrally located core and also inability to inspect and repair them (Narita K, 2000). The biggest challenge for Legemera dam design option is availability of core materials within economically justifiable hauling distance that meets the design requirement (ANRSWRDB, 2007).

2.2.2 Geo-membrane faced rock fill dam

Geo-membranes are typically comprised of polymeric materials that are thin, flexible, and watertight. In general, the main types of geo-membranes used as seepage controller in dam construction include poly vinyl chloride (PVC) and high-density polyethylene (HDPE). According to ICOLD, 2011, many places around the world, geo-membranes are used as water barrier for temporary or permanent cofferdam and as a liner within main canals to reduce seepage losses. Some of the advantages of using such an impervious membrane: rock fill zone is unsaturated and downstream slope can be constructed with steeper slopes, plinth and grouting can take place independently, and membrane's flexibility to accommodate rock fills deformations. Whereas the following disadvantages play major role in their selection; vulnerable to impacts ice loads, sabotage, effects of weathering and aging, and this require partial or fully covered protective layer that increase cost, cannot provide storage during construction and Sometimes liable to thefts, if it is placed as along the upstream side, by the surrounding community (ICOLD, 2011). By the reasons mentioned above, the option to utilize geo-membrane as the main water barrier for the main Legemera micro dam construction is discarded.

2.2.3 Asphalt concrete core rock fill dam and asphalt concrete face rock fill dam

The AC Core

The impervious clay core change to asphalt concrete is called asphalt concrete or AC cores. AC cores, first developed in 1948, were sometimes constructed on a slope (upstream toward downstream, as the dam rises), but now are usually vertical in cross-section, and follow or parallel the axis of the dam, in plan. For high dams, the upper segment of the core may be slope to maintain positive stresses on the core during reservoir operation (USBR, 1990).

Some of the advantages of using such an impervious membrane less total area exposed to water, shorter grout curtain lengths, and protection from the effects of weathering and external damage. Whereas the weaknesses in their selection once constructed the core is difficult or inaccessibility to access for damage inspection, difficulty in repairing damage should it occur, damaged for any reason (e.g., abrupt differential settlement of the embankment; or earthquake), being in the interior of the dam the core is incapable of doubling as protection against wave action on the upstream face of the dam so far rip-rap or other type revetment will be required ,inability to place rock fill material without the simultaneous placement of impervious core material and filters, and the dependence on a smaller section of the dam for stability against sliding (Kaare ,1997).

The AC Facing

The asphalt concrete facing, either for a dam or reservoir, is a composite diaphragm constructed as the water barrier on the upstream face. It is second most common facing for rock fill dams next to concrete facing. Asphaltic concrete provides more flexibility and can thus tolerate larger settlements than reinforced concrete facings. But it is high cost as compared to concrete facing (ICOLD, 2011).

Some of the advantages of using such an impervious membrane can be used as slope protection, can be constructed after completion of the rock fill section, relatively easy to raise the dam at a later date, readily available for inspection, readily available for repair, provides more flexibility and can thus tolerate larger settlements. The fact that a revetment will be required on the upstream face has a significant cost impact as compared to concert face rock fill dam. Whereas drawbacks in their selection: they need for preparation and regularization of the sub-grade until the dam body completion is achieved, before cover placing does not allow the possibility of progressive water storage during construction, the asphalt concrete changes its properties due to the temperature change in the exposed zone, possible need cover for cracking repair operation especially at the dam upstream toe, complex manufacturing of the cover due to its multi-layered structure and critical connection to the U/S dam slope toe concrete structure (Bunn, R. A, 1992)

2.2.4 Concrete faced rock-fill dam

Concrete face rock fill dam had its origin in the mining region of the Sierra Nevada in California in the 1850s. Experience up to 1960 using dumped rock fill, demonstrated the CFRD to be a safe and economical type of dam, but subjected leakage caused by the high compressibility of the segregated dumped rock fill (ICOLD, 2004).

Data from the year 2006 report more than 200 CFRDs above 100 m between the complete and design phases due to economical reason. For example, Agbulu (234m, Philippines), Shuibuya (233m, China), and La Yesca (210m, Mexico) etc. Today, CFRD is an established major dam type to be included in initial project feasibility studies (Resende and Materon, 2000).

Concrete faced rock-fill dam is the type of dam that has a dam body of either rock-fills or gravel fill that is compacted in layers and an anti-seepage system of concrete face slab. The facing concrete slab acts as an impervious layer while the rock-fill body with a high permeability, gives support to the concrete face slab and the overall stability of the dam (ICOLD, 2011).

The main advantage is utilization of less volume of rock fill consequently considerable economy is saved, larger portion of the dam is available for stability against sliding, relatively easy to raise the dam at a later date, less costly as compared to asphalt concrete, working in very cold or rainy conditions, can be constructed after completion of the rock fill section, readily available for inspection and repair, in wet climates the absence of impervious soil fill simplifies and speeds construction, Some of the disadvantages are design for leakage through opened joints and cannot provide storage during construction (Vncold, 2008).

2.2.5 Cost comparisons for designs from different projects experience

The Rio Esti Hydropower Project (Rio Esti) in the Chiriquí province is the oldest project of the case studies and was finished already in 2003. Two alternatives were compared at the design stage: RCC and CFRD. The latter was chosen due to cost .So it is constructed as a CFRD (Resende and Materon, 2000)

Changheba dam project was completed in 2016 .Two alternatives were compared during design stage: AC and CFRD the latter was chosen due to cost. So Changheba dam was constructed as CFRD (Resende and Materon, 2000).

Concrete face rock fill dam is proved to be an economical alternative due to the fast rate of progress and smaller volume of rock fill and also the dam is available for stability against sliding (USACE, 1994). In such a way that fast rate of construction which enables the dam to be completed before fixed schedule. But, due to weather dependent construction of Legemera embankment dam, the construction activity of the dam is totally stopped during summer season, in such a way that the construction activity proceeds from September half to June half i.e. for nine months per year (ANRSWRDB, 2007). Relative merits of four embankment design alternatives were considered. CFRD, ACCRD, ACFRD and geo-membrane face rock fill dam were not practical and economic alternatives for the Legemera dam site. CFRD option was selected from the point of view of economy, ease of construction and safety considerations.

2.3 Concrete Faced Rock-fill Dam Design

2.3.1 Dam body design

The dam body design includes several properties of the CFRD are

- Inclination of the dam slopes
- Height and Crest width of the dam
- Settlements of the dam body

When designing the dam slopes, consideration to the gravel material has to be taken. If the material is of good enough quality, the up and downstream slopes can be designed at a slope of 1:1.3 to1:1.4. Whereas poorer quality rock-fill the slopes should not be steeper than 1:1.5 to 1:1.6. The crest width should be within the limits of four to eight meters wide. The height of the dam is decided on an economical as well as on a practical base by considering demand and supply analysis (Vncold, 2008). It is also important to make an analysis of the dam design to ensure that stresses and deformation is within the set limits. Several models should be made on different parts of the dam body.

These include, but are not limited to, the concrete face slab, perimeter joint of the dam body and rock fill creep etc. When doing the analyses it is important to include dynamic loading to see how the dam responds to seismic loads **(**Hunter, 2002).

2.3.2 Section of CFRD

According to ICOLD, (2004) concrete face rock fill dam section look like in Figure 2.1 and its properties as follow.

- Zone 1 used for concrete face protection, which is cohesion less silt or fine sand, is placed lower part of the face slab. This zone has be ranges from 2 to 7 meter length from heel in upstream direction depending on the height of the dam and material quality. Compaction of this zone is by hauling and spreading equipment.
- Zone 2A and 2B used as supporting concrete face. Zone 2A is a processed fine filter or fine sand with specific gradation limits to 20 mm with horizontal width ranges from 2 meter to 5 meter. Zone 2B is crushed rock with in average size of 75 mm and used to support concrete slab. The width of this zone ranges from 1 meter to 3 meter.
- Zone 3 is quarry run rock fill and categorize in to A, B and C depend on layer thickness, size and type of rock. Zone 3A is to provide compatibility to support Zone 2B with horizontal width is 1 meter. Zone 3B resists the water loading and limits face deflection. Zone 3C receives little water loading, and settlement is essentially during construction. Large rock is often placed at the downstream toe with in the height of (10-20%) height dam to resist scour and tail water wave action.
- The angel as shown in Figure 2.1 depend on height of dam, rock quality and rock fill grading. Its range from 45^0 to 135^0 in clockwise direction from horizontally blue dash line (ICOLD, 2011).
- Face slab it is main watertight component of the dam. Selection of face slab thickness is typically based on previous experience, height of the dam, and minimum conditions for reinforced concrete. However, current guidelines suggest a slabs thickness increasing with water head for CFRDs over about 100m (Kjarnsli, 1999). The thickness of face slab ranges from 0.3 + 0.002*H to 0.3 +0.004*H (Frossard, 2006b).

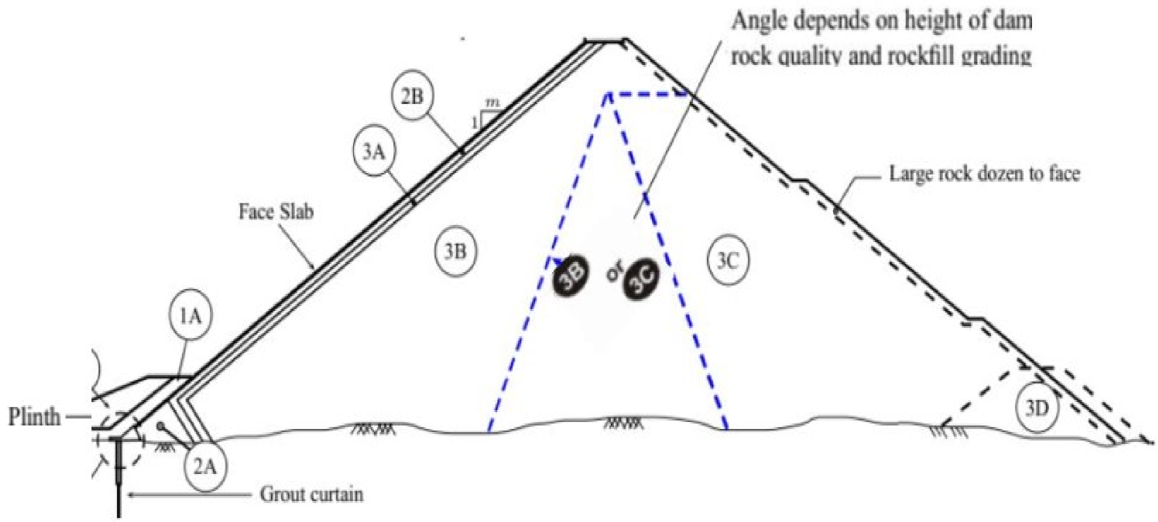

Figure 2.1 Typical Section of the concrete face rock fill dam

2.4 Slope Stability Analysis

Stability analysis is carried out in order to determine the factor of safety of a potential (shear) failure surface. The factor of safety is defined as the ratio of the resisting force or moment to the driving force or moment. The slope stability may also get reduced by increased shear stress due to changed loading conditions. The shear stress could be increased by increased stresses acting in directions being driving; external loads at the top of the slopes, water pressure in cracks at the top of the slope, increased soil weight due to increased water content etc. And factors making the resistant forces reduced; any kind of removal of soil at the bottom of the slope, and drop in water level at the slope base. (J. M. Duncan, 2005).

There are two essential ways to deal stability analyses; Limit equilibrium methods, and Finite element method (Griffiths, 1999). The primary distinction between these two investigations approaches is that the LEM depend on the static of harmony while FEM use the stress-strain relationship or constitutive law (Krishna, 2006).

2.4.1 Limit equilibrium method

Currently, most slope stability analyses involve LE analysis due to its simplicity and accuracy. These methods consist of cutting the slope into fine slices and applying appropriate equilibrium equations (equilibrium of the forces and moments). According to the assumptions made on the efforts between the slices and the equilibrium equations considered (Duncan, 2005). However, limitation of Limit equilibrium methods is not considering soil strain-stress.

The limit equilibrium methods use the Mohr-Coulomb expression to govern the shear strength (τ_f) along the sliding surface of embankment structures. Limit-equilibrium (LE) methods have been used to analyses slope stability for a long time (Duncan, 2005).

2.4.2 Finite element method

FEM is that a complicated model of a body or structure is divided into a number of smaller elements. Those elements are then connected by nodes. At every node there are one or more degrees of freedom where the quantity of functions is described. By solving the values at the nodes the stress and strains in every element can be calculated (Duncan, 2005). These methods have several advantages: to model slopes with a degree of very high realism (complex geometry, sequences of loading, presence of material for reinforcement, action of water, laws for complex soil behavior) and to better visualize the deformations of soils in place. However, it is critical to understand the analysis output due to the larger number of variables offered to the engineer (Puzrin, 2010)

The finite element method (FEM) can be used to compute displacements and stresses caused by applied loads. However, it does not provide a value for the overall factor of safety without additional processing of the computed stresses. The FEM can help identify local regions where overstress may occur and cause cracking in brittle and strain softening materials (Krishna, 2006)

2.5 Static Stability Analysis

Different types of load acting on the dam changes its slope and shear strengths with time result in changes in the factors of safety of slopes. Therefore, a stability analysis of the dam is necessary 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).

Deformation of an embankment dam starts to occur during the construction of the dam. These deformations are caused by the increase of effective stresses during the construction of the consecutive stages of earth material and also by the effects of creep of material, and embankment deformations under static loading occur as a result of volumetric changes, lateral spreading, or shear displacements within the embankment and foundation materials (Cooke, 1991).

Standard deformation and settlement according to ICOLD is assumed that for strong rock fill dam vertical settlements is less than 1% of the fill height and horizontal deformation should not be greater than 50% of the vertical deformation (Cooke, 1991). Although these assumptions were correct for many dams, a few dams did show much more settlements (Kjaernsli et al. 1992). The major loading conditions that embankment dams should be evaluated sudden drawdown end and during construction and steady state seepage condition (Costas et al., 2013).

2.5.1 End and during construction

The critical condition to be analyzed is at the completion of embankment dam construction but prior to filling with water. In this case there is no water table present in the reservoir and in the embankment dam (Costas et al., 2013).

2.5.2 Steady state seepage conditions

When the reservoir full of water steady state seepage pore pressures which are fully developed as a result of the reservoir have being storing water over a long period of time. In this case there is a phreatic surface line under steady seepage state (Costas et al., 2013).

2.5.3 Sudden drawdown condition

Fluctuations in reservoir water level may cause the upstream face stability to become critical mainly due to the removal of the supporting water. When the reservoir is rapidly evacuated and drawn down, pore water pressures in the dam body are reduced in two ways.

There is a slower dissipation of pore pressure due to drainage and there is an immediate elastic effect due to the removal of the total or partial water load (Ranjan, Rao, 2005).However, in case of concrete face rock fill dam sudden draw down effect is insignificant related to stability because of there is no water in dam body (Yuezhan et al., 2000).

2.6 Dynamic Stability Analysis

Earthquakes significantly affect the stability embankment slopes. Dams in seismic region need to be evaluated to make sure protecting public safety, life and property from earthquake hazard. The behavior of dams and their foundation under earthquake loading is an extremely complex problem which is not yet fully understood. Earth quakes impose additional loads on embankment dams. Typically, these loads are for short period, cyclic and involve motion in the horizontal and vertical directions (Brandes, 2004).

In dynamic analysis horizontal and vertical acceleration time histories are key input parameters. Therefore, site specific horizontal and vertical Acceleration Time History for Legemera micro earth dam should be produced using the peak accelerations and records of actual earthquakes. However, there is no Acceleration Time History records near the dam site, actual accelerograph recorded elsewhere have been used. Therefore, the following three Acceleration Time History data have been considered for the analysis (Messele Haile, 1996).

- The 1940 Elcentro Record, USA, magnitude (M) = 6.7, depth (H) = 11 km, radius (R) = 11.5 km.
- The 1995 Kobe JMA record, Japan, magnitude (M) = 7.2, depth (H) = 14.3 km, radius (R) = 19 km.
- The 1968 Hachinohe record, Japan, magnitude (M) =7.9, depth (H) =0 km, radius (R) =200 km.

Messle and Hadush, (2006) conducted liquefaction analysis on Tendaho dam project and during their study they suggested that the 1940 Elcentro record appears to have the closest resemblance with the earthquake records reported for the Tendaho dam site area. Hence, the 1940 Elcentro record have been used for Legemera micro earth dam Project site which is located 360 km far from earthquake region such as Kara Kore (Western margin of afar depression).However, the effect of earth quake is not that much on the project(ANRSWRDB, 2007)

According to Brandes, (2004) earthquake can affect embankments dams by causing settlement or cracking on the crest, loss of freeboard, instability of upstream and downstream slopes, and differential settlement and so on.

2.7 Software for Embankment Dam Analysis

2.7.1 Geo-Studio

Geo-Studio software is one of this numerical modeling software developed by Geo-Studio international based LE and FEM principles used specially for the analysis of seepage, stability and deformation of geotechnical structures (see section 2.4.1 and 2.4.2).

2.7.2 Plaxis 2d

Plaxis is a numerical program based on the finite element method and very useful for calculation of loads, deformations and slope stability in geotechnical engineering. It was developed in the end of the ´80s at Delft University in the Netherlands. During the ´90s it was further developed for commercial purposes and is since 1998 available in Windows environment. Plaxis is mainly a two-dimensional program for statically computing. However, deformations along the longitudinal direction of the dam assume to be zero, do not show failure surface and do not show seepage through the dam and foundation are limitation of Plaxis 2d software (Brinkgreve *et al.,* 2011). So Geo-studio software has been used in this thesis due to the reason listed in section 2.4.1 and 2.4.2.

3. MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location

Legemera small scale micro earth dam irrigation project is located in South Wello, Debre Sina Woreda, The project site has ground distance of about 4 km from Mekaneselam town which is the capital city of the Woreda. The project lies on Latitude 11^0 $10^0 21^0$ to $11^0 15^0 56^0$ N Longitude $38^0 37^0 34^0$ to $38^0 39^0 35^0$ E .Its elevation above mean sea level ranges from 2437 to 2843 m (ANRSWRDB, 2007). The location of the project area is shown in Figure 3.1.

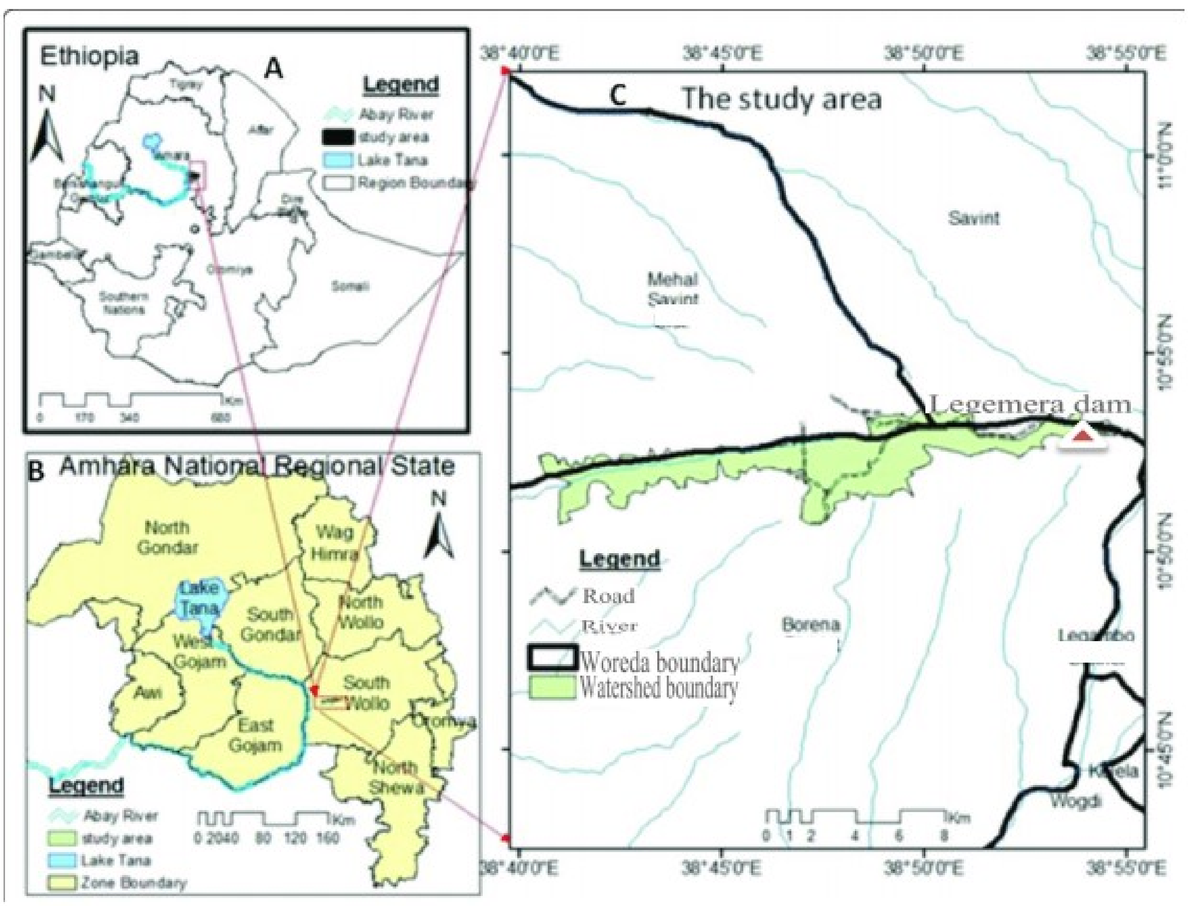

Figure 3.1 Location map of Legemera Micro Embankment dam

3.1.2 Climate

Climatic condition of the area is generally classified as semi-arid (Wayna Dega) zone with an altitude that ranging from 2186 m to 2924 m above sea level, and a mean annual temperature that varies from 13.76 ºC to 25.9 ºC. The Average annual rain fall is 863.65 mm. The mean monthly wind speed varies from 4.6 m/sec to 8 m/s. Sunshine hour ranges from 4.2 hour in July to 8 hour in November. Mean monthly humidity varies from 37% in June to 61% in August. Total annual evaporation loss is 1199.00 mm (ANRSWRDB, 2007).

