



STABILITY ANALYSIS OF MIDDLE AWASH MULTIPURPOSE DAM BY
INTRODUCING INCLINED CLAY CORE

MASTER OF SCIENCE THESIS
SPECIALIZED IN DAM ENGINEERING

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MAY, 2017



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

IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE IN DAM ENGINEERING

MAY, 2017

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

ADVISORS APPROVAL SHEET

This is to certify that the thesis entitled “Stability Analysis of Middle Awash Multipurpose Dam by Introducing Inclined Clay Core” submitted in partial fulfillment of the requirements for the degree of **Master’s** with specialization in Dam Engineering, the graduate program of the **School of Water Resource Engineering**, and has been carried out by Melaku Sitotaw Eshete Id. No PGDENG/011/08, 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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We, the undersigned, members of the Board of Examiners of the final open defense by **Melaku Sitotaw Eshete** have read and evaluated his/her thesis entitled "**Stability Analysis of Middle Awash Multipurpose Dam by Introducing Inclined Clay Core**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of Master's of **science** Specialization in **Dam engineering**.

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Acknowledgement

First and foremost, I praise GOD for being with me all the time in my life.

I thank my advisor Dr. Moltot Zewdie for his close supervision, timely guidance and limitless effort, valuable and constructive comments through the stages of the study. Without his guidance and persistent help this thesis would not have been possible. I would like also to thank my co advisor Dr. Shimeles A. for his critical comments and advice he gave me during my research.

I gratefully acknowledge my best friends namely: Ashenafi Y. (senior geotechnical engineer), Dawit F. (senior engineering geologist), Mekonnen T. (senior engineering geologist), Muluken K. (senior water supply engineer), Tewodros T. (hydraulic engineer), Yared M. (senior hydraulic engineer) and many others for their unconditional help, cooperation and appreciations they gave me since the beginning.

My families my mother, my sisters (Wessu S. and Yehualashet S.) and my brother (Dawit E.) deserve special mention for their enthusiastic support and pray.

Finally, I would like to thank everybody especially staffs of Ribb Dam project who has involved and participates for successful completion of this work.

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Declaration

I, undersigned declare that this thesis is my original work, has not been presented for a degree in any other universities and that all sources of materials used for this thesis have been duly acknowledged.

Kind regards

Melaku Sitotaw Eshete

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List of Acronyms

a.s.l	above sea level
ATH	Acceleration time history
BC	Boundary condition
DBE	Design base earthquake
E	Young modulus
ECDSWC	Ethiopian construction design and supervision works corporation
FEM	Finite element method
FEMA	Federal Emergency Management Agency
FLAC	Fast Lagrangian Analysis of Continua
FS	Factor of safety
G	Dynamic shear modulus
H	Depth
ICOLD	International Commission on Large Dams
LE	Limit equilibrium
LEM	Limit equilibrium method
LS	Lump sum
M	Magnitude
MCE	Maximum credible earthquake
MDE	Maximum design earthquake
MER	Main Ethiopian rift
MoWIE	Ministry of Water, Irrigation and Electricity
MWL	Maximum water level
NPL	Normal pool level
OBE	Operating base earthquake
PGA	Peak ground acceleration
R	Radius
SEE	Safety evaluation earthquake
SSE	South of South-West
SVWC	Saturated volumetric water content
TP	Test pit
USACE	United States Association of Civil Engineers
USBR	United States Bureau of Reclamation
WWDSE	Water Works Design and Supervision Enterprise

List of nomenclatures and symbols

o	degree
°c	degree centigrade
%	percentage
C'	effective cohesion
e	void ratio
e _{cs}	void ratio at critical state
g	acceleration due to gravity
G	shear modulus
G _{max}	maximum shear modulus
GPa	Giga Pascal
hr	hour
km	kilometer
kPa	kilopascal
m	meter
m ³	cubic meter
m/sec	meter per second
mm	millimeter
Na	Afar series
Nn	Nathreth series
Nb	Bofa series
Qd	Dino formation
S	Shear strength
u	pore water pressure
ru	pore water pressure ratio
γ _w	unit weight
Ø'	effective internal angle of friction
σ' _m	mean effective stress
τ	available shear strength