3.1.3 Availability of construction material

According to ANRSWRDB, 2007 the availability of clay core material is located greater than fifteen kilometer far from the dam site. Whereas rock fill materials burrow area for the casing/shell zone is close to the quarry site. This burrow area is found at the central portion of the mountain or a ridge located about 0.5 to 0.8 km North West of the dam site.

3.1.4 Geology

The valley floor area/Streambed is characterized by gentle to plain grazing land that is mainly covered by soil which is floodplain, slightly weathered basalt rock and strong basalt rock. There is surface outcrop of the lower unit of the local geology at some portion of the streambed, mainly downstream of the dam axis (at about 100 to 200 m from the dam axis).

3.1.5 Topography

The topographic condition of Legemera micro earth dam project area is steep hill escarpments on both sides with an average elevation of 2437 m above sea level and average river bed is as wide as 25 m.

3.1.6 Seismology

The seismicity of Ethiopia and neighboring countries has been studied by Gouin (1997), Kebede (1991), Asfaw (1996) and others. Gouin (1997) compiled a catalogue of earthquake and produced the first seismic hazard map of Ethiopia shown in Figure 3.2. Contours indicate peak ground accelerations as a fraction of gravity. Several potentially damaging earthquakes have occurred. Examples are the 1906 Main Ethiopian rift earthquake (M is equal to 6.8), the 1961 Kara Kore (western margin of Afar depression) earthquake sequence (M is equal to 6.6), and the

1969 Central Afar earthquake sequence (M is equal to 6.3). The criteria adopted for the selection of design earthquake loads were based on the recommendations of USACE, 1999

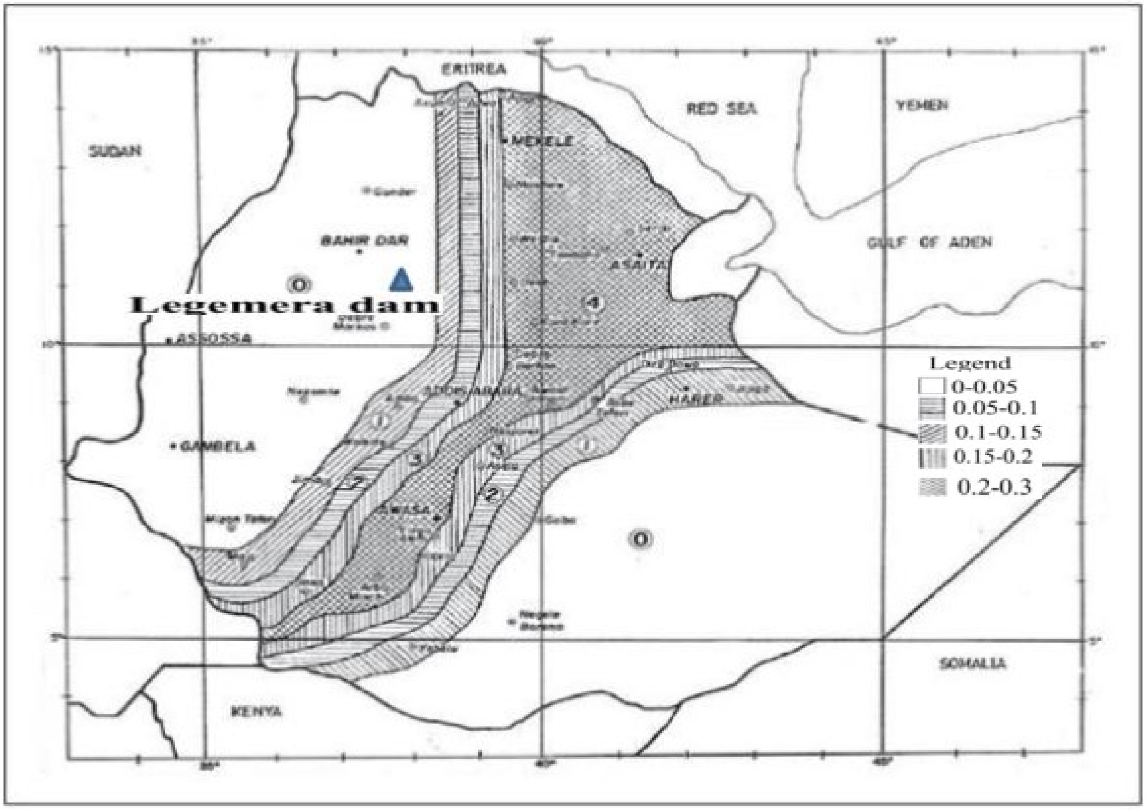

Figure 3.2 Seismic hazard maps of Ethiopia (MWUD, 2005)

Maximum Design Earthquake (MDE):-The MDE is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated.

Operating Basis Earthquake (OBE):- represents the level of ground motion at the dam site for which only minor damage is acceptable within the service life of the project. Linear elastic analysis has been used for computing seismic response of the structure. Return period (Approx. 145 years), Performance of the dam (No structural damage, i.e. stresses in concrete dams must remain within allowable stresses) and Dam safety (not relevant for dam safety it represents a serviceability limit state and is basically an economical criterion, which is of main interest for the dam owner) are the criteria of operating basis earth quake (ICOLD, 2011).

Table 3.1 Design Peak Ground Acceleration (ANRSWRDB, 2007)

Design Earth quake	PGA (g)	
	Horizontal acceleration (α_h) *	Vertical acceleration (α_v)**
Operating basis earthquake	0.03	0.015
Maximum design earthquake	0.05	0.025

*ANRSWRDB, 2007 and Figure 3.2 ** $\alpha_v = \frac{\alpha_h}{2}$ (USACE, 1999)

The project area is confirmed to be of relatively limited seismicity; mainly because of the location of the dam around 360 km far from earthquake region such as Kara Kore (Western margin of afar depression).The dam must be safe for the operating basis earthquake and must be check with the maximum design earthquake value for its adequacy.

3.2 Tools and Materials Used for Analysis

3.2.1 Tools

This study aimed to evaluate stability analysis of embankment dam by introducing concrete face rock fill dam in case of Legemera micro irrigation dam using Geo-studio 2012 model. In this research dam design document, geological and geotechnical laboratory report data etc. were used.

3.2.2 Primary data

Primarily data which are necessary for the purpose of this study were collected from the site. These data include physical observation of the site condition and other relevant information.

3.2.3 Secondary data

Essential input data for the Geo-studio 2012 such as; dam cross sectional design, geological report of the dam site, embankment material property (hydraulic conductivity, unit weight, angle of internal friction, cohesion of soil, conditions of soil water content) for core material, impervious material, foundation material, aggregate, filter and rip rap from design guide line documents of Amhara National Regional State Water Resources Development Bureau, Amhara Water Work Construction Enterprise and other sources were gathered.

Embankment material parameters used for seepage analysis

Hydraulic conductivity (K)

The ability of a soil to transport or conduct water under both saturated and unsaturated conditions is reflected by the hydraulic conductivity function. From the design report of Legemera micro earth dam hydraulic conductivities of embankment materials are listed in Table 3.2 (ANRSWRDB, 2007).

Volumetric water content (n)

The volumetric water content function describes the capability of the soil to store water under changes in matric pressure or what portion or volume of the voids remains water-filled as the soil drains. Volumetric water content values for different embankment materials are listed in Table 3.2 below.

Table 3.2 Hydraulic conductivity and volumetric water content used in the analysis

Type of material	Hydraulic conductivity	Volumetric water content	Remark
Bed rock	$1.52 * 10^{-7}$	0.3388	ANRSWRDB
Alluvial bed	$1.06*10^{-6}$	0.335	ANRSWRDB
Concrete face	$1.3*10^{-9}$	0	USACE,2008
Cutoff wall	$1.7*10^{-8}$	0	USACE,2008
1A	$1.52*10^{-7}$	0.33	ANRSWRDB
2A	$1.02*10^{-5}$	0.35	ANRSWRDB,SEEP/W
2B	$1.89*10^{-4}$	0.32	ASTM,2015
3A	$1.48*10^{-3}$	0.3	ASTM,2015
3B	$8.51*10^{-2}$	0.25	ANRSWRDB,SEEP/W
3C	$2.1*10^{-1}$	0.27	ANRSWRDB,SEEP/W
3D	$5.6*10^{3}$	0.25	ANRSWRDB,SEEP/W

Embankment material parameters used for slope stability analysis

For slope stability analysis shear strength parameters of the dam and embankment materials have been adopted from the final design report of Legemera earth dam (ANRSWRDB, 2007)

Table 3.3 Material parameter used for slope stability analysis

Material type	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (ϕ^0)	Remark
Bed rock	24	18	20	ANRSWRDB
Alluvial bed	18.5	22	35	ANRSWRDB
Concrete	24	45	36.5	USACE,2008
Cutoff wall	18	45	0	USACE,2008
1A	18	0	32	ANRSWRDB
2A	22	0	36	ANRSWRDB
2B	18	0	36	ASTM,2015
3A	21.5	0	33	ASTM,2015
3B	21.5	10	42	ANRSWRDB
3C	21.5	6	44	ANRSWRDB
3D	19	0	48	ANRSWRDB

Embankment material parameters used for static stress deformation analysis

For the analysis of static deformation strength and stiffness material properties are required and these data are adopted from the design report of the project and literatures as shown in Table 3.2

Table 3.4 Material parameter used for static stress deformation analysis

Type of material	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction angle (ϕ^0)	Effective elastic modulus (MPa)**	Poisson ratio **
Bed rock	24	18	20	3400	0.33
Alluvial bed	18.5	22	35	350	0.23
Concrete face	24	45	36.5	28000	0.17
Cutoff wall	18	45	0	200	0.4
1A	18	0	32	15	0.3
2A	22	0	36	90	0.3
2B	18	0	36	80	0.3
3A	21.5	0	33	70	0.3
3B	21.5	10	42	60	0.3
3C	21.5	6	44	40	0.3
3D	19	0	48	30	0.3

** USACE, 2008 and ANRSWRDB, 2007

Embankment material parameters used for dynamic analysis

Newmark's method is only applicable for materials which do not lose strength in post-earthquake conditions which means that the slope is still stable after shaking has ceased. It is also useful for situations where no changes in pore-water pressure will occur, i.e. when the slope materials are cohesion less (Clough, 1966). The procedure used for approximating the material stress-strain behavior is by successive load increments, within which the material behavior is assumed to be linear. The simplified input data for the linear elastic model are listed in Table 3.5 with respect to material and the maximum shear modulus (Gmax) is determined by equation 3.1.

$$G_{max} = \frac{E}{2*(1+\upsilon)} \qquad \text{……………………………………………………………………………3.1}$$

Where E= Modulus of elasticity, ν= Poisson's ratio

Table 3.5 Material parameter used for dynamic analysis

Type of material	Unit Weight (kN/m^3)*	Damping ratio **	Effective elastic modulus (MPa)**	Poisson's ratio **	Maximum Shear Modulus (MPa)***
Bed rock	24	0.1	3400	0.33	1278.195
Alluvial bed	18.5	0.1	350	0.23	142.2764
Concrete	24	0.1	28000	0.17	11965.81
Cutoff wall	18	0.1	200	0.4	71.42857
1A	18	0.1	15	0.3	5.769231
2A	22	0.1	90	0.3	34.61538
2B	18	0.1	80	0.3	30.76923
3A	21.5	0.1	70	0.3	26.92308
3B	21.5	0.1	60	0.3	23.07692
3C	21.5	0.1	40	0.3	15.38462
3D	19	0.1	30	0.3	11.53846

* ANRSWRDB, 2007   ** USCEA, 2008   *** Calculated value using equation 3.1

3.3 Methods Used

After creating a computer model of a concrete face rock fill dam with selected material properties and geometrical conditions, the pore water pressures developed within the body of the dam and in the foundation under steady state seepage were initially estimated with the help of the SEEP/W software. With determined material parameters and geometrical conditions initial static analysis were performed with SIGMA/W. The analyses were carried out with the aim of controlling the model and accuracy of results together by using the stresses obtained from static analysis in dynamic analysis. In the next step, dynamic analyses were accomplished on dam with the help of the QUAKE/W software. The methods and materials model used for each activity is discussed as follows:

3.3.1 Seepage analysis

The basic requirement for the design of embankment dams is to ensure safety against internal erosion, piping and development of excessive pore pressures. Steady state seepage analysis is carried out to estimate the amount of seepage through the embankments and foundation materials (Narita, 2000). The analysis is conducted using the state of the art Finite Element Method based computer program. There are two fundamental types of seepage analysis: steady state and transient. A steady-state Seepage analysis is an analysis type where water pressures and water flow rates do not change with time. A transient analysis, on the other hand, has pressure conditions that change with time (SEEP/W, 2008). Since this thesis is considered a primer for seepage analysis to determine the quantity of seepage passing through the embankment as well as the foundation. SEEP/W software use for calculates the amount of seepage passing through the embankment as well as well foundation using partial differential equations. According to Hasani, (2013) partial differential equation for two dimensional flow mathematical expresses as:

$$\frac{\partial}{\partial x}(kx. \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y}(ky. \frac{\partial H}{\partial y}) = 0 \ldots\ldots\ldots 3.2$$

Where, kx,ky =Coefficient of permeability in (x, y) and

H=Total head

3.3.2 Slope stability analysis

The slope stability investigation was carried out using the SLOPE/W computer program based on the limit equilibrium method. Analysis types in limit equilibrium method are Fellenius, Spencer, Bishop and Morgenstern-Price methods. Among them Morgenstern-Price method was used to obtain the factors of safety because it satisfies all three conditions of equilibrium: force equilibrium in horizontal and vertical direction and moment equilibrium condition (Cheng, 2008).Limit equilibrium methods use the Mohr-Coulomb expression to govern the shear strength (τ_f) along the sliding surface of embankment structures. The shear stress(τ) at which a soil fails in shear is expressed as the shear strength of the soil and the shear strength is usually expressed by the Mohr-Coulomb linear relationship, where shear strength (τ_f) mathematically expressed as:

$$\tau_f = C' + (\sigma - u)\tan\phi' \ldots\ldots\ldots 3.2$$

Then factor of safety (FOS) is expressed as the ratio of the shear strength (resisting force), and the shear stress (driving force) required for equilibrium

$$\text{FoS} = \frac{C' + (\sigma - u)\tan\phi'}{\tau} \quad \ldots\ldots 3.3$$

Where, normal stress and pore pressure are denoted as σ and u respectively, and the friction angle and strength parameters, Ø' and cohesion, c' (expressed in terms of effective stresses, i.e. for drained situations) are denoted respectively.

Both upstream and downstream face of the dam under different loading condition, the slope stability tries to satisfy minimum recommended factor of safety. Table 3.6 summarizes the loading conditions and corresponding minimum factor of safety (FOS min) requirements proposed by (USACE, 2003) and used worldwide.

Table 3.6 Recommended factor of safety (USACE, 2003)

Load Combinations	DL	Reservoir Water Level			Earthquake		Slip Circle Location		Case type	Required FoS
		Empty	NPL	MPL	OBE	MDE	DS	US		
After Construction at DS	*	*					*		Unusual	1.3
After Construction at US	*	*						*	Unusual	1.3
Operation at DS	*		*				*		Usual	1.5
Operation + MDE at DS	*		*			*	*		Extreme	1.0
Operation at US	*		*					*	Usual	1.5
Operation + MDE at US	*		*			*		*	Extreme	1.0
Operation at DS	*			*			*		Usual	1.5
Operation at US	*			*				*	Usual	1.5

* Load combinations were considered in the analysis

3.3.3 Stress deformation analysis

The deformations (settlements and displacements) were calculated with GeoStudio 2012 SIGMA/W software. SIGMA/W is a finite element software product that can be used to perform stress and deformation analyses of dam with and without pore water pressure arise from the reservoir. Insitu analysis type was used for calculating total stress and shear stress analysis whereas load-deformation used for analysis of horizontal and vertical displacement.

3.3.4 Dynamic analysis

QUAKE/W can be used for the dynamic analysis of earth structures subjected to earthquake shaking or to point dynamic forces from a blast or a sudden impact load. QUAKE/W determines the motion and excess pore-water pressures that arise due to shaking. Constitutive models include linear elastic model and equivalent linear model was used for analysis.

3.4 Comparison of Concrete Face Rock Fill Dam with Central Clay Core Earth Dam

After, completion evaluation and stability analysis of concrete face rock fill dam comparison with central clay core earth dam was conducted. For comparison purpose the result of seepage analysis, slope stability analysis, static analysis, dynamical analysis and economic analysis has been accounted as comparison parameter.

4. RESULT AND DISCUSSION

4.1 Dam Zoning and Geometry

It has been discussed earlier in this document that Legemera micro earth dam originally being designed an earth fill dam with central clay core. However, the biggest challenge is availability of core materials within economically justifiable hauling distance that meets the design requirement (ANRSWRDB, 2007). To solve this problem concrete face rock fill dam was proposed as an alternative dam design instead of embankment dam with central clay core and all required comparative analysis was performed.

The upstream and downstream slopes of the dam were taken as 1.5 H: 1 V and 1.7 H: 1 V respectively. The dimension of the dam are 74 m of bottom width at maximum cross section, 4 m top width, dam crest length of 319 m, and dam height of 22 m, maximum pool level 21.5 m and normal pool level of 19.5 m. According to (ICOLD, 1998, 2002, 2011; USACE, 2003, 2004; USBR, 1987) the dam comprises the following key zones (Figure 4.2):

- A constant low permeable concrete face thickness of 0.3 m width or thickness.
- Zones 1A: this zone has 3 meter length from heel of the dam in the upstream and in the inclined concrete face with 1 meter thickness.
- Zone 2A: Is a processed fine filter (sand and gravel) used to intercept seepage with 3 meter horizontal width.
- Zone 2B: This zone provides support to the face slab and consists of sand and gravel sized particles with horizontal width of 1 meter.
- Zone 3A: This zone is a transition between zone 2B and zone 3C. The horizontal width of this zone is 1 meter.
- Zone 3B: This zone consists of rock fill material with maximum particle size up to 1 m.
- Zone 3C: This zone consists of rock fill material with maximum particle size up to 2 m.
- Zone 3D: This zone provides drainage conditions within the embankment. The height of this zone is 4 m.
- Grout Curtain: It is performed under the toe slab and outside of the embankment body. Depend on site condition depth of grout curtain also restrict to eight meter.

The maximum dam cross-section of the original design (Earth dam with central clay core) designed by Amhara National Regional State Water Resource Development Bureau ANRSWRDB, (2007) is presented in Figure 4.1 and its alternative CFRD in Figure 4.2.

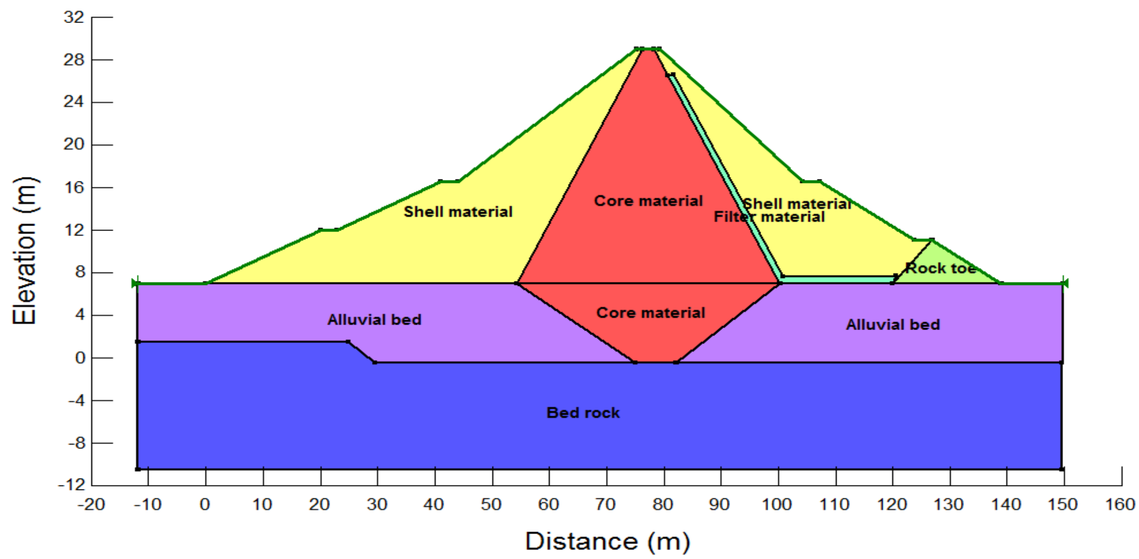

Figure 4.1 Dam cross-sections with central clay core designed by ANRSWRDB (2007)

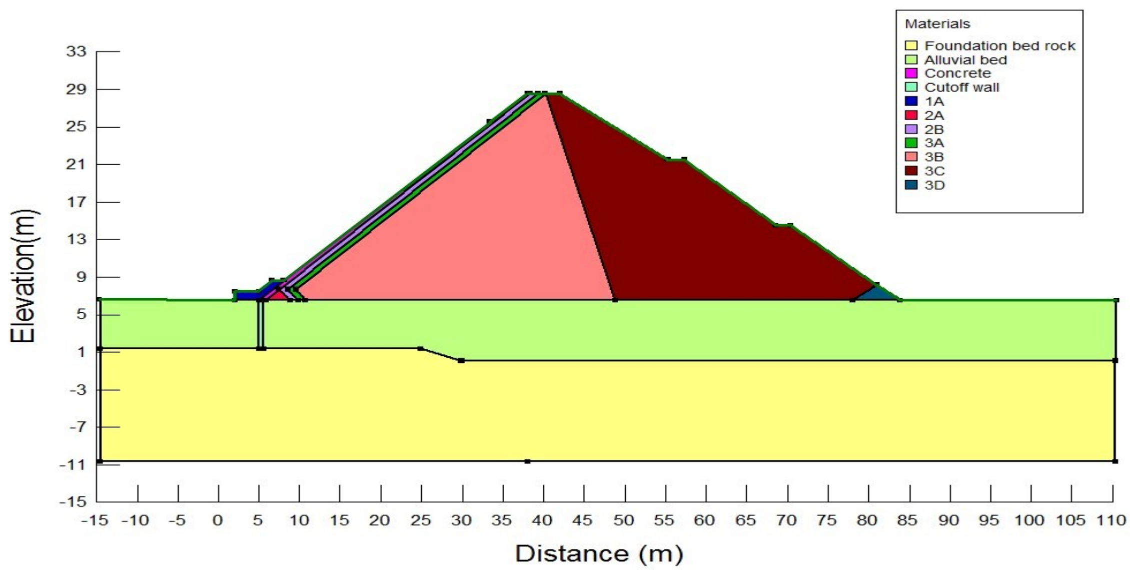

Figure 4.2 Alternative section of concrete face rock fill dam

4.2 Seepage Analyses

Leakage is a key parameter that is related to the overall performance of the CFRD. Large seepage rates are an indication that damage has occurred to the perimeter joint and/or that the concrete face has cracked to some extent. The seepage analyses were performed by means of the SEEP/W module of the GeoStudio finite element method geotechnical software package. The result of seepage analysis obtained for concrete face rock fill dam in this study confirms to what is found and stated by researcher Jansen, (1988).

Analysis of the expected quantity of seepage through the embankment and the dam foundation, which is the total flow per unit distance into the section, was computed using SEEP/W software model as $6.5906 x 10^{-16}$ m^3/s and $7.5864 x 10^{-9}$ m^3/s per unit distance, respectively. With the crest length of Legemera micro irrigation dam is 319 m, the total volume of water seeps through the dam body and the foundation was also computed to be 2.42×10^{-6} m^3/s. It is less than 0.03 m^3/s the standard set by Jansen (1988).

Jansen, (1988) that the maximum permissible seepage through the embankment and foundation with in incorporated proper filter material and drainage system is $0.03 m^3/s$. Therefore, the proposed dam cross section is satisfactory with respect to its ability of water retaining. The quantities of seepage flow calculated on the original design prepared by the Amhara national regional state water resource development bureau is 1.885227×10^{-5} m^3/s and $3.053 \times 10^{-7} m^3/s$ per unit width through the foundation and dam body respectively. So that the total volume of water seeps through the dam and foundation is expected to be $0.0062 m^3/s$ which is below allowable limit. Therefore, even though the amount of seepage loss were within the allowable limit, using CRDF will have an advantage over the central clay core in conserving stored water against seepage loss.

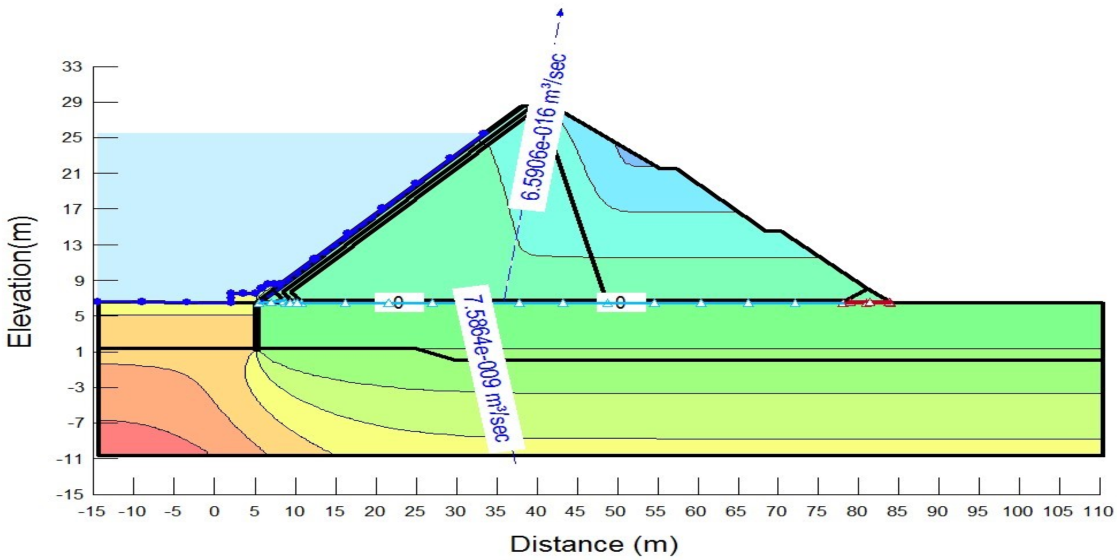

Figure 4.3 Seepage through the dam body and foundation

4.2.1 Location of phreatic line and pore water distribution

As can be seen from SEEP/W results in Figure 4.4, the blue thick line indicates that the line of zero pressure which rapidly drops from upstream concrete face to heel of the dam then horizontally up to the toe of the dam. It indicates that the use of concrete face as a water barrier material is quite effective and alluvial foundation is assumed to be slightly free drain and energy is dissipated as compared to concrete face. The distribution of pore water pressure through the dam body and the foundation at steady state condition is also shown in Figure 4.4. The negative sign indicates that the portion of the dam above blue thick line (above phreatic line) is a negative/suction pressure zone which means dry area.

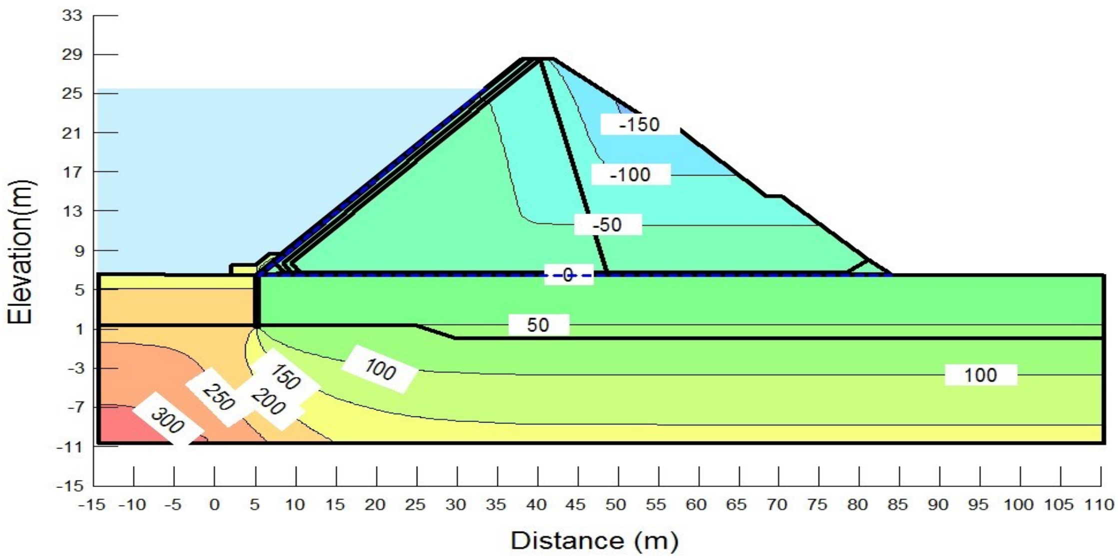

Figure 4.4 Pore water pressure difference through the dam body and foundation

4.3 Static Slope Stability Analysis

4.3.1 During and end of construction condition

During construction

The stability of concrete face rock fill dam has been analyzed during construction stages. For seek of analysis let assume the dam is construct in two stages and checking upstream and downstream stability at intermediate stage. The computed factor of safety during construction stage has been found to be 1.873 and 1.608 (refer Appendix A1 and A2) for upstream and downstream slope respectively. The result obtained at intermediate stage indicates that, use of concrete face rock fill dam can be satisfying the minimum factor of safety (1.3) requirement recommended by USACE, (2003).Since; the embankment is safe with respect to safety factor during constructions.

End of construction condition

In this condition both upstream and downstream slopes are critical slopes. The computed static factor of safety for upstream and downstream slope has been found to be 1.824 and 1.590 respectively which is shown in Figures 4.5 and Figure 4.7. The minimum factor of safety against static slope stability analysis is 1.3 at end of construction condition (USACE, 2003). Hence, the embankment is safe at end of construction. The half circles displayed by red color in Figure 4.6 and 4.8 indicate that, the region in which most possible slide surface during construction.

The computed static factors of safety of central clay core earth dam (original design) for upstream and downstream slope has been found to be 1.652 and 1.527, respectively (Appendix A3 and Appendix A4). Therefore, even though safety factor for upstream and downstream slope were within the allowable limit, using CRDF will have an advantage over the central clay core due to higher safety factor.

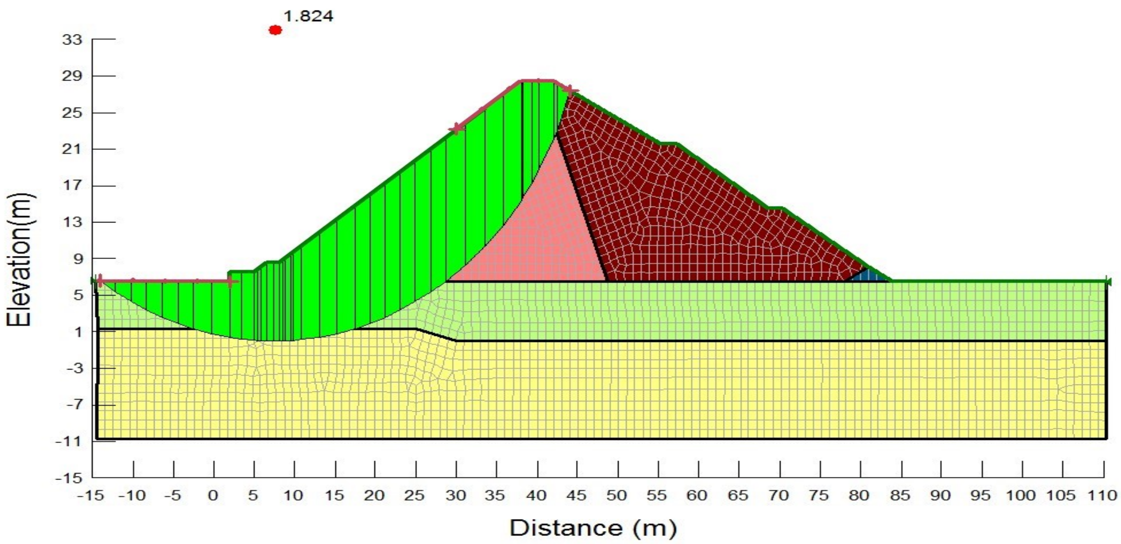

Figure 4.5 Factor of safety for upstream slope at end of construction at critical slip surface

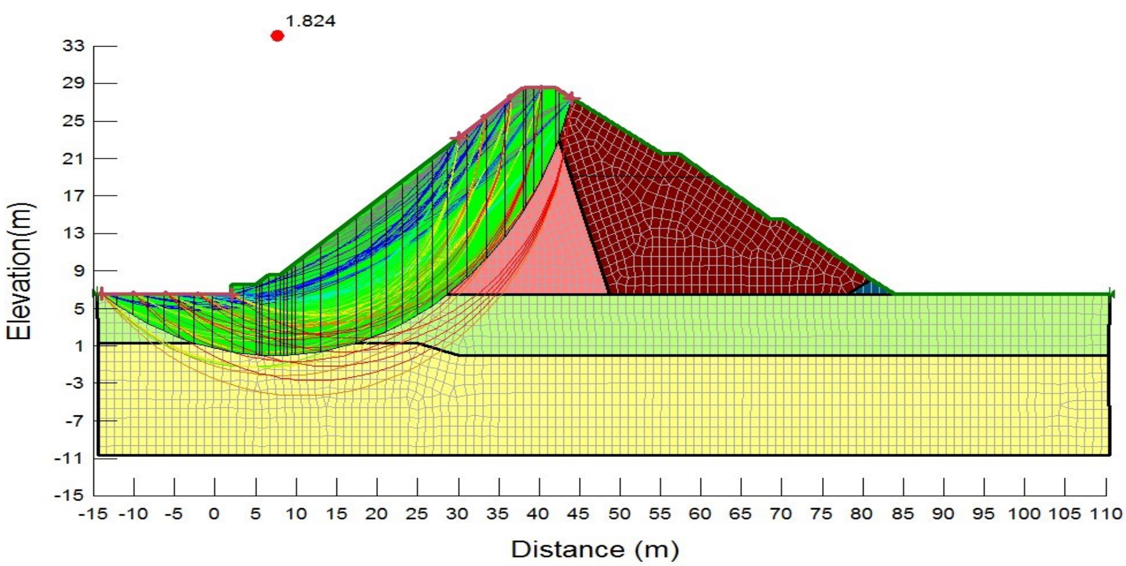

Figure 4.6 Factor of safety for upstream slope at end of construction for all slip surfaces

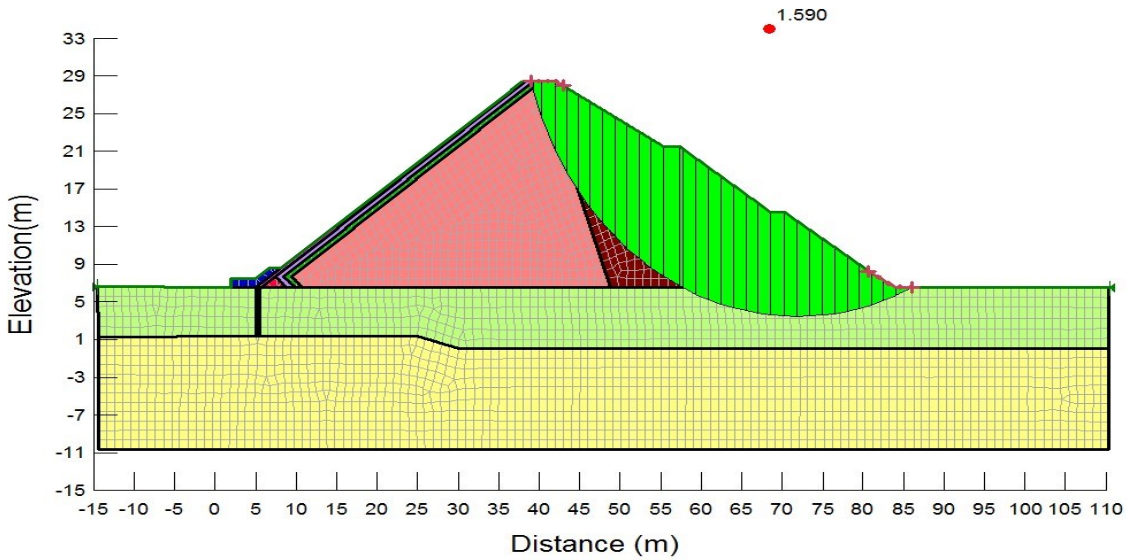

Figure 4.7 FoS for downstream slope at end of construction at critical slip surface

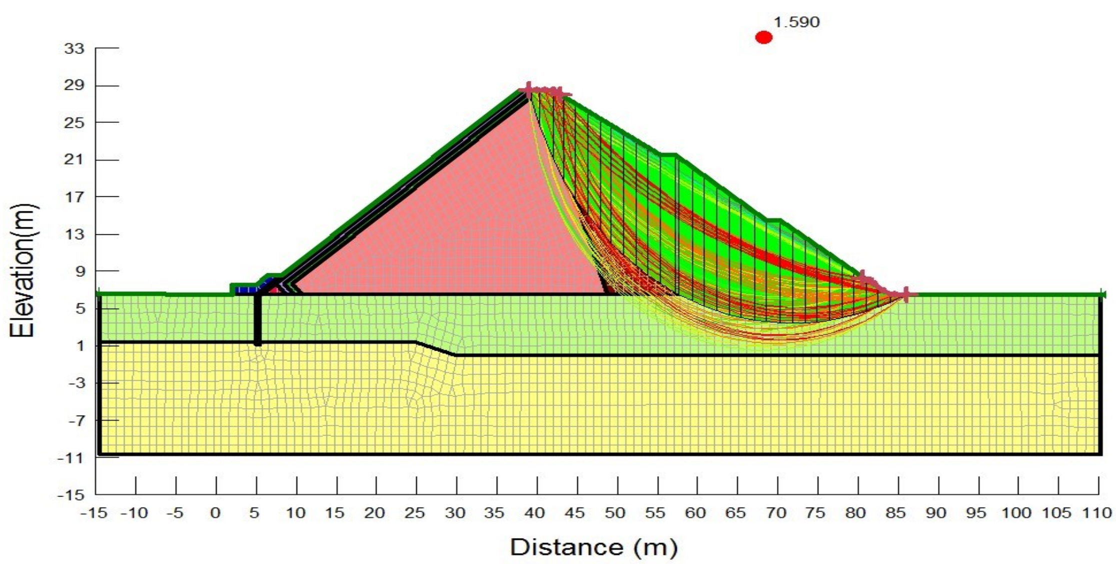

Figure 4.8 Factor of safety for downstream slope at end of construction for all slip surfaces

4.3.2 During steady state condition

The analysis through SLOPE/W was based on Morgenstern-Price half-sine function method that deals with both moment and force equilibrium equations. During stability of downstream slope of embankment dam, the most critical condition occur when the reservoir is full and steady state conditions. Based on the computation, the minimum factor of safety under steady state condition has been found to be 1.954 and 1.544 (Figure 4.9 and Figure 4.10) at the downstream and upstream slope respectively.

Factor of safety in case of normal pool level is greater as compared to empty reservoir because of vertical component of water load is used as stabilizing force against sliding in case of concrete face rock fill dam.

The computed static factors of safety of central clay core earth dam (original design) for upstream and downstream slope has been found to be 1.641 and 1.532 (Appendix A5 and Appendix A6). The result indicates that, concrete face rock fill dam and central clay core dam (original design) has reasonably satisfied the minimum factor of safety (1.5) requirement recommended by USACE, (2003). The reason why in case of central clay core factor of safety decrease after impounding while increase in case of concrete face rock fill dam is that, in case of central clay core the average dam body is wet due to seepage this tends to reduce shear strength of the material while water load act as stabilizing force against sliding in case of concrete face rock fill dam. Therefore, even though upstream and downstream factor of safety were within the allowable limit, using CRDF will have an advantage over the central clay core in case of stability.

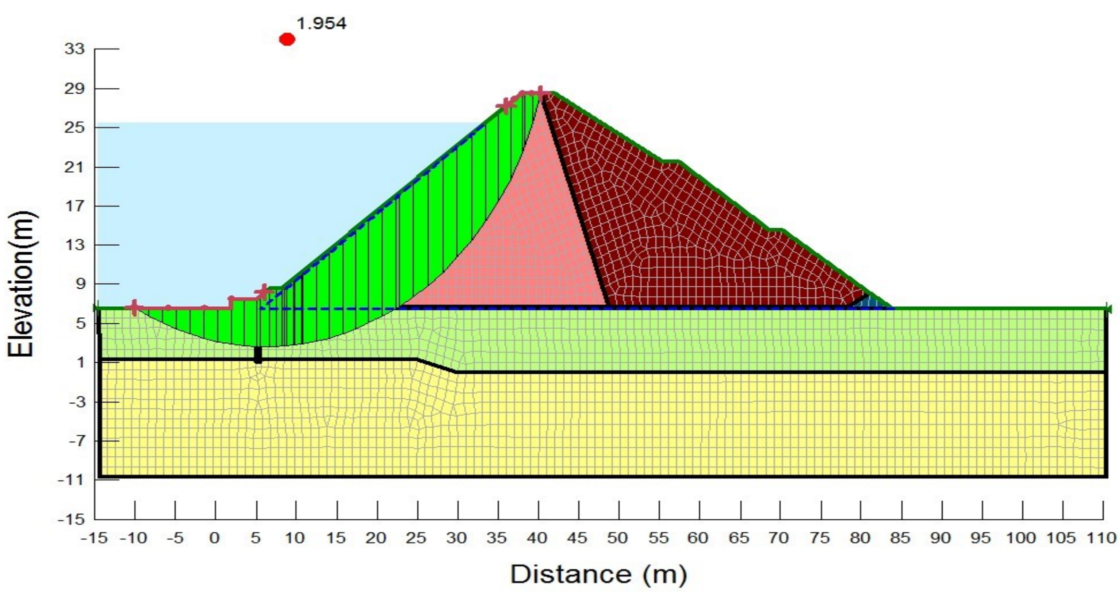

Figure 4.9 Factor of safety for upstream slope during steady state seepage

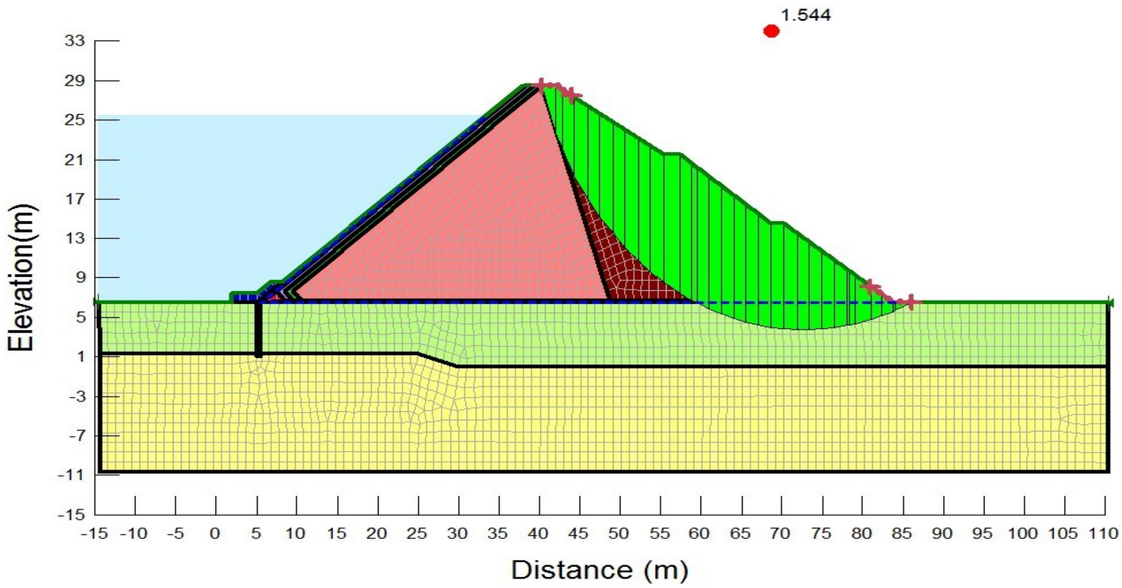

Figure 4.10 Factor of safety for downstream slope during steady state seepage

4.3.3 Maximum water level condition

Based on the computation, the minimum factor of safety under maximum water level condition has been found to be 1.969 and 1.529 (Figure 4.11 and Figure 4.12) at the upstream and downstream slope respectively. The result indicates that, concrete face rock fill dam has reasonably satisfied the minimum factor of safety (1.5) requirement recommended by USACE, (2003). The reason behind upstream factor of safety improved as the water level increased is that when the reservoir level increases vertical component of water load also increases. So the vertical component of water load acting on the upstream face of the dam is serves as a resistance force against sliding. The region in which most possible slide surface during maximum water level condition is displayed by red color and its magnitude has been found to be 1.969 and 1.529 (Appendix A7 and Appendix A8) for upstream and downstream slope respectively.