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Abstract

Earth rock fill dam with inclined core geometry is well exercised practice in most countries and this type of core arrangement has its own advantage if implemented on the right site condition, however this experience has not been exercised in our country so far. This thesis will present stability analysis of earth rock fill dam with an inclined core geometry which is proposed as an alternative design option for middle Awash dam project. The dam was designed by Ethiopian Construction Design and Supervision Works Corporation in 2016 as a rock fill dam with vertical clay core. The core geometry has been changed from vertical to inclined arrangement in this study and all analysis has been conducted and compared with the original design. The new dam geometry was fixed and checked for static loading condition and same cross section has been used for dynamic analysis. The analysis has been carried out with numerical modeling computer based software called GEO-SLOPE 2007. Based on computation, the flux through the dam has been found to be $0.0057 \text{ m}^3/\text{sec}$. A factor of safety of 1.72 for downstream slope during steady state, 1.464 during sudden draw down and 1.225 of downstream slope during earthquake are among the results found. The permanent deformation of the dam after earthquake, 0.803 m is also within the allowable limit. The analysis result shows that the water tightness of the dam, stability of the embankment slope and deformation of the embankment with and without earthquake has never been compromised and even the alternative design proposed in this thesis has performed in a better way for some loading conditions like downstream slope at steady state, construction stage and during dynamic loading. In addition, the revised section found to be significantly economical as compare to the original design (vertical clay core).

Key words: - *inclined clay core, numerical modeling, limit equilibrium method, finite element method,*

1 INTRODUCTION

1.1 Background of the Study

Dams which are constructed of earth and rock fill material are generally referred to as filled embankment dams. The history of construction of these embankment dams is much elder than that of other dam types. About 63 countries of the world are now associated with, dams with the height greater than 15 m, which is referred as a high dam (ICOLD, 2005). Among these dams constructed all over the world, more than 70% are of embankments dams. Constructing such dams is highly important for developing countries like Ethiopia, which does not utilize its water resource potential to the required level.

Recently, design and construction of dams is acquiring more attention around the world due to their friendly behavior to environment to use as source of sustainable and renewable source of energy. Rock fill dams are types of embankment dams where rock materials are used as the main fill material in combination with other earth materials. The design and construction of such dams becomes common practice in our county now a days however the art of construction and designing of these dams is still challenging for the industry.

During the early stages of planning and designing, selection of dam site and type of dam which is supposed to be constructed should be considered carefully. It is only in exceptional cases that only a single type of dam is suitable for a given dam site. Thus, except in cases where the selection of type is obvious, it is a routine procedure that all the possible kind of dam should be considered as an alternative and technical so that economical comparisons should be analyzed to select the type of dam which fits best.

It is therefore important to emphasize that; the project is likely to be expensive unless best decisions regarding to the selection of dam type is not made. The selection of the best type of dam for a particular site calls for thorough consideration of the characteristics of each type, as related to physical features of the site and the adaptation to the purposes the dam is supposed to serve, as well as safety, economy, and other pertinent limitations. And the final choice of dam type should be made after consideration of all these factors (Emiroglu, 2008).

Middle Awash Multipurpose Dam (Afar Region, Ethiopia) is one of the biggest embankment dams designed to be rock fill with central clay core supposed to be constructed in earliest possible time. During designing of this dam, of course different dam options has been

considered and evaluated before the selection of rock fill dam with central clay core as the best choice (ECDSWC, 2016).

Almost all embankment dams which are designed and constructed in our country have default vertical core geometry regardless of the specific site condition and this has been found as a gap to be assessed. So, this research is expected to contribute a problem solving alternative design through proposing a better alternative which may not have experienced in our country that could as well introduce designers to consider different core geometry while designing.

1.2 Statement of the Problem

Although the construction of embankment dams in our country has increased since the last two decades, the design and construction art are still a major challenge for the sector. The designs of most embankment dams in our country use similar types of core geometry arrangement (vertical clay core) regardless of the actual site condition, construction material availability, seismic condition of the site etc... and such practice leads projects to unnecessary cost, extended construction period, demands huge technology for foundation treatment and so on.