The computed static factors of safety of central clay core earth dam (original design) for upstream and downstream slope has been found to be 1.469 and 1.498 (Appendix A9 and Appendix A10) respectively. The result obtained shows that, during maximum water level condition due to upstream water pressure and its distribution in embankment material the safety factor for clay core dam is poorly attain the minimum safety factor requirement suggested by USACE, (2003).

But, under the same loading condition the factor of safety computed for concrete face rock fill dam gives attractive result and it reasonably attain the minimum safety factor requirement recommended by USACE, (2003). The reason why factor of safety below allowable limit for clay core dam is that, the development of pore pressure occurs in the body of the dam this tends to reduce shear strength of the material.

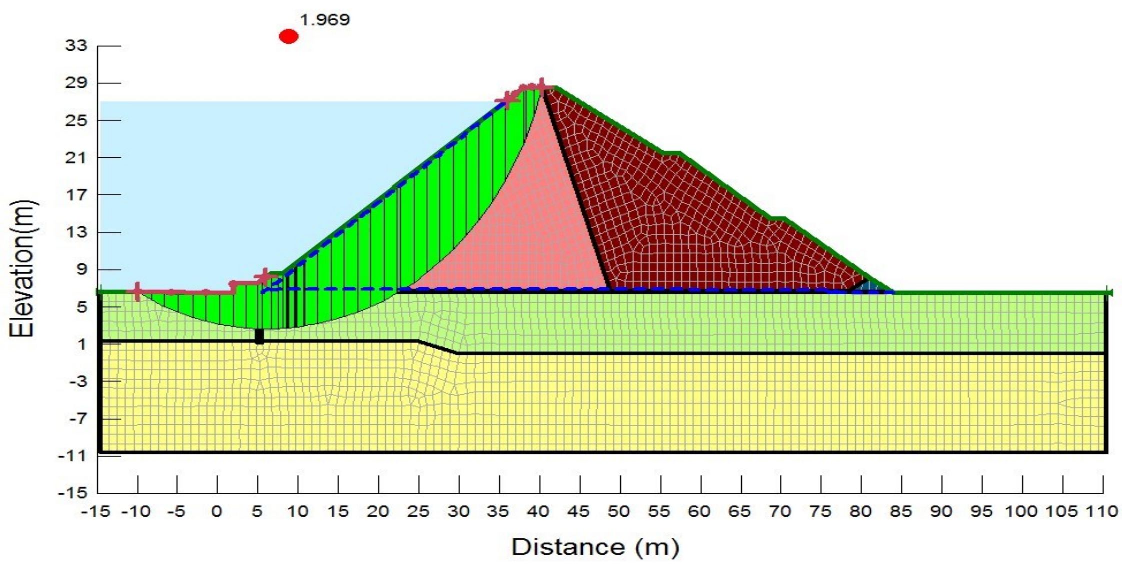

Figure 4.11 Factor of safety for upstream slope during maximum water level condition

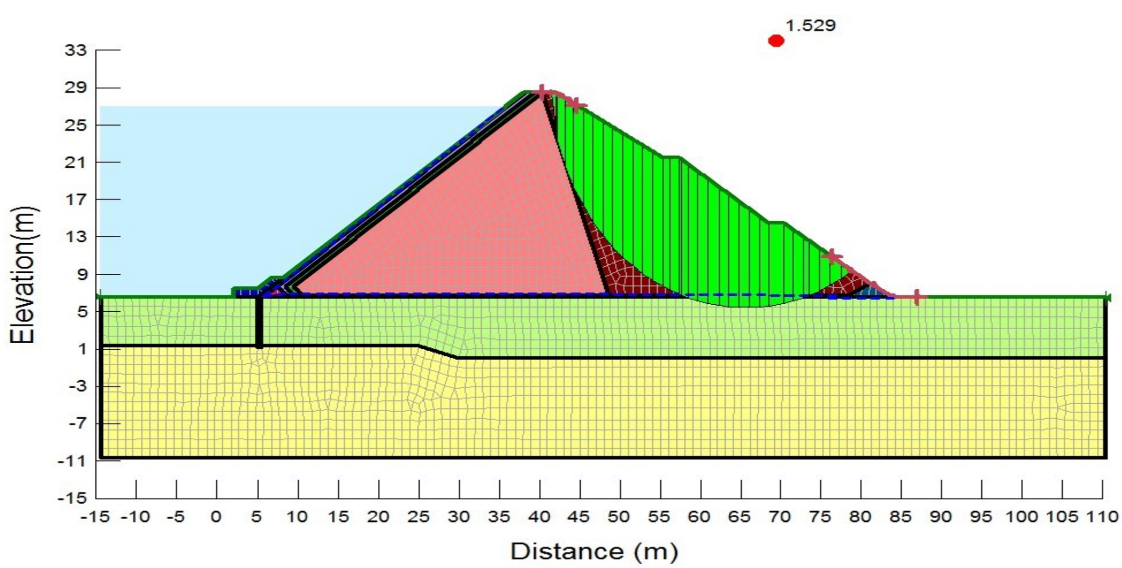

Figure 4.12 Factor of safety for downstream slope during maximum water level condition

Generally, from stability point of view factors of safety against slope stability under different loading conditions has been computed. From the result reported that, at end construction upstream and downstream slope stability of concrete face rock fill dam has been found to be 1.824 and 1.59 respectively. In case of steady state condition safety factor for upstream and downstream has been found to be 1.954 and 1.544 respectively. Similarly, during maximum water level condition also 1.969 and 1.529 for upstream and downstream slope respectively (Table 4.1).

Table 4.1 Summary of computed FoS obtained for CFRD and its comparison with standard

Load Combinations	Recommended FoS (USACE, 2003)	Computed FoS		Remark
		U/S	D/S	
End Construction at DS	1.3	1.824	1.590	Ok
Steady state condition at NPL without earthquake	1.5	1.954	1.544	Ok
Maximum water level conditions	1.5	1.969	1.529	Ok
Sudden drawdown*	1.3	---	---	

*The phreatic line is located at the base of the dam. This indicates there is no seepage water entering into the dam body due to impervious membrane which lies on the upstream slope. Therefore, draw down effect is limited in such case. As a result, stability analysis during sudden draw down is not conducted in this study

As can be seen clearly in Table 4.1 the factors of safety calculated in this research coincides with the recommended allowable range. Therefore, the new proposed dam cross section satisfies minimum requirement under static stability conditions such as; steady state condition, maximum water level condition and end of construction condition for upstream and downstream slope stability.

4.4 Static Deformation Analysis

Static analysis of an embankment dam is carried out by considering mainly self-weight and hydrostatic force. Boundary condition and finite element mesh of the domain used for stress deformation analysis has been shown in Figure 4.13.The size of mesh and number of element is 1 meter and 3130 element respectively with 3252 node. The static stress deformation analysis mainly vertical deformation, horizontal deformation and stress state for different loading conditions has been computed. Embankment deformations under static loading occur as a result of volumetric changes, lateral spreading, or shear displacements within the embankment and foundation materials. Volumetric changes are due to either an increase in the normal stresses on a soil element causing a decrease in void volume undergoing shear. Lateral spreading and shear displacements are due to squeezing, distorting, and localized shear failures of material elements as the materials adjust to the stress conditions imposed by constructing the embankment and operating the reservoir.

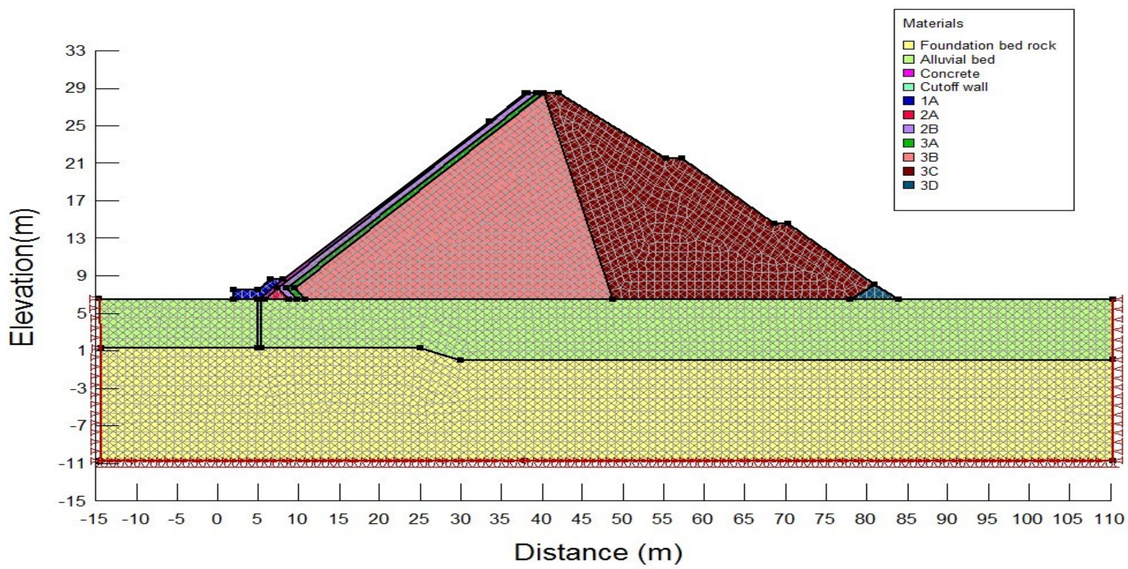

Figure 4.13 Boundary condition and finite element discretization of the domain

4.4.1 During construction / End of condition

Vertical displacement of concrete face rock fill dam

The deformation and stress condition of the dam has been computed at the end of construction. Figures 4.14 shows that, the maximum vertical displacement of concrete face rock fill dam is negative 0.05 m and occurs near the crest of the dam which is shaded by the deep blue color.

The negative sign on vertical displacement shows that the direction of the deformation tends to occur in the downward direction. This value is found within allowable range 1% of dam height (i.e. 0.01*22m=0.22m) recommended by Cooke, (1991). Therefore, with regard to vertical displacement, the analyzed result shows that concrete face rock fill dam gives good result.

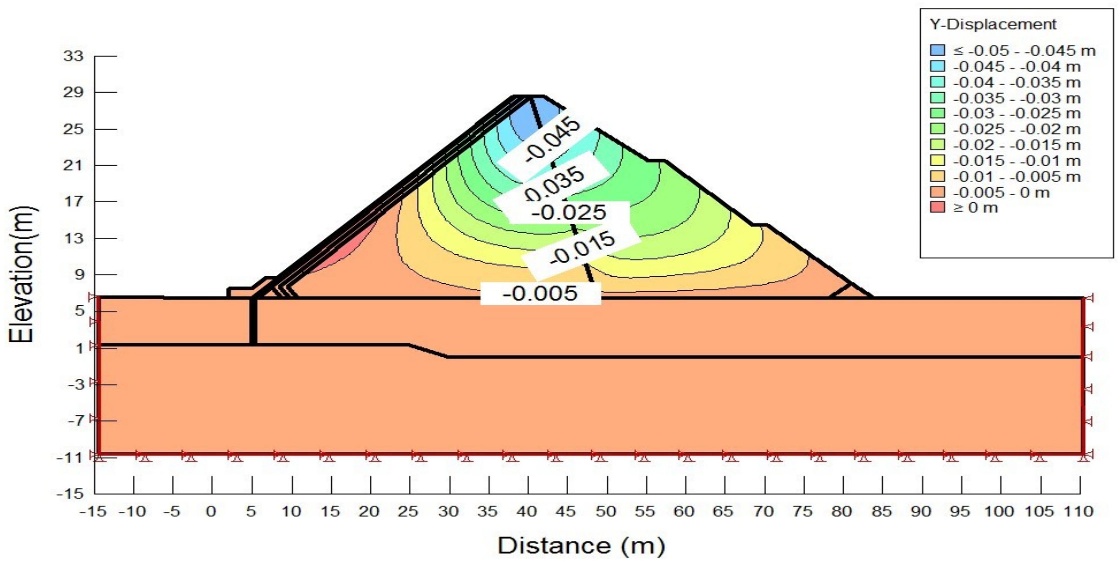

Figure 4.14 Contours of vertical displacement for alternative design at end of construction

Horizontal displacement of concrete face rock fill dam

The simulated models result of maximum horizontal displacement of concrete face rock fill dam is 0.025 (Figure 4.15) which is below the allowable values (less than 50% of vertical settlement) recommended by Cooke, (1991). Therefore, the dam is not bring any series problem on stability. The negative sign on horizontal displacement shows that the deformation tends in the upstream direction and positive value shows the deformation of the embankment tends to in the downstream direction.

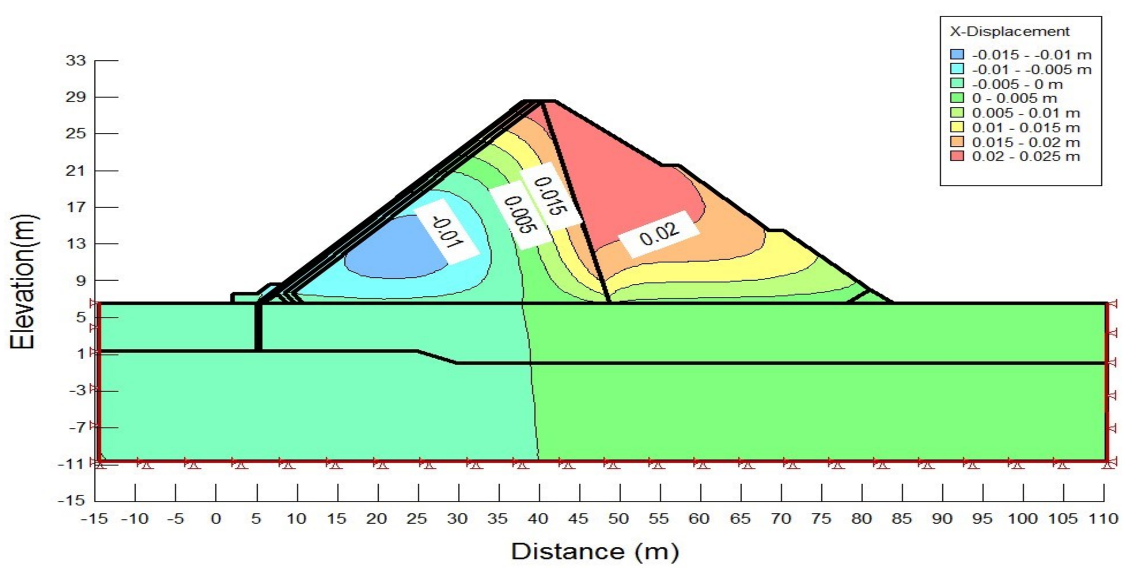

Figure 4.15 Contours of horizontal displacement for alternative design at end of construction

Total stress and shear stress distribution

The stress distribution for end of construction loading condition inside the dam and foundation was analyzed. The result shows that, the region of maximum vertical displacement is 750kPa which is shaded by the deep red color, and occur at the base of the foundation due to gravity loading. The variation of total stress is increased uniformly from the crest of the dam toward the foundation of the dam. The deep red colored region at the bottom of the dam is used to represent the region of maximum total stress. On the hand the region shaded by the dark blue color at the top most portions is used to represent region of minimum stress. As numerically expression maximum and minimum total stress is 750kPa and 50kPa respectively (Figure 4.16). Therefore, maximum total stress much lower than the expected bearing capacity of the foundation rock (i.e. uniaxial compressive strength of 35-43MPa) studied by ANRSWRDB (2007).This shows that the dam will not bring any series problem on stability.

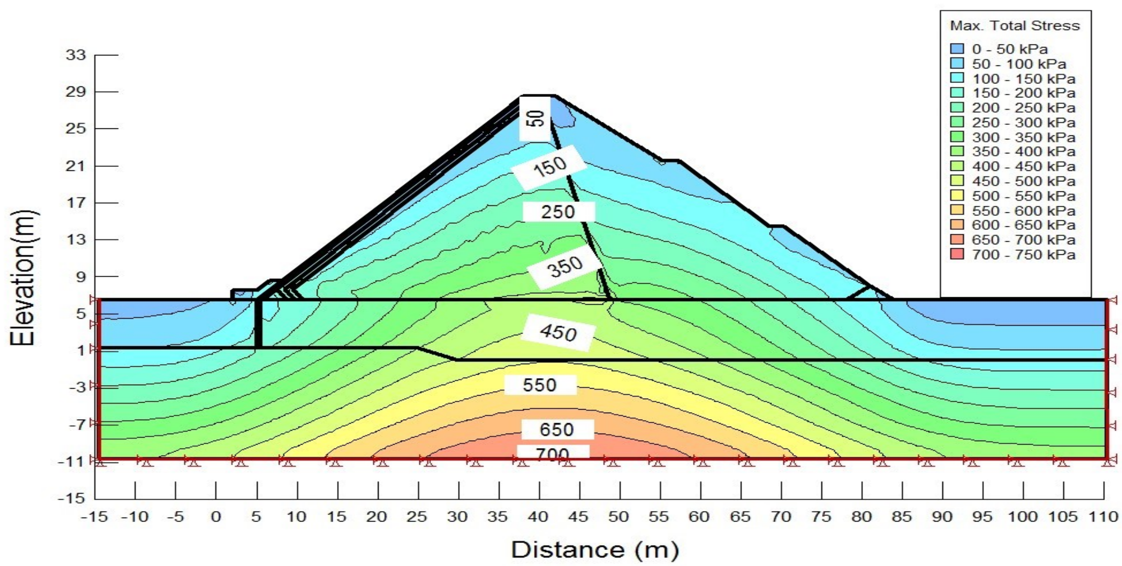

Figure 4.16 Computed maximum total stresses in dam body and foundation (Static analysis)

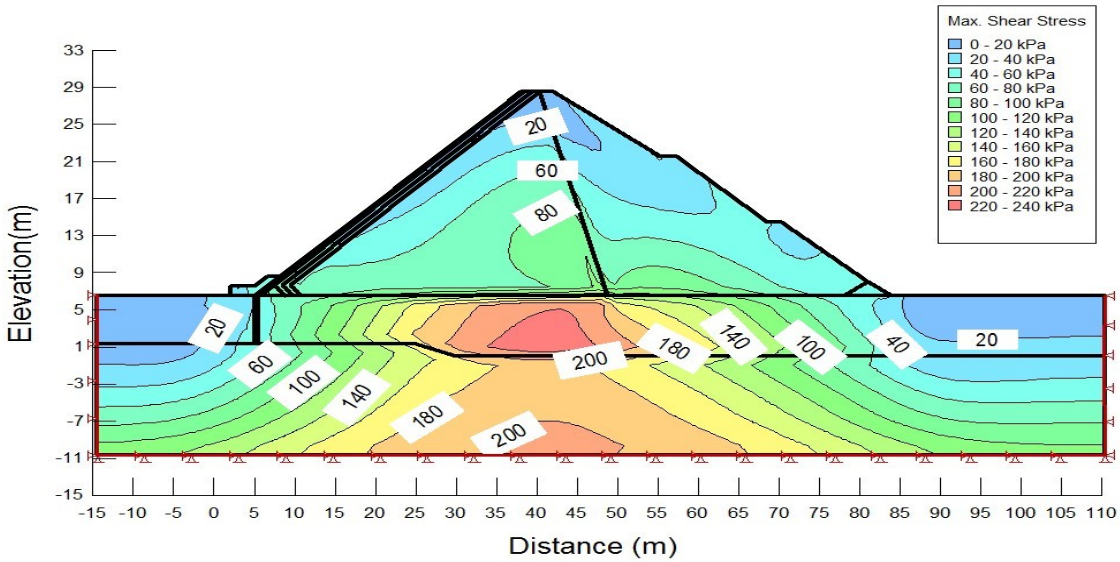

Figure 4.17 computed maximum shear stress in dam body and foundation (Static analysis)

4.4.2 Normal pool level condition

Vertical displacement of concrete face rock fill dam

When the reservoir impounds water to its normal pool level the settlement of concrete face rock fill dam increases by 0.005 m. As indicated in the Figures 4.18 the maximum vertical displacement found for concrete face rock fill dam was -0.055 m. The result shows that, the region of maximum vertical displacement is shaded by the deep blue color which occurs at the crest of the dam. The position at which the maximum vertical displacement obtained in this study is confirmed to the result found by Mustafa, (2014.). From the result maximum displacement of concrete rock fill dam is below the allowable values (less than 1% of dam height) recommended by Cooke, (1991). So, the dam is safe under steady state condition with respect to horizontal deformation. The reason why maximum vertical settlement happens nearly at the crest of the dam is that, maximum total load of the dam act here.

The maximum vertical settlement calculated on the existing dam design by Amhara National Regional State Water Resource Development Bureau is 0.1m under steady state condition (ANRSERDB, 2007). Therefore, even though maximum vertical deformation clay core dam were within the allowable limit, using CRDF will have an advantage over the central clay core in in terms of deformation resistance.

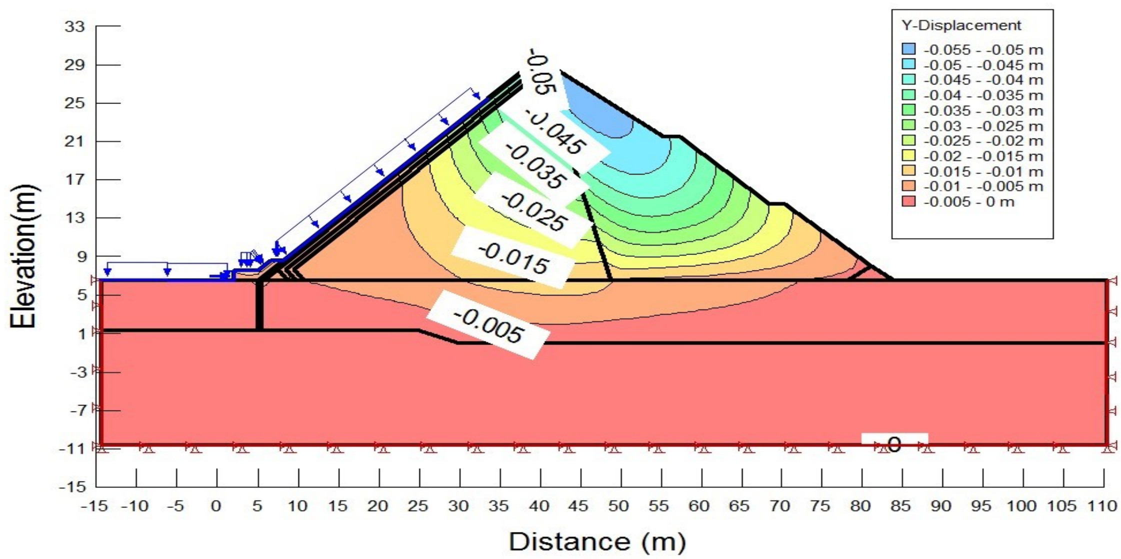

Figure 4.18 Contours of vertical displacement at normal pool level condition

Horizontal displacement of concrete face rock fill dam

Maximum horizontal displacement of concrete face rock fill dam is 0.032 m which is shown in Figure 4.19. So, the extreme horizontal displacement at normal pool level bounded by red color which was registered nearly the downstream shoulder of dam. The horizontal displacement pattern of concrete face rock fill obtained in this study gives nearly similar result of horizontal displacement pattern and position obtained by Mustafa, (2014) which increased to the downstream direction.

The lateral deformation of the dam increases to 0.032 m after impoundment. This increment results due to the load imposed by the reservoir. However, the displacement which is not satisfy the allowable values (less than 50% of vertical settlement) recommended by Cooke, (1991). But also the displacement which is satisfy the allowable values (0.1-0.5% height of dam) suggested by ICOLD, (1993). So, the dam is safe under steady state condition with respect to horizontal deformation.

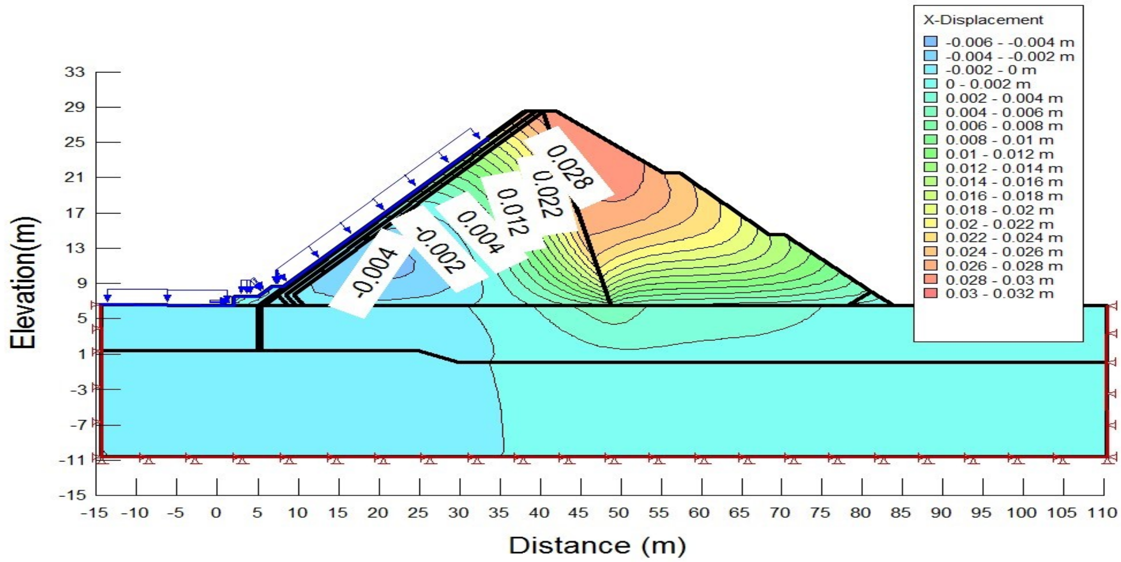

Figure 4.19 Contours of horizontal displacement at normal pool level condition

Total stress distribution

The total stress after impoundment is increased to 800kPa due to the effect of external water load. Total stresses are controlled by the self-weight of the dam, external loads and pore-water pressure. In this study the whole part of the dam is dry, so the effect of pore pressure is neglected. In case of concrete face rock fill dam vertical component of water load serves as stability against sliding .However, impoundment of reservoir has some negative effect on the dam due to additional load exerted on the structure. Rock mass quality of foundation is tested by ANRSWRDB and its average RQD value ranges from 50-75 % (i.e. fair to good quality) with uniaxial compressive strength (UCS) of 35-43MPa (ANRSWRDB,2007).The computed maximum total stress is 800kPa on the base or rock foundation (Figure 4.20). This stress level is much lower than the expected bearing capacity of the foundation rock so that the dam is safe under steady state condition.

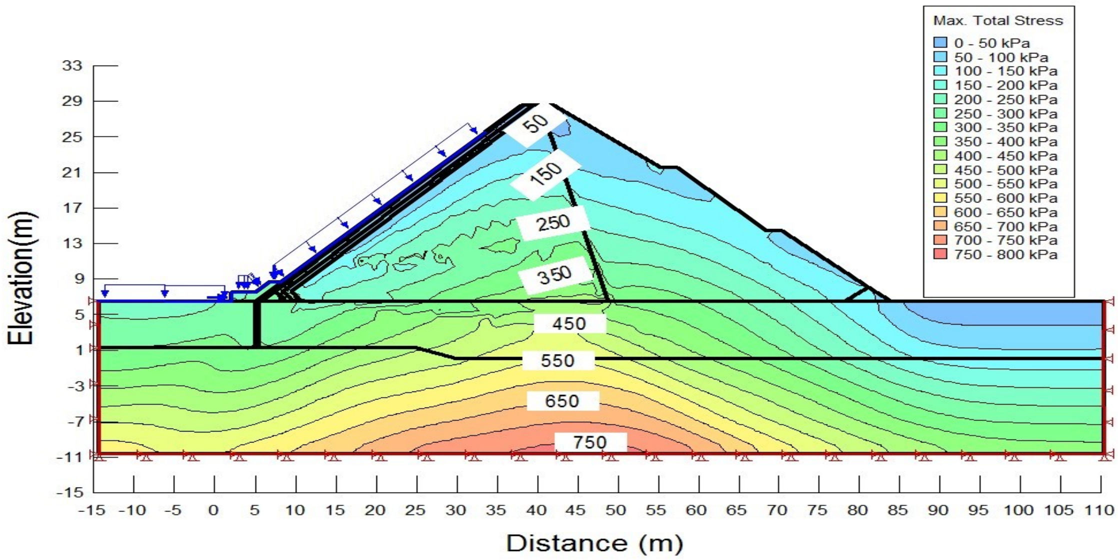

Figure 4.20 Computed Maximum total stresses in dam body and foundation (Static analysis)

The distribution of minor total stress inside the dam body and foundation has to be ranges from 50kPa to 450kPa at normal full reservoir level condition (Appendix A11). From the result positive value indicate that, the dam body as well as foundation is not subjected to cracking.

4.4.3 Maximum Pool Level Condition

Vertical displacement of concrete face rock fill dam

When the reservoir water reaches to its maximum level, vertical displacement also increases as shown in Figure 4.21. Maximum vertical displacement of the dam during maximum pool level is appears near the crest of the dam shaded by the dark blue color. The maximum vertical settlement for this loading condition is computed to be -0.09 m. However, it is acceptable as the deformation is less than 1% of the total dam height suggested by Cooke, (1991). So, the dam is safe under maximum pool level condition with respect to vertical deformation.

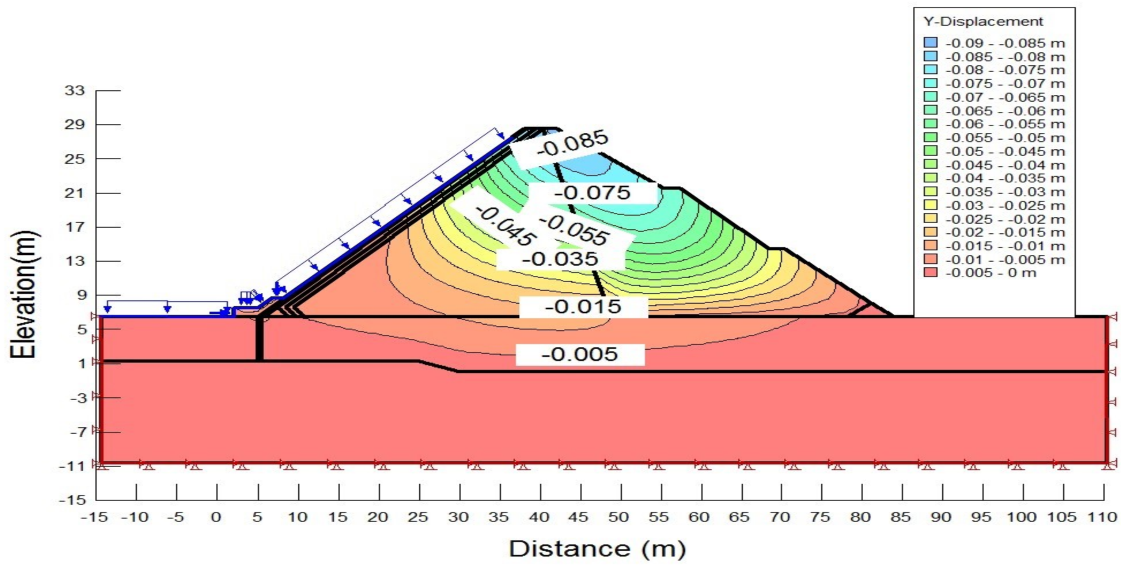

Figure 4.21 Contours of vertical displacement at maximum pool level condition

Horizontal displacement of concrete face rock fill dam

The increment of water level from normal pool level to maximum water level further deforms the embankment to lateral direction by 0.035 in Appendix A 12 due to load exerted on the dam body by water pressure. The negative sign on horizontal displacement shows that the direction of the deformation tends to occur towards the upstream and positive value shows the deformation of the embankment to be at downstream side. However, the displacement which is below the allowable values (less than 50% of vertical settlement) recommended by Cooke, (1991). So, the dam is safe under maximum pool level condition with respect to horizontal deformation.

Generally, from deformation point of view vertical as well as horizontal deformation has been computed under different loading conditions. From the result presented in Table 4.2 that, at end construction vertical deformation and horizontal deformations has been found to be -0.05 and 0.025 respectively. In case of steady state vertical and horizontal deformations has been found to be -0.055 and 0.032 respectively. Similarly, during maximum water level condition also -0.09 and 0.035.

Table 4.2 Summary of vertical and horizontal displacement obtained for CFRD

Load Combinations	Recommended settlement		Computed settlement		Remark
	Vertical settlement	Horizontal settlement	Vertical settlement	Horizontal settlement	
After Construction	Less than 1% of dam height	Less than 50% of vertical settlement	-0.05	0.025	Ok
Steady state conditions @NPL without earthquake	Less than 1% of dam height	Less than 0.5% of height of dam	-0.055	0.032	Ok
Maximum water level conditions	Less than 1% of dam height	Less than 50% of vertical settlement	-0.09	0.035	Ok

4.5 Dynamic Analysis

The dynamic stability of concrete face rock fill dam has been analyzed with the seismic data presented in Table 3.1. Horizontal and vertical acceleration time history data has been produced for the dam from the respective peak ground acceleration (PGA) values based on the 1940 El centro (USA) record. In order to remove site and path effects, deconvolved ATH data have been used in the analyses. In addition the acceleration time history data have been scaled to PGA values of 0.05g horizontal and 0.025g vertical corresponding for site specific maximum design earthquake, and 0.03g horizontal and 0.015g vertical corresponding to site specific operating base earthquake. However, the reason which in stated in section 3.1.6 only maximum design earthquake have been checked. Hence, Figure 4.22 and Figure 4.23 shows that the deconvolved acceleration time history data used related maximum design earthquake horizontal and vertical acceleration for the project site.

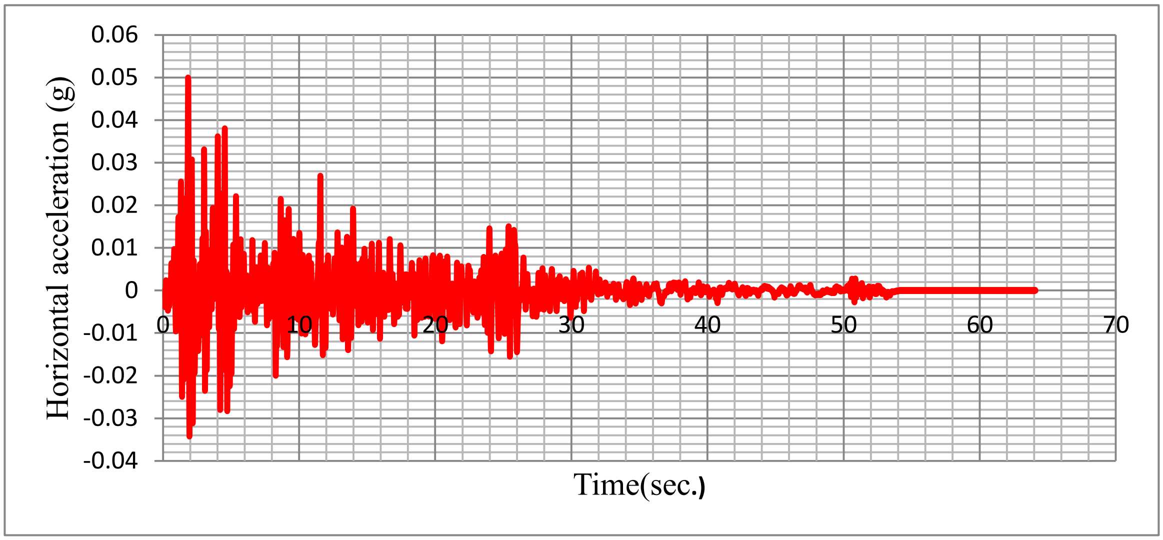

Figure 4.22 Deconvolved ATH of MDE horizontal based on Elcentro 1940

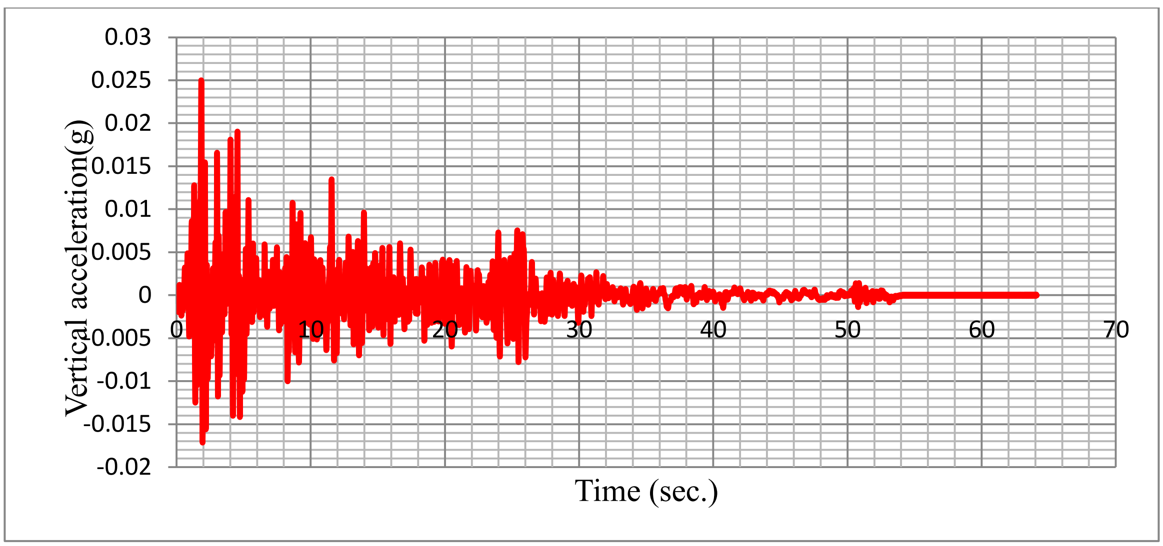

Figure 4.23 Deconvolved ATH of MDE vertical based on Elcentro 1940

4.5.1 Slope stability analysis after earthquake shaking

The pseudo static force is used in a conventional limit equilibrium slope stability analysis. The pseudo static force is treated as a static force and acts in only one direction, whereas the earthquake accelerations act change direction at certain instances of time with short period. The vertical components of the earthquake accelerations are usually neglected in the pseudo static method, and the seismic coefficient usually represents a horizontal force (Jibson R, 2011).

Downstream slope

Maximum factor of safety the dam under dynamic loading condition has been computed to be 1.199 which is shown in Figure 4.24. The computed minimum factor of safety is greater than the minimum requirement suggested by Janson, (1989) for unusual loading condition (i.e. greater than one) which insures that the embankment is stable during earthquake shaking time. The variation in factor of safety of the downstream slope during earthquake has been presented in Figure 4.25 and the factor of safety varies between 1.199 and 2.431.This variation comes from due to the cycling effect of earthquake.

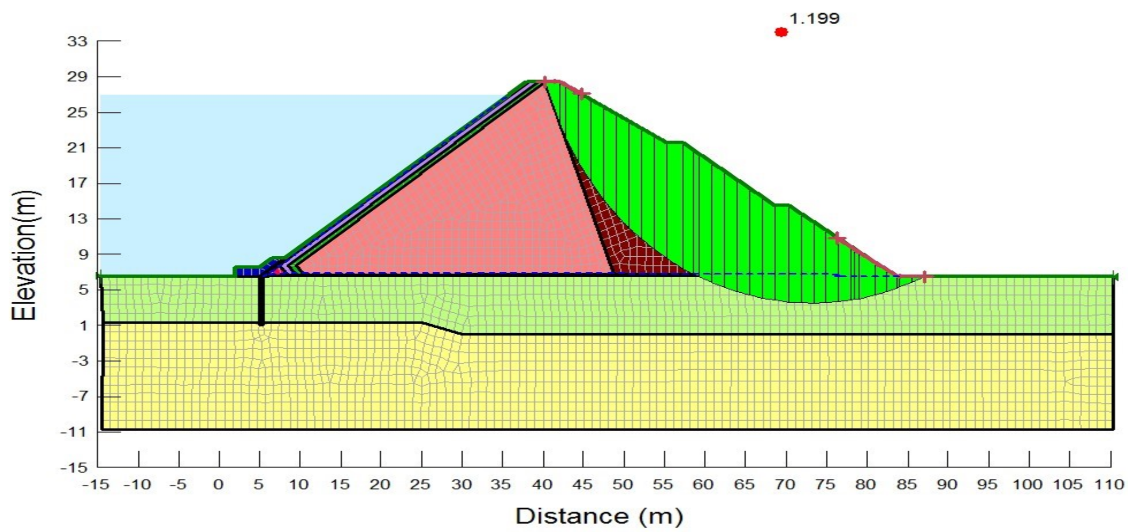

Figure 4.24 Factor of safety for downstream slop steady state after earthquake condition

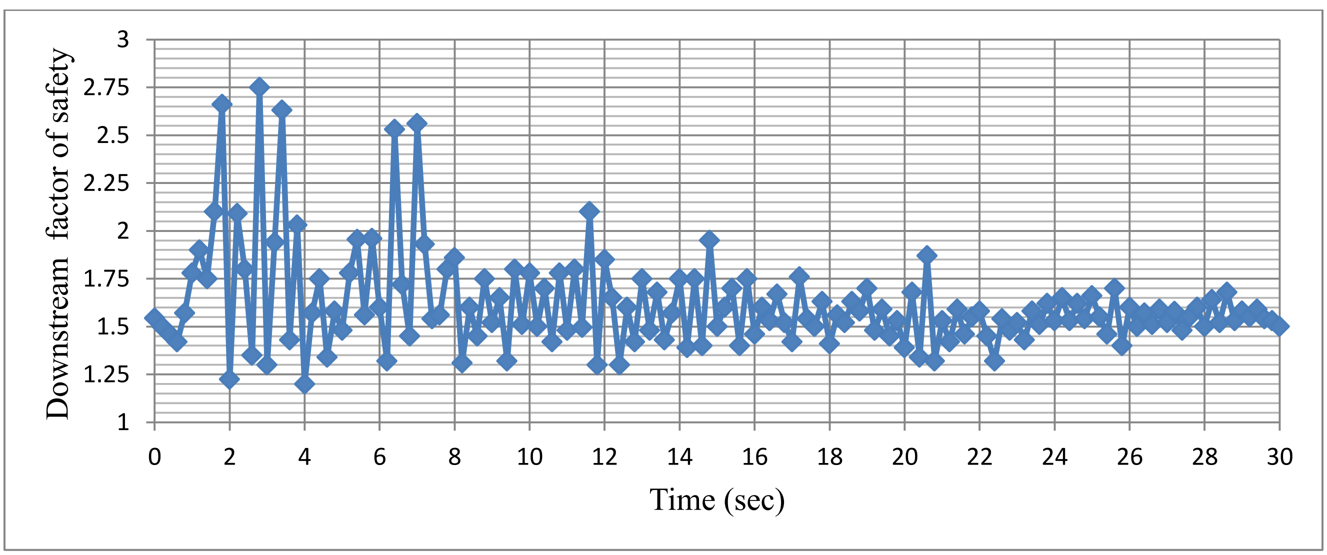

Figure 4.25 Variation of minimum factor of safety for downstream slope during earthquake

Upstream slope stability

Similarly the upstream slope has also been analyzed during earth quake and a minimum factor of safety is 1.256 has been obtained. It is still greater than the minimum requirement suggested by Janson, (1989). So, the embankment is expected to be stable during earthquake. The variation in factor of safety of the downstream slope during earthquake has been presented in Figure 4.27 and the factor of safety varies between 1.256 and 2.92.