As to the ECDSWC, (2016) detail design report of middle Awash multipurpose dam, the dam core is designed to be placed on fault breccia which is unacceptable to use it as core foundation unless extensive foundation treatment (jet grouting) has been conducted. Besides the location of borrow area identified for the impervious zone is a little bit far from the dam site (> 15 km) which is relatively uneconomical hauling distance as compare to the location of other fill material. Hence, the need for extensive foundation treatment of the fault breccia with jet grouting and the shortage of clay material in reasonable hauling distance expected to result a huge extra cost to the project.

Despite many dam type options has been considered and analyzed as a possible alternative during preliminary design phase, rock fill dam with an inclined clay core was not considered and this has been identified as a gap to be review. Use rock fill dam with an inclined core geometry looks an appropriate solution to avoid the aforementioned problems of unnecessary cost of jet grouting on the fault breccia, fears of core material migration and leaching of infilling of the fault breccia. Moreover, provision of inclined core will be more reliable than applying of jet grouting as there will be uncertainty on the effectiveness of this new grouting technology.

Thus, this study will present the analysis of middle Awash multipurpose rock fill dam by introducing inclined clay core as a better alternative for the project specific and also introduces the geometry of inclined core geometry for our country.

1.3 Objective of the Study

1.3.1 General objective

The general objective of this study is to asses and provides an appropriate solution for the problem associated with fault breccia foundation and shortage of impervious core material in middle Awash multipurpose dam and also introduces the application of inclined clay core arrangement.

1.3.2 Specific objectives

The specific objectives of this particular study are:

- to compare inclined core against vertical core with regards to stability and seepage control
- to compare the static stability of inclined and vertical core
- to compare the static deformation of inclined and vertical core
- to compare dynamic stability analysis of the two core geometries
- to compare over all construction project cost

1.3.3 Scope of the study

This study has been intended to address the application of inclined core geometry especially for the area where the dam foundation at the dam center is relatively uncertain regards to seepage control and bearing capacity. This study will cover the detail stability of embankment design including dam dimensioning and zoning, estimation and evaluation of the seepage through the embankment and the foundation, static stability analysis during different loading condition, dynamic stability analysis and comparisons of vertical core with inclined core geometry.

1.4 Significance of the Study

Design of embankment dams should comprise of all required designing arts considering geological condition of the site, construction material availability, construction difficulty, and economy of the project and a good design is the one which optimize these entire criterions.

As mention earlier, dam designing practice in our county follows a default type of design and almost all embankment dams have identical core geometries no matter what the actual site condition looks alike and such practice leads the projects to unnecessary extra cost, extended construction period, demand new treatment technologies etc.

Therefore, this study will have considerable significance to

- introduce dam designers to consider inclined core geometry as alternative option
- provide a reliable and economical mitigation measure for the problems associated to the specific site (i.e. to middle Awash multipurpose dam)

1.5 Research Question

The main questions of this study are:

- Will rock fill dam with inclined core geometry have economic advantages for the case of Middle Awash multipurpose dam?
- Does the introduction of inclined core compromise the stability of the dam?
- Does an inclined core is a better alternative over vertical clay core under given conditions?

2 LITERATURE REVIEW

2.1 Dam Type Selection

Selecting the best type of dam for a particular site calls for thorough consideration of the characteristics of each type, as related to physical features of the site and the adaptation to the purposes the dam supposed to serve (Emiroglu, 2008). In addition, technical justification and overall cost of the project plays significant role in selection of dam type (Tancev, 2005). In the condition where two or more types of dams are technically feasible, a cost comparison should be conducted to select the least expensive. It has to be noted that the selection of the best types of dam does not mean voting for the best type against others of lesser quality. All dams designed and constructed according to the state of the art are equivalent in their standard. The task is just to find the most appropriate type of dam as an option of all technical and economic aspects under consideration of actual facts of the site (Kutzner, 1997).