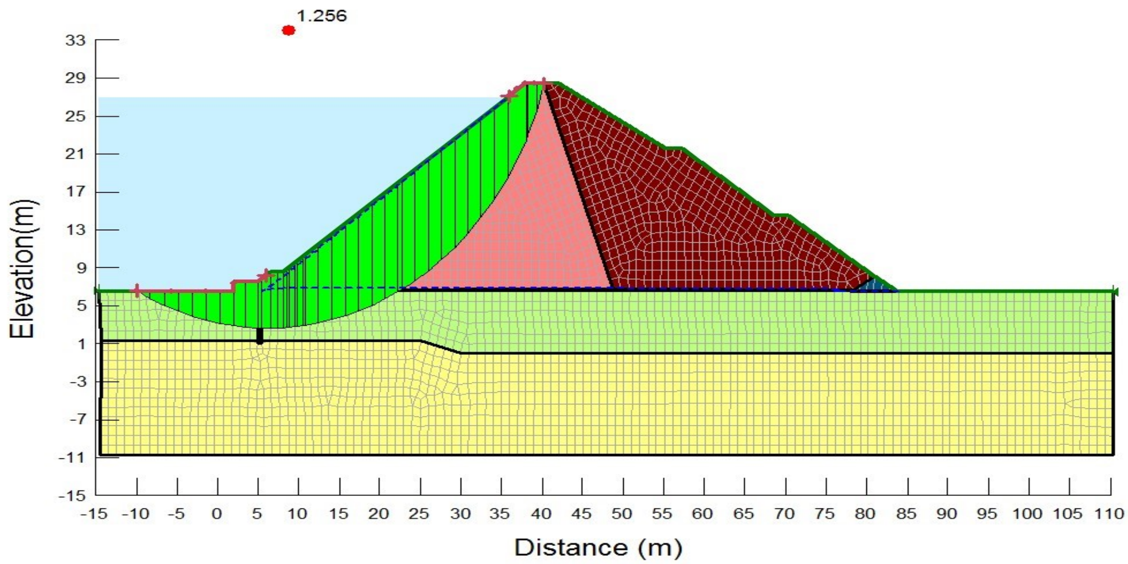

Figure 4.26 Factor of safety for upstream slop steady state after earthquake

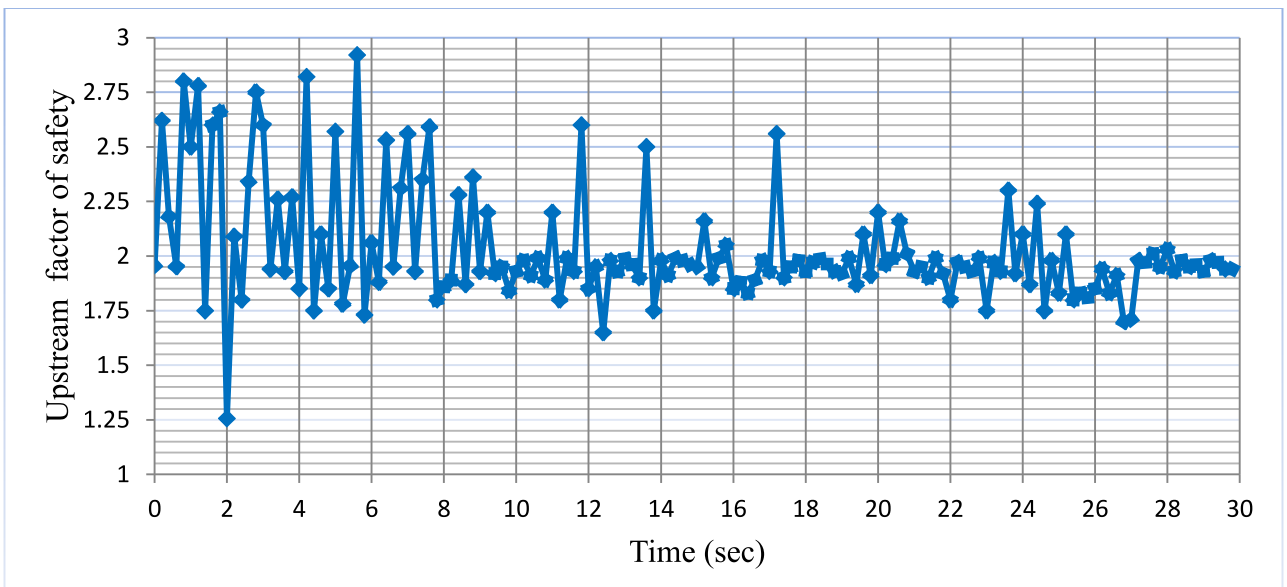

Figure 4.27 Variation of minimum factor of safety for upstream slope during earthquake

In Equivalent Linear Dynamic analysis, QUAKE/W generated stress is incorporated in SLOPE/W to determine factor of safety values for the entire duration of earthquake applied. Slope Stability Analysis using LEM and FEM after Earthquake Shaking has been computed. The variation of safety factor ranges from 1.256 and 2.92 for upstream slope and1.199 and 2.431 for downstream slope after earthquake shake. Therefore, based on the above analysis results, it is possible to conclude that the rock fill dam with concrete face can withstand the earth quake shaking without further potential damage.

4.5.2 Deformation analysis after earthquake shaking

Horizontal and vertical displacement of concrete face rock fill dam section is shown in Figure 4.28 and Figure 4.29 below under dynamic loading condition. The pseudo static force is tends to move the structure from upstream to downstream direction and reverse it with a certain instant of time. When the earth quake duration end seismic force move the dam into upstream direction maximum settlement is occur near to upstream slope. Maximum horizontal displacement for this loading condition is computed to be 0.154 m in the upstream face of the dam. Since the rock fill is essentially dry, earthquake shaking cannot generate excess pore water pressures. With the absence of pore water pressures and the high shear strength of compacted rock fill, this dam type is considered inherently resistant to seismic loading (Cooke, 1991).As a result horizontal deformation satisfy minimum recommended value i.e.0.8% to 1% height of dam (Cooke, 1991) .

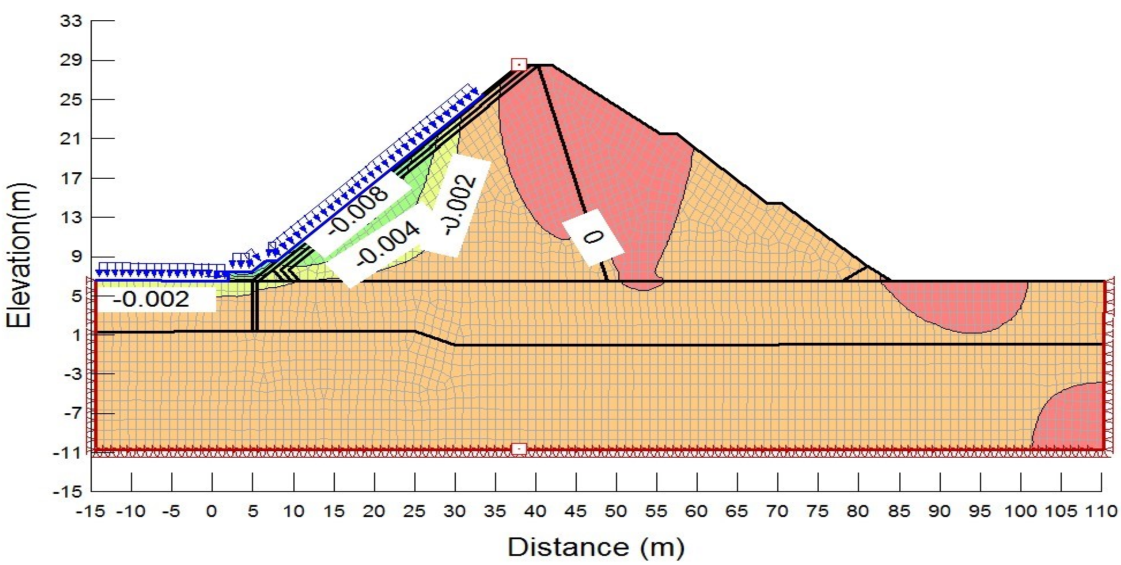

Figure 4.28 Maximum vertical displacements in the dam body and foundation

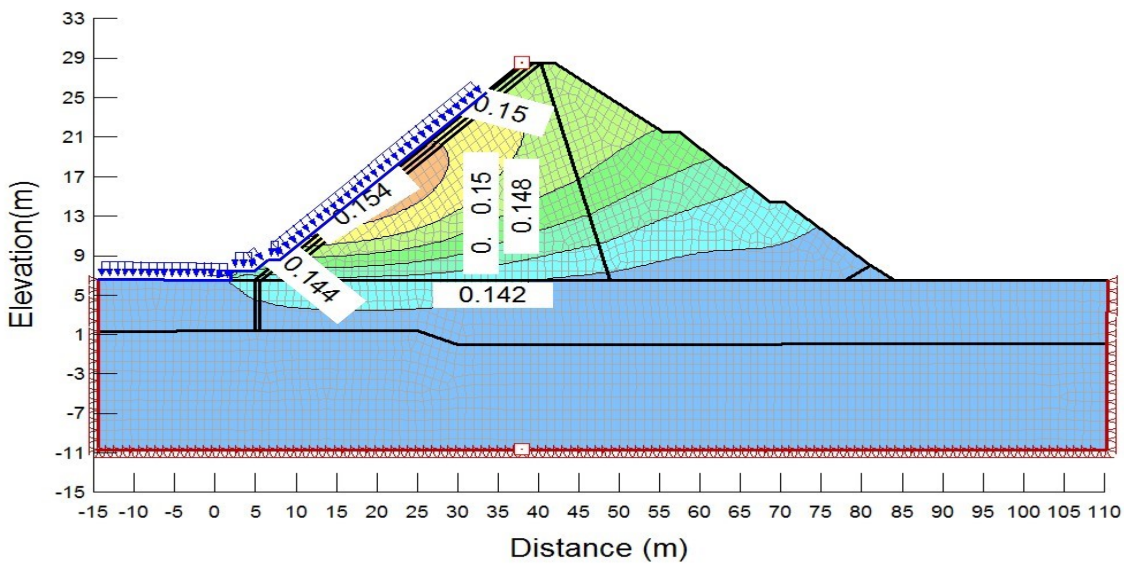

Figure 4.29 Maximum horizontal displacements in the dam body and foundation

4.6 Economic Analysis

4.6.1 Financial saving related to labor, material and equipment

Economic analysis of concrete face rock fill dam and original earth dam design was performed.

In case of earth fill dam with central clay core upstream slope and downstream slope is1:4 and 1:3 respectively with 140m bottom width. However, concrete face rock fill dam upstream and downstream slope is 1:1.5 and 1:1.7 respectively with 74m bottom width (Figure 4.1 and Figure 4.2).From this the total volume of concrete face rock fill dam is much smaller than the existing dam (earth dam with central clay core).So due to smaller cross section and shorter depth of cutoff concrete face rock fill dam is economical over the existing dam. As a result, concrete face rock fill dam was an advantageous by saving **20,239,787.75ETB** from direct and indirect cost. The quantity estimation of major activities for both dam has been shown in Table B1and Table B2 of Appendix B.

4.6.2 Financial saving due to shorter construction period

According to ANRSWRDB, (2007) the annual net return from irrigated agriculture from one hectare of land with application of improved farm input at full development is estimated to be **12,800ETB per ha** and the total command area to be irrigated after completion of the project is ***86ha*** with 200%cropping intensity. As discussed so far under section 2.2.5 of this study, concrete face rock fill dam is considered as the best alternative both technically and economically due to continues progression of dam construction even during rainy season. Construction of Legemera Earth fill irrigation dam project was started in 2010 G.C and completed in 2015 G.C but the new alternative CFRD was completed before one year.

Table 4.3 Production returns profit of concrete face rock fill dam up to end of 2015

Type of the dam	Starting of construction	Construction time per year	End of construction	Time taken for completion
Earth fill dam	2011	9 month *	2015	5 year
CFRD	2011	12 month**	2014	3.75 year
Construction time difference between earth fill and CFRD				5-3.75 =1.25 year take one year
Production return obtained from CFRD up to 2015 G.C with 200% cropping intensity				
Cropping intensity	Time of saving(year)	Rate (ETB/ha/year)	Command area (ha)	Production
200%	1	12800	86	=2*1year*12,800ETB/ha/year*86ha =**2,201,600 ETB/year**

*ANRSWRDB, 2007 ** USACE, 1994

So, even when production benefit alone is considered at the end of 2015 G.C, use of concrete face rock fill dam can give a return profit of **2,201,600 ETB.** The calculated direct and indirect cost of material, labor and equipment in case of CFRD and earth dam with central clay core have been presented in Table B1 and Table B2 of Appendix B. Based on this economic advantage of CFRD saving an amount of **20,239,787.75 ETB.** So the net benefit is the sum of direct and indirect cost and one year return profits which give as **22,441,387.75 ETB** can be obtained at the end of 2015 G.C.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This research presented evaluating and stability analysis of Legemera micro earth dam, a major embankment dam material using clay core, the biggest challenge is the hauling distance at an economically feasible distance. In addition earth fill and rock fill dams with clay core have weather dependent during construction time. Therefore, they need a long time for completion of construction. Due to these problems and for constructing more economical earth dams an alternative approach is developed by substituting clay core with concrete face rock fill dam. They are mainly used in areas where natural impermeable materials of sufficient quality or quantity are not available.

The quantities of seepage through foundation and dam body reduced in 2.42×10^{-6} m^3/sec. Hence, concrete face rock fill dam is remarkable in terms of preventing seepage as compared to earth dam with central clay.

Stability of slope under static and pseudo-static condition is investigated in terms of factor of safety using Morgenstern-Price limit equilibrium method. The slope stability analysis is important consideration in design of embankment dam using SLOPE/W of Geo-studio 2012 software and the analyzed conditions like upstream and downstream slopes during end of construction, normal pool level and maximum water level condition (with or without earthquake). Hence, factor of safety for upstream and downstream slopes of concrete rock fill dam design satisfies the minimum requirement suggested by (USACE, 2003), design standard of embankment dam and guideline under all loading condition; such as, end of construction, steady state and maximum water level.

Deformations of an embankment dam start occurring during the construction of the dam. These deformations are caused by the increase of effective stresses during the construction by the consecutive layers of fill material and also by effects of creep of material. The results found in this study were in line with ICOLD, (1993). Therefore, concrete face rock fill dam is satisfies minimum recommended deformation with respect to horizontal and vertical deformations.

The major significance of this alternative dam entirely built in rock fill were not only reduces volume of the dam while reducing total cost of the dam but also shortens the time required to accomplish the project. It is quite interesting to mention the amount of cost that was proposed to be invested on this project was **75,814,699.76 ETB,** but now the total amount of cost was reduced to **55,574,912 ETB** related to related to direct and indirect cost in addition to this **2,202,600 ETB** profit due to shorten time required to accomplish the project. The total amount of money saved by this work was **22,441,387.75 ETB.**

In general, the results of the proposed method and embankment dam design considerations can assist designers of concrete face rock fill dam in judgment about dam stability during static and dynamic analysis as well as economic point of view.

5.1 Recommendations

From the analysis conducted on Legemera concrete face rock fill irrigation dam the following recommendations has been given

- The location of phreatic line in case of alternative dam design located between dam body and alluvial foundation. So horizontal filter material will be recommended to avoid existing of piping effect in alluvial foundation.
- The joint connecting face slab and plinth (or toe slab) is the most critical element in the dam. When it ruptures leakage will occur. Hence, I will be recommended that providing adequate filter zones to avoid washing out of foundation material.
- For the region which experience severe weather condition throughout the years, construction of concrete face rock fill dam is better in such a way that, construction of the dam can be proceeds without significant interruption.

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APPENDIXES

Appendix A

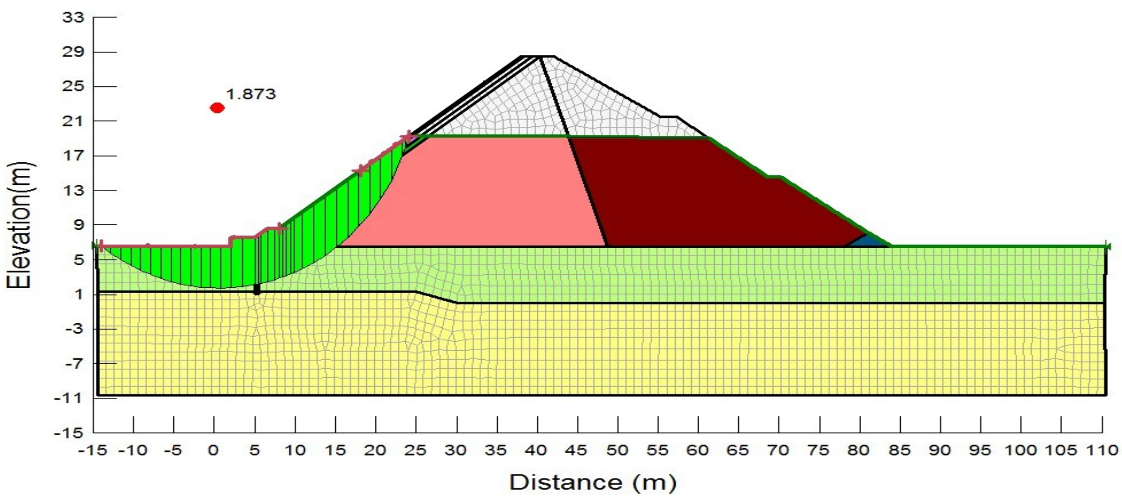

Figure A 1 Factor of safty for upstream slope at intermediate stage (CFRD)

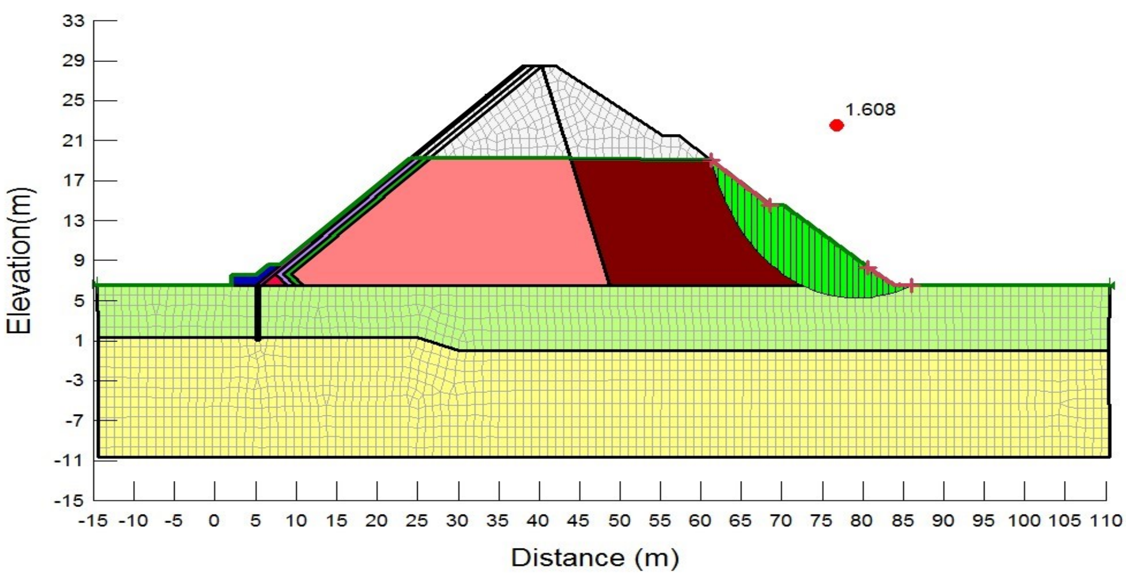

Figure A 2 Factor of safety for downstream slope at intermediate stage (CFRD**)**

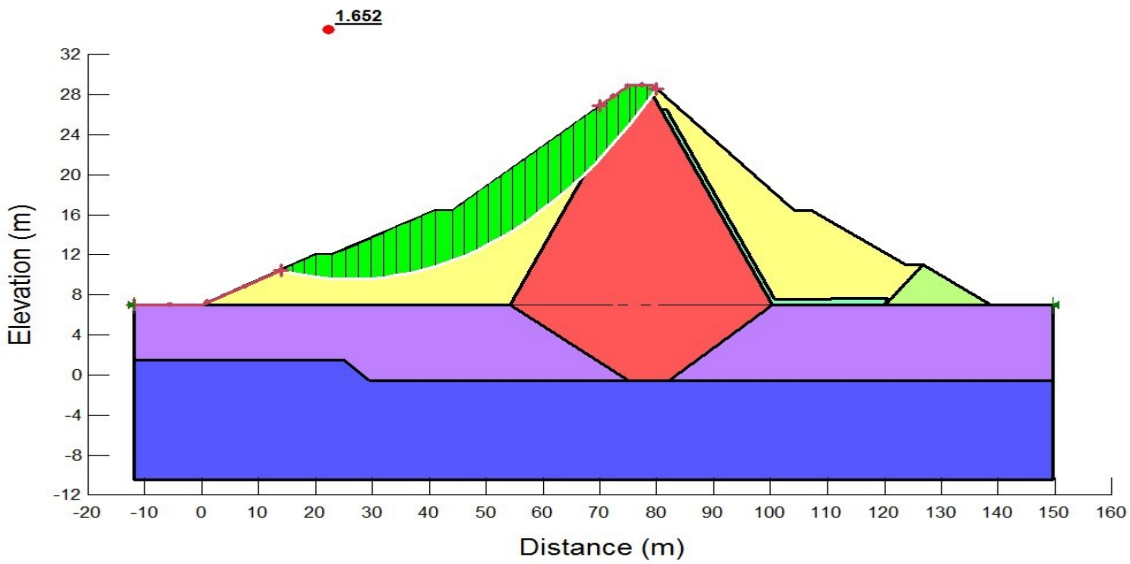

Figure A 3 Factor of safety for upstream slope at end of construction (exiting dam)

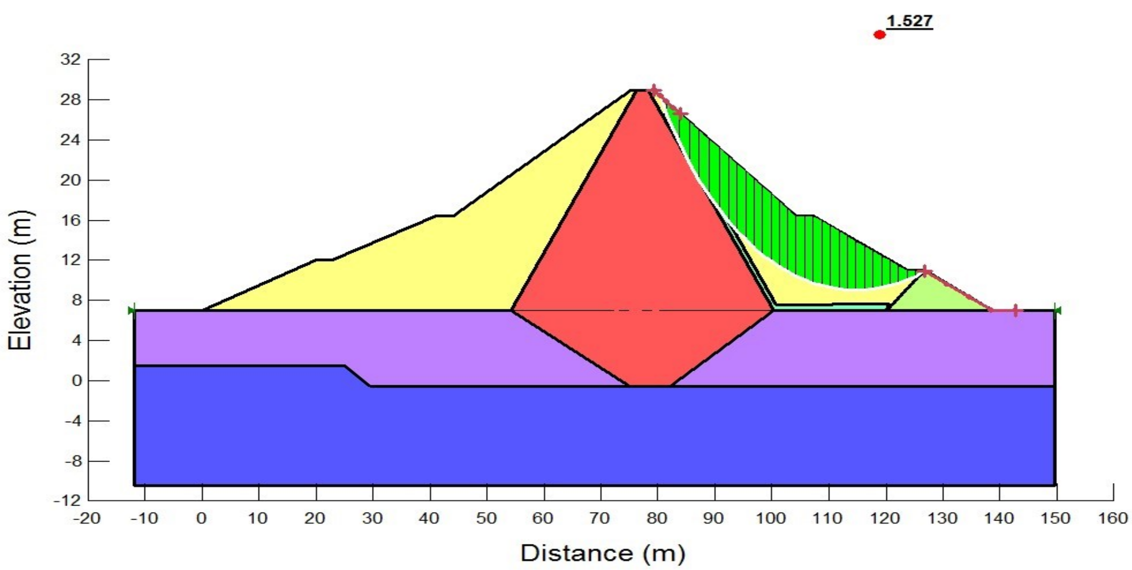

Figure A 4 Factor of safety for downstream slope at end of construction (exiting dam)

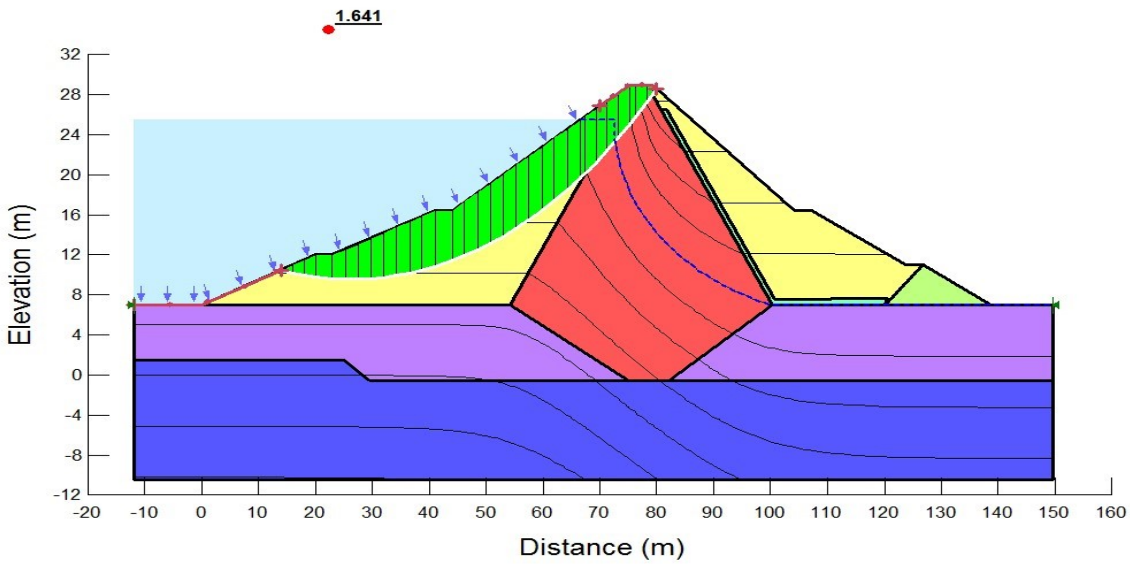

Figure A 5 Factor of safety for upstream slope during steady state condition (existing dam)

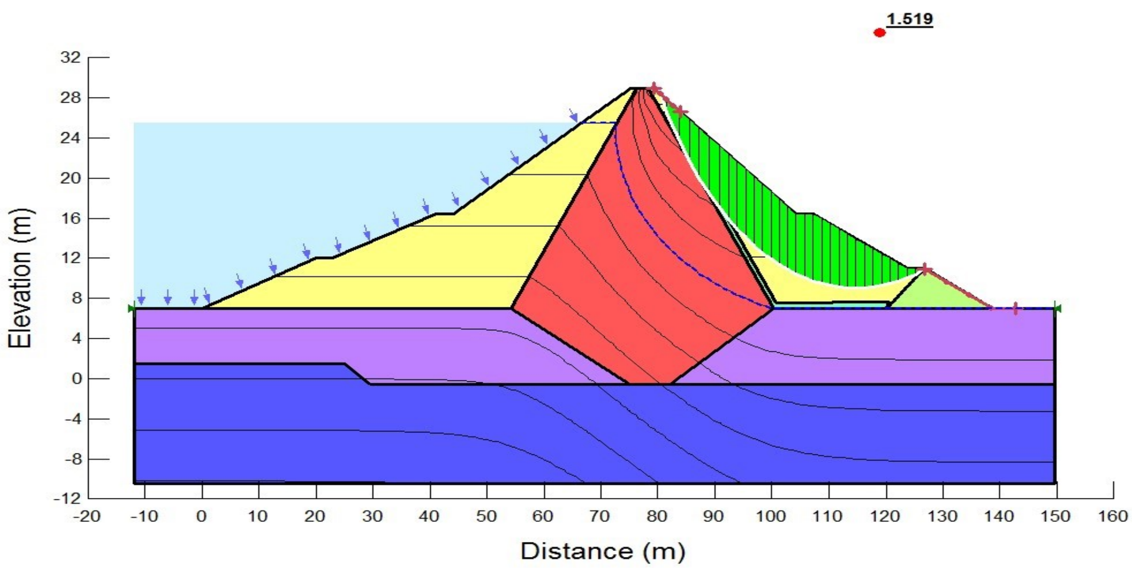

Figure A 6 Factor of safety for downstream slope during steady state condition (existing dam)

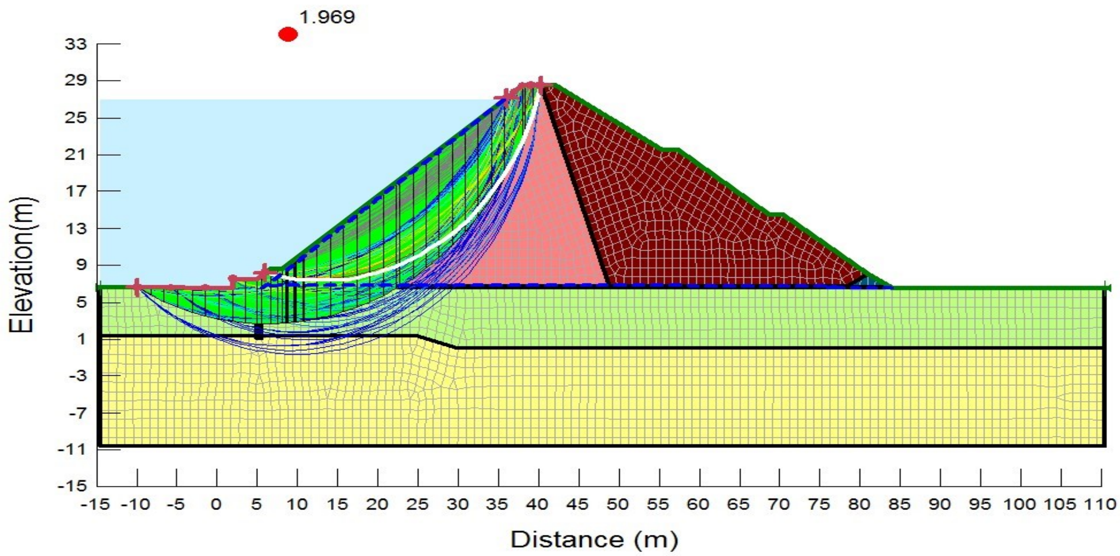

Figure A 7 FoS for upstream slope during maximum water level condition for all slip surfaces

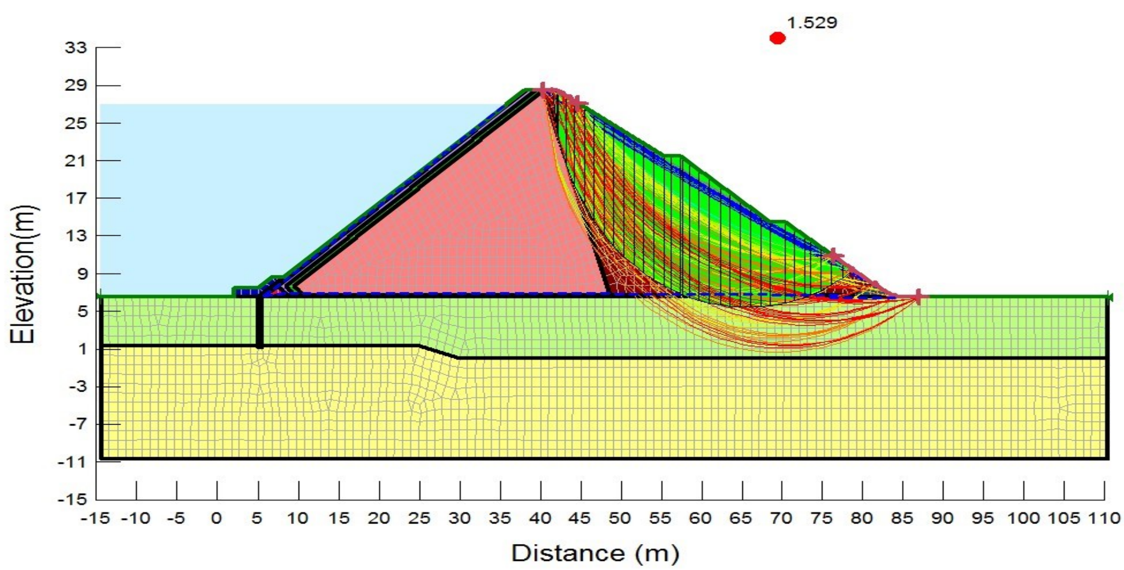

Figure A 8 FoS for downstream slope at maximum water level condition for all slip surfaces

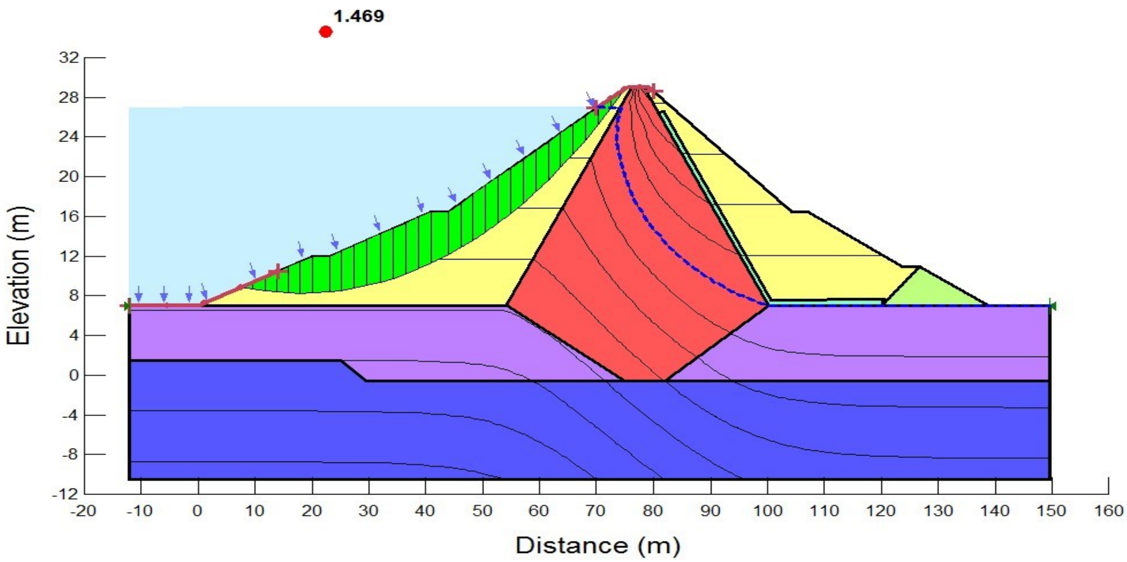

Figure A 9 Factor of safety for upstream slope during steady state condition (existing dam)

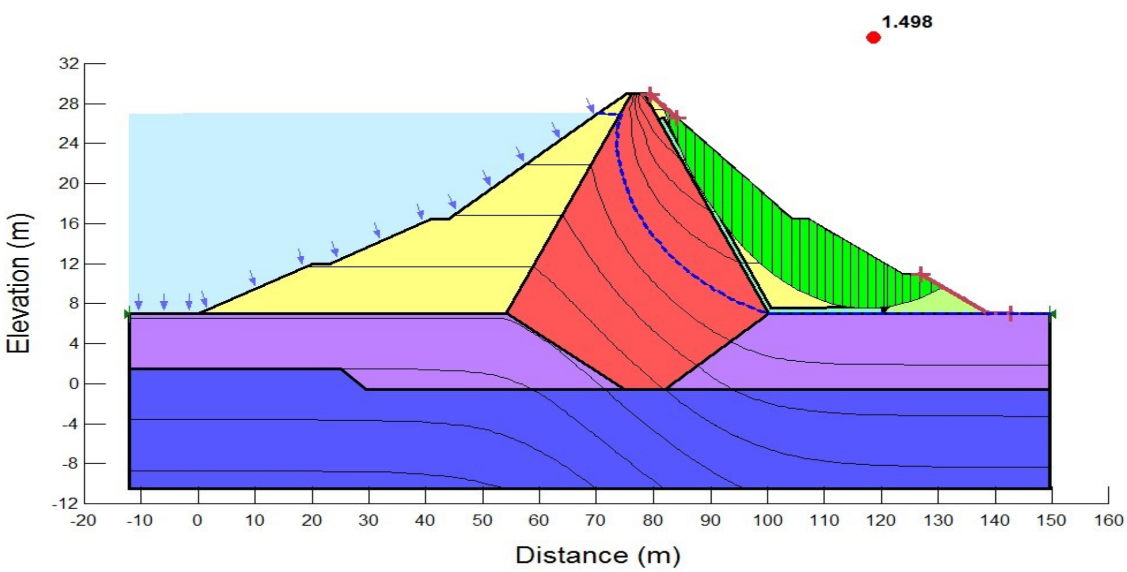

Figure A 10 FoS for downstream slope at steady state condition (existing dam)

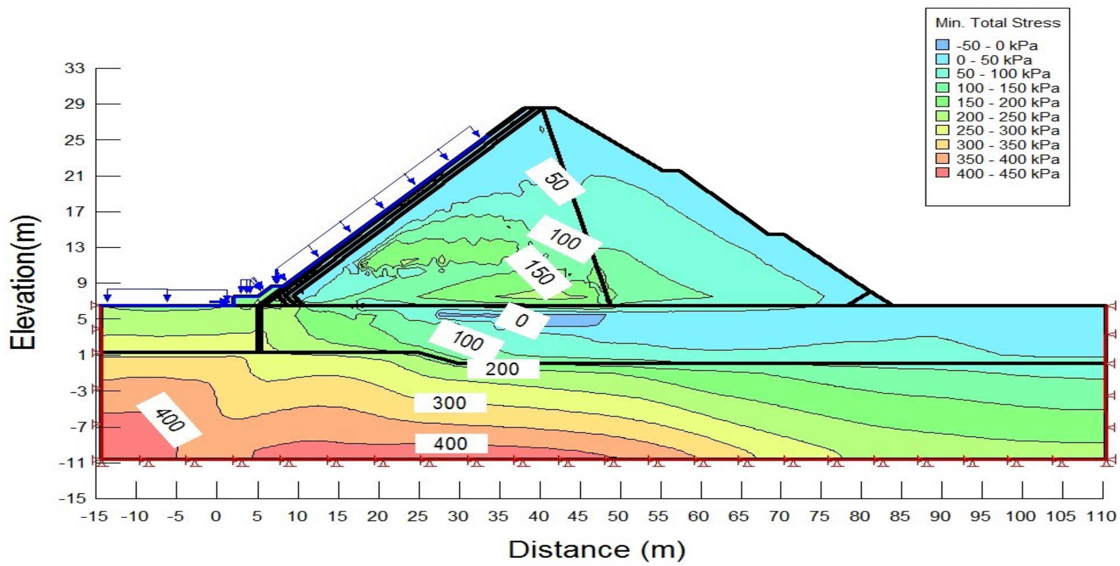

Figure A 11 Contours of minimum total stress for alternative design at NPL

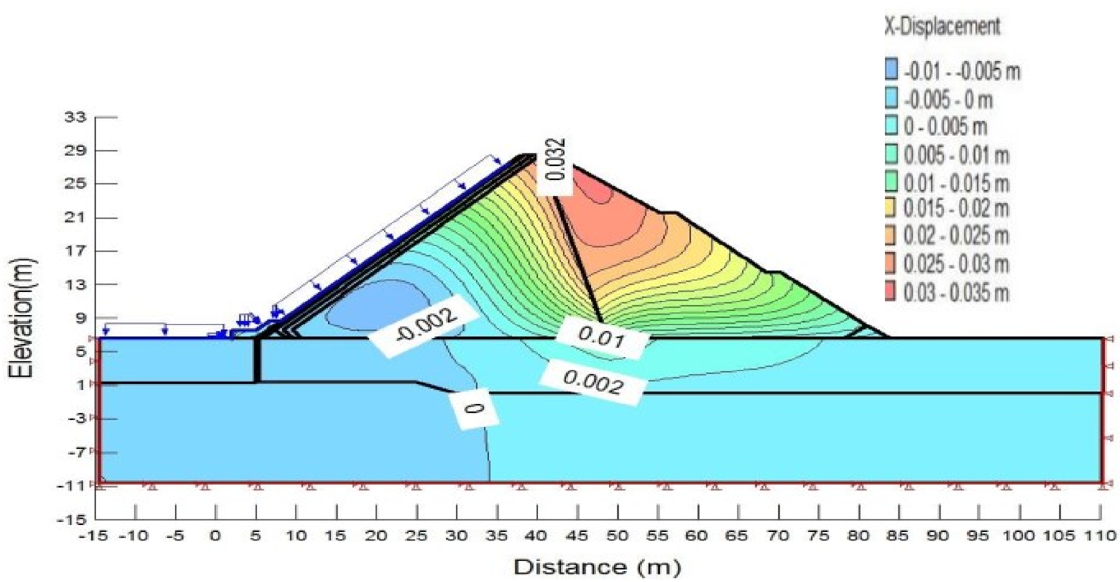

Figure A 12 Contours of horizontal displacement at maximum pool level condition

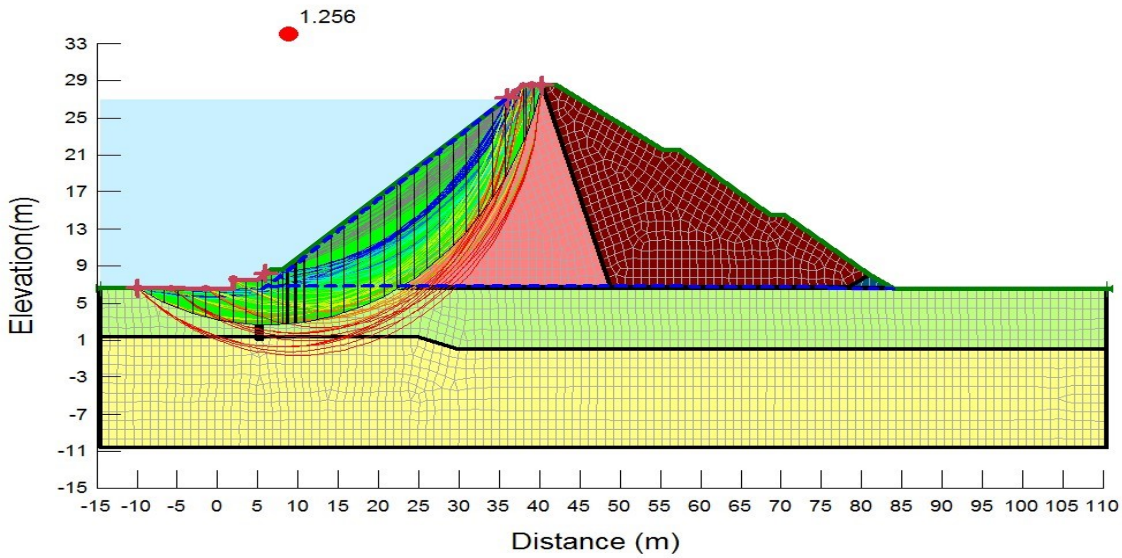

Figure A 13 FoS for upstream slope during earth quake condition for all slip surfaces

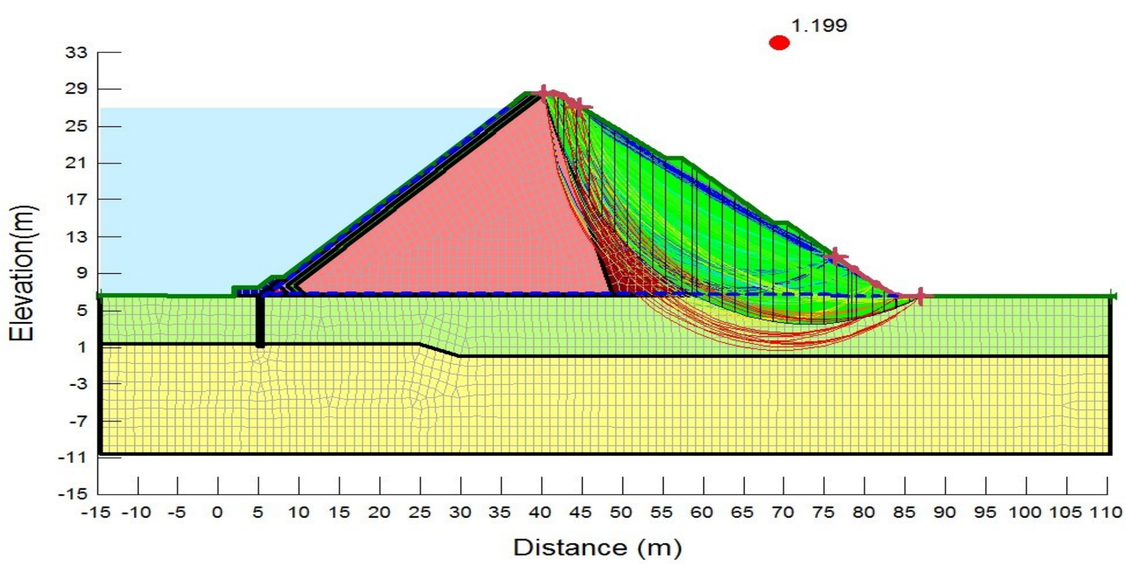

Figure A 14 FoS for downstream slope during earth quake condition for all slip surfaces

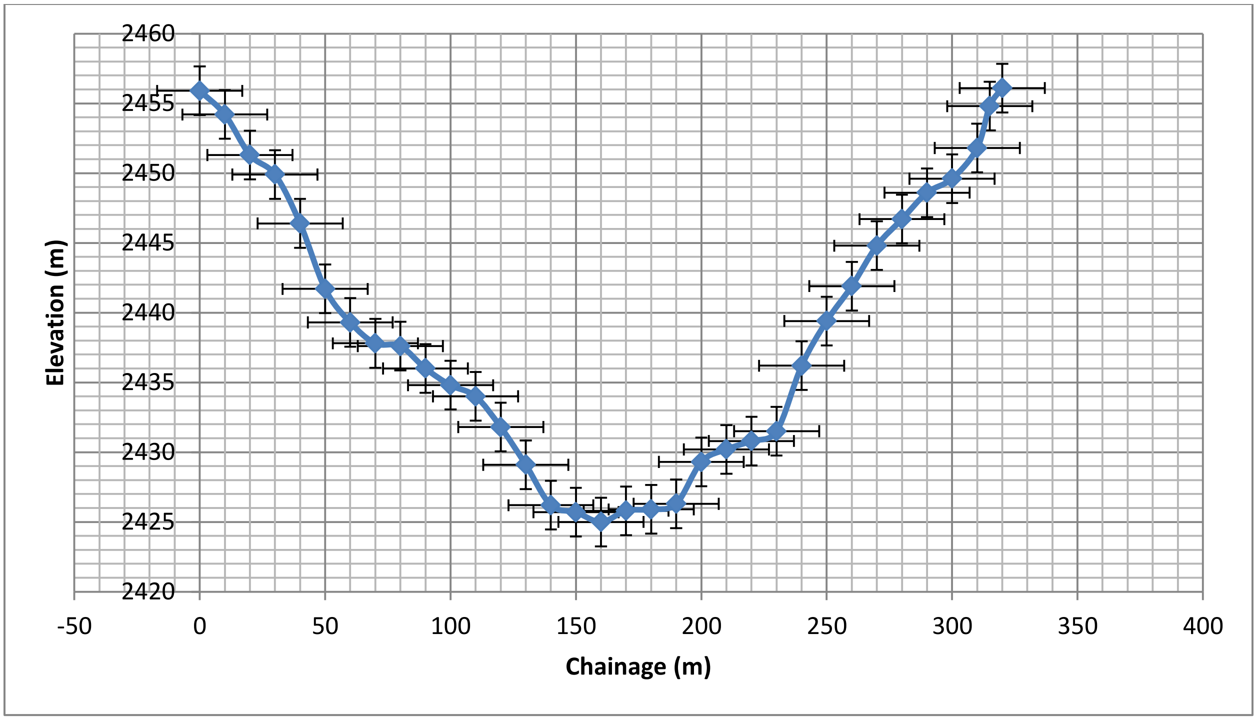

Figure A 15 Dam axis coordinates of key trench profile

Appendix B

Table B 1 Work volume estimated for major activities of the alternative design

Elevation (m)	Length (m)	Volume of CFRD (m3)									Volume of earth dam (m3)			
		Cutoff	Face slab	1A	2A	2B	3A	3B	3C	3D	Clay	Shell	Filter	Rock Toe
2426	50	0	0	0	0	0	0	0	0	0	1637	0	0	0
2429	70	87.5	0	0	0	0	0	0	0	0	4424.21	0	0	0
2431.5	110	137.5	0	0	0	0	0	0	0	0	10755.9	0	0	0
2434	130	0	157.3	1153.8	292.5	487.5	487.5	13636	12787.3	585	14097.2	23294.1	5265	3250
2436.5	150	0	179.29	486	0	562.5	562.5	14386	12886.5	0	14296.9	25716	393.75	2250
2439	190	0	223.82	0	0	712.5	712.5	16785	15479.1	0	15615.2	29605.8	498.75	0
2441.5	213	0	247.13	0	0	798.75	798.75	15210	16407.7	0	14718.3	26545.1	559.125	0
2444	226	0	258.26	0	0	847.5	847.5	12310	16406.2	0	12642.4	22785.3	593.25	0
2446.5	248	0	279.5	0	0	930	930	9307.8	16840.7	0	10619.4	20507.1	651	0
2449	270	0	303.82	0	0	1012.5	1012.5	7134.3	17136.9	0	8015.63	17425.8	708.75	0
2451.5	294	0	328.25	0	0	1102.5	1102.5	5465.5	12908.4	0	6798.75	17088.8	771.75	0
2454	306	0	333.62	0	0	1147.5	1147.5	2264.4	12074.8	0	3538.13	11761.7	803.25	0
2456	320	0	343.28	0	0	1200	1200	998.4	8785.92	0	1684.48	6000	0	0
Total		**225**	**2654.3**	**1639.8**	**292.5**	**8801.3**	**8801.3**	**97497**	**141713**	**585**	**118843**	**200730**	**10244.6**	**5500**

Table B 2 Bill of Quantity of CFRD and Earth dam

	Item No	Description	Unit	Quantity (clay core)	Quantity (CFRD)	Rate (ETB)	Amount (ETB) (clay core)	Amount (ETB) (CFRD)
	1	Clearing, grubbing and strip topsoil to a depth of 0.5m	m^2	235221.3	235221.3	7.38	1735933.194	1735933.19
	2	Dam foundation top soil clearance(0.3m)	m^2	45298	22011	51.92	2351872.16	1142811.12
	3	Excavation for embankment foundation to max depth of one meter depth	m^3	45298	22011	31.45	1424622.1	692245.95
	4	Excavation for cutoff trench up to 5m depth at U/S and 8m at the middle	m^3	68266	1595	59.36	4052269.76	94679.2
	5	Cement slurry treatment of rock surface in cutoff trench	m^3	0	1196.25	196.65	0	235242.563
Earth dam	6	Excavated Random shell material	m^3	200730	0	92.89	18645809.7	0
	7	Hauling of Excavated shell material from borrow area within 5 km	m^3	200730	0	54.18	10875551.4	0
	8	Compacting of shell material	m^3	200730	0	13.74	2758030.2	0
	9	Excavated Impervious clay material	m^3	118843	0	34.86	4142866.98	0
	10	Hauling of Excavated Impervious clay material from quarry site within 31 km	m^3	118843	0	209.48	24895231.64	0
	11	Compacting of clay material	m^3	118843	0	9.06	1076717.58	0
	12	Transition chimney filter obtained from the river bed screened	m^3	10245	0	290.49	2976070.05	0
	13	Excavated and Hauling of Rock toe	m^3	5500	0	159.95	879725	0
CFRD	14	Excavation, and hauling Random mix of silt gravel material	m^3	0	1639.75	92.89	0	152316.378
	15	Excavation and hauling Random mix of silt gravel material	m^3	0	8801.25	115.6	0	1017424.5
	16	Concrete face slab	m^3	0	2654.263	3668.9	0	9738225.5
	17	Excavation and hauling of transition material	m^3	0	8801.25	220.5	0	1940675.63
	18	Compacting of 2A,2B and 3A	m^3	0	19242.25	13.74	0	264388.515
	19	Excavated of 3B	m^3	0	97496.84	92.89	0	9056481.47
	20	Hauling of 3B	m^3	0	97496.84	54.18	0	5282378.79
	21	Excavated of 3C	m^3	0	141713.4	92.89	0	13163757.7
	22	Hauling of 3C	m^3	0	141713.4	54.18	0	7678032.01
	23	Compacting of 3B and 3C	m^3	0	239210.2	13.74	0	3286748.7
	24	Excavated and hauling of Rock toe	m^3	0	585	159.95	0	93570.75
Total cost							**75814699.8**	**55574912**
Difference =75,814,699.76-55574912= 20239787.75 ETB								

Table B 3 Type of construction equipment and its hourly cost (AWWCE, 2008)

Type of construction equipment and its hourly cost											
Dozer	Loader	Sheep foot Roller	Roller	Excavator with bucket	Excavator with jackhammer	Damp truck for clay			Damp truck for shell material		
						0-10km	10-30km	30-50km	0-10km	10-25km	25-50km
2950 Birr/hr.	580 Birr/hr.	680 Birr/hr.	530 Birr/hr.	1500 Birr/hr.	2650 Birr/hr.	4.45Birr/km for1m^3	4.65Birr/km for1m^3	4.85Birr/km for1m^3	7.65Birr/km for 1m^3	7.85Birr/km for 1m^3	8.15Birr/km for 1m^3
						Hauling distance is 31 km and 8m^3/hr. *			Hauling distances 5 km and 12m^3/hr. *		
						=31km*8m^3/hr.*4.85Birr/km/m^3			=5km*12m^3/hr.*7.65Birr/km/m^3		
						=1302 Birr/hr.			**=459 Birr/hr.**		

* Interview senior site engineer who work in Legemera site

Table B 4 Analysis of direct and indirect unit cost

- 1 -

Analysis Sheet for Direct and Indirct Unit cost

Project: **Excavation and Earth Work** Hourly output***: 500 m^2/hr.
Work Item: Clearing of bushes and trees Equipment: Dozer
Total quantity : 1 m^2 Result: 7.38 Birr/m^2

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	Indexed Hourly Cost**	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time keeper	1	1	22	22	Dozer	1	2950	2950
				0	Oprator	1	1	50	50				0
				0	Forman	1	0.75	25	18.75				0
				0	Site Engineer	1	0.5	47	23.5				0
					PM	1	0.25	50	12.5				0
Total (1:-01)				0	**Total (1:02)**				126.75	**Total (1:03)**			2950

A= Materials Unit Cost 0 Birr/m^2

B= Manpower Unit Cost $\frac{\text{Total of (1:02)}}{\text{Hourly Output:}}$ $\frac{0.25}{1.44}$ Birr/m^2

C= Equipment Unit Cost $\frac{\text{Total of (1:03)}}{\text{Hourly output:}}$ 5.90 Birr/m^2

Direct Cost of Work Item = A+B+C = 6.15 Birr/m^2

Over head cost : 5% 0.31 "

Profit Cost: 15% 0.92 "

- 2 -

Analysis Sheet for Direct and Indirct Unit cost

Project: **Excavation and Earth Work** Hourly output***: 80 m^2/hr.
Work Item: Dam Foundation top soil clearance (0.3 m depth) Equpment: Dozer
Total quantity : 1 m^2 Result 51.92 Birr/m^2

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time keeper	1	1	22	22	Dozer	1	2950	2950
				0	Oprator	1	1	50	50				0
				0	Forman	1	0.75	25	18.75				0
				0	Site Engineer	1	0.5	47	23.5				0
				0	PM	1	0.25	50	12.5				0
Total (1:-01)				0	**Total (1:02)**				126.75	**Total (1:03)**			2950

A= Materials Unit Cost 0 Birr/m^2

B= Manpower Unit Cost <u>1.58</u> Birr/m^3
Total of (1:02)
Hourly Output:

C= Equipment Unit Cost 36.88 Birr/m^2
Total of (1:03)
Hourly output: _______________

Direct Cost of Work Item = A+B+C = 38.46 Birr/m^2
Over head cost : 10% 3.85 "
Profit Cost: 25% 9.61 "
Total Unit Cost : 51.92 Birr/m^2

- 3 -

Analysis Sheet for Direct and Indirct Unit cost

Project: **Excavation and Earth Work.** Hourly output***: 62 m^3/hr.
Work Item: Bulk excav. to a depth of 8m Equipment:Excavator with bucket
Total Qunt. 1 m^3 Result 31.45 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	20	20	Excavator	1	1500	1500
				0	Oprator	1	1	50	50				0
				0	Forman	1	0.75	25	18.75				0
				0	Site Engineer	1	0.5	47	23.5				0
				0	PM	1	0.25	50	12.5				0
Total (1:-01)				0	**Total (1:02)**				124.75	**Total (1:03)**			1500

A= Materials Unit Cost 0 Birr/m^3

B= Manpower Unit Cost <u>2.0121</u> Birr/m^3
Total of (1:02)
Hourly Output:

C= Equipment Unit Cost 24.19 Birr/m^3
Total of (1:03)
Hourly output: _______________

Direct Cost of Work Item = A+B+C = 26.21 Birr/m^3
Over head cost : 5% 1.31 "
Profit Cost: 15% 3.93 "
Total Unit Cost : 31.45 Birr/m^3

- 4 -

Analysis Sheet for Direct and Indirct Unit cost

Project:	**Excavation and Earth Work.**	Hourly output***:	56 m^3/hr.
Work Item:	Excation and loading Clay Material	Equipment:Excavator	
Total Qunt.	1 m^3	Result	34.86 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	22	22	Excavator	1	1500	1500
				0	Oprator	1	1	50	50				0
				0	Forman	1	0.75	25	18.75				0
				0	Site Engineer	1	0.5	47	23.5				0
				0	PM	1	0.25	50	12.5				0
Total (1:01)				0	**Total (1:02)**				126.75	**Total (1:03)**			1500

A= Materials Unit Cost 0 Birr/m^3

B= Manpower Unit Cost <u>2.2634</u> Birr/m^3
Total of (1:02) / Hourly Output:

C= Equipment Unit Cost 26.79 Birr/m^3
Total of (1:03) / Hourly output:

Direct Cost of Work Item = A+B+C =		29.05	Birr/m^3
Over head cost :	5%	1.45	"
Profit Cost:	15%	4.36	"
Total Unit Cost :		34.86	Birr/m^3

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Analysis Sheet for Direct and Indirct Unit cost

Project:	**Excavation and Earth Work.**	Hourly output****:	8 m^3/hr.
Work Iten	Hualing Clay material with hualing distance 31km	Equipmer Damptruck	
Total Qunt.	1 m^3	Result	209.48 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kep	1	1	22	22	Damptruc	1	1302	1302
				0	Oprator	1	1	42	42				0
					Forman	1	0.75	25	18.75				
					Site Engi	1	0.25	47	11.75				
Total (1:-01)				0	**Total (1:02)**				94.5	**Total (1:03)**			1302

A= Materials Unit 0 Birr/m^3

B= Manpower Unit Cost <u>11.8125</u> Birr/m^3
Total of (1:02) / Hourly Output:

C= Equipment Unit Cost 162.75 Birr/m^3
Total of (1:03) / Hourly output:

Direct Cost of Work Item = A+B+C =		174.56	Birr/m^3
Over head cost :	5%	8.73	"
Profit Cost:	15%	26.18	"
Total Unit Cost :		209.48	Birr/m^3

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Analysis Sheet for Direct and Indirct Unit cost

Project:	**Excavation and Earth Work.**	Hourly output***:	36 m^3/hr.
Work Item:	Excation and loading Shell Material	Equipment:Excavator with jackhammer	
Total Qunt.	1 m^3	Result	92.89 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	22	22	Excavator	1	2650	2650
				0	Oprator	1	1	60	60				0
					Forman	1	0.75	25	18.75				
					Site Engineer	1	0.5	47	23.5				
					PM	1	0.25	50	12.5				
Total (1:01)				0	**Total (1:02)**				136.75	**Total (1:03)**			2650

A= Materials Unit Cost 0 Birr/m^3

B= Manpower Unit Cost = Total of (1:02) / Hourly Output: 3.7986 Birr/m^3

C= Equipment Unit Cost = Total of (1:03) / Hourly output: 73.61 Birr/m^3

Direct Cost of Work Item = A+B+C = 77.41 Birr/m^3

Over head cost :	5%	3.87	"
Profit Cost:	15%	11.61	"
Total Unit Cost :		92.89	Birr/m^3

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Analysis Sheet for Direct and Indirct Unit cost

Project:	**Excavation and Earth Work.**	Hourly output****:	12 m^3/hr.
Work Item:	Hualing shell material with hualing distance 5 km	Equipment: Damptruck	
Total Qunt.	1 m^3	Result	54.18 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	22	22	Damptruck	1	459	459
				0	Oprator	1	1	42	42				0
				0	Forman	1	0.75	25	18.75				
Total (1:01)				0	**Total (1:02)**				82.75	**Total (1:03)**			459

A= Materials Unit Cost 0 Birr/m^3

B= Manpower Unit Cost = Total of (1:02) / Hourly Output: 6.8958 Birr/m^3

C= Equipment Unit Cost = Total of (1:03) / Hourly output: 38.25 Birr/m^3

Direct Cost of Work Item = A+B+C = 45.15 Birr/m^3

Over head cost :	5%	2.26	"
Profit Cost:	15%	6.77	"
Total Unit Cost :		54.18	

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Analysis Sheet for Direct and Indirct Unit cost

Project: **Excavation and Earth Work.** Hourly output***: 85 m^3/hr.
Work Item: Compacting of Clay material Equipment: Roller
Total Qunt. 1 m^3 Result 9.06 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	22	22	Roller	1	520	520
				0	Oprator	1	1	45	45				0
					Forman	1	0.75	25	18.75				
					Site Engineer	1	0.5	47	23.5				
					PM	1	0.25	50	12.5				
Total (1:01)				0	**Total (1:02)**				121.75	**Total (1:03)**			520

A= Materials Unit Cost 0 Birr/m^3

B= Manpower Unit Cost = Total of (1:02) / Hourly Output: 1.4324 Birr/m^3

C= Equipment Unit Cost = Total of (1:03) / Hourly output: 6.12 Birr/m^3

Direct Cost of Work Item = A+B+C = 7.55 Birr/m^3
Over head cost : 5% 0.38 "
Profit Cost: 15% 1.13 "
Total Unit Cost : 9.06 Birr/m^3

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Analysis Sheet for Direct and Indirct Unit cost

Project: **Excavation and Earth Work.** Hourly output***: 70 m^3/hr.
Work Item: Compaction of Shell Material Equipment: Sheep foot Roller
Total Qunt. 1 m^3 Result 13.74 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Time kepeer	1	1	22	22	Sheep foot I	1	680	680
				0	Oprator	1	1	45	45				0
					Forman	1	0.75	25	18.75				
					Site Engineer	1	0.5	47	23.5				
					PM	1	0.25	50	12.5				
Total (1:-01)				0	**Total (1:02)**				121.75	**Total (1:03)**			680

A= Materials 0 Birr/m^3

B= Manpower Unit Cost = Total of (1:02) / Hourly Output: 1.7393 Birr/m^3

C= Equipment Unit Cost = Total of (1:03) / Hourly output: 9.71 Birr/m^3

Direct Cost of Work Item = A+B+C = 11.45 Birr/m^3
Over head cost : 5% 0.57 "
Profit Cost: 15% 1.72 "
Total Unit Cost : 13.74 Birr/m^3

Analysis Sheet for Direct and Indirct Unit cost

Project: ***Concrete Work***
Work Item: C-25 RC (Mechanical Mix) 1:2:3
Total Qunt. 1 m^3

Labour Hourly Output***: 12 m^3 / hr.
Equipment: Transit Mixer & Vibrator
Result: 3668.92 Birr/m^3

Material Cost (1:01)					Labour (1:02)					Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Trade	No.	UF	** Indexed Hourly Cost	Hourly Cost	Type of Equipment	No.	Hourly Rental	Hourly Cost
Cement	Qnt.	3.6	230	828	Foreman	1	1	25	25	Transit mix	1	850	850
Sand	m3	0.51	120.00	61.2	Metal worker	4	1	20	80	Vibrator	1	650	650
Gravel (02)	m^3	0.76	450.00	342	D/L	6	1	15	90				0
Water	m^3	0.213	0.75	0.160	Site Engineer	1	0.5	47	23.5				0
Metal	Kg	29.3	57.4	1681.82	PM	1	0.25	50	12.5				0
Total (1:01)				2913.18	**Total (1:02)**				231.00	**Total (1:03)**			1500

A= Materials 2913.18 Birr/m^3

B=Manpower Unit Cost 19.25 Br./m^3
Total of (1:02)
Hourly Output 0.75

C= Equipment Unit C 125.00 Br./m^3
Total of (1:03) 46.85
Hourly output: 0.75

Direct Cost of work item = A+B+C = 3057.43 Birr/m^3
Overhead Cost: 5% 152.87 "
Profit Cost: 15% 458.61 "
Total Unit Cost : 3668.92 Birr/m^3

Remark ____

UF: UTILIZATION FACTOR
*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.
**: INCLUSIVE OF BENEFITS, TRAVEL SUBSIDES AND COST OF OVERTIME RELATED TO TARGETED OUTPUT.
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