The selection of type of dam for a particular site location depends mainly on availability of construction material, topography, geological condition of the foundation, height of the dam, seismicity of the dam site and other factors. The influence of these major factors in selection of dam type has been discussed below: -

Studies by Emiroglu (2008); Tancev (1982) and Briddle (1988) showed how the presence or absence of construction material like dam fill, cement, aggregate, sand, and filter drainages in the construction area influences the type of the dam to be selected. If earth fill material is not available in the proposed dam site within economical distance a rock fill with artificial sealing could be considered. Besides uniformity of the borrow area also affected a dam type like if two kinds of material can be found in the vicinity, a zoned type of embankment dam would be preferable.

Topography is significant factor too on the selection of dam type as narrow gorges are favorable for gravity or arch dams whereas embankment dams are favorable for wide and plain terrains (Stematiu, 2006). Height of the dam is also an important criterion for the selection of dam type and high earth rock fill dam with artificial and natural sealing has been constructed all over the world (Emiroglu, 2008). Rogun 335m high dam in Tajikistan, Manuel Torres 261m high dam in

Mexico, Tehri 261m high dam in India, Oroville 230 m high dam in USA are among the biggest dam of this type across the world (Tancev, 2005).

Geologic formation of a selected dam site could limit the type of dam to be constructed. Foundations of competent rock with high bearing capacity and resistant to percolation and erosion are required for concrete arch, multiple arch and buttress dams because these structures are sensitive to foundation deformation. However, embankment dams can be built over a wide range of foundation material without any stability and safety problem as far as the required considerations has been made. The geological condition of a site might make a dam site economically unfeasible, poor and complex foundation condition may lead to high construction cost and extended construction schedule for treatment works (ICOLD, 2005). Kesseem and Tendaho dam projects, in Ethiopia are of good example for poor and complex geological conditions which take extended foundation treatment period and huge cost than originally planned.

Seismic condition of the site is dominant factor on the selection of the dam type to be constructed. If the dam site is located in a seismically active zone the most suitable type of dam is the one which can resist the earthquake shock without much damage. The main effects of earthquake on embankment dams is instability of slope, liquefaction hazard, longitudinal transverse cracking, excess deformation and loss of freeboard if proper considerations are not taken (Gazetas, 1987). Embankment (Earth dams and rock fill) dams are generally more preferable than other type of dams due to their flexible nature during dynamic loading (Stematiu, 2006).

2.2 Embankment Dams

Embankment dams are the simplest and oldest type of dam which has been constructed since early time. Generally, large embankment dams have two major components: an impervious water retaining zone and coarse earth fill zone for structural stability purpose (Novak et al., 2007).

Today as that of the past, embankment dams continue to be the most common choice, principally because their construction involves utilization of locally available material with minimum of stockpiling and processing cost (USBR, 2012). According to Novak et al., (2007) embankment dam have many advantages as compare to other types of dams because of their suitability for wide and step valleys, adaptability for wide range of geologic formation, flexibility of

accommodating locally available material, capability of withstanding earthquake load and their low construction cost. Embankment dam is general description which includes earth fill, rock fill as well as combinations of the two. Dams with majority of their volume is rock but uses concrete, asphalt or clay core as impervious zone also included in the category of rock fill dam. The entire range of soils from clays, boulders and excavated rock is used in their construction and an embankment dam can be characterized as a rock fill dam depending on the volume of rock material used in the construction (Stematiu, 2006).

2.3 Foundation Treatment

Embankment dams with impervious clay core induce lower stress on the foundation comparatively to other ridged type of dams. However, the foundation of embankment dams has to be treated to increase the bearing capacity, to minimize seepage loss and deformation. Poor foundation conditions including geological discontinuities, faults, bedding planes and fractures could affect the stability of any dam. Foundation treatment of a dam foundation should always account for the loading conditions imposed by the dam, the reservoir and loads due to earthquakes. Foundation treatment includes excavation, surface treatment, sealing measure, provision of drainage and and strengthening and consolidation mechanisms (ICOLD, 2005).

Grouting is one the main foundation treatment mechanism to improve the foundation condition with respect to seepage control and bearing capacity. It is a process of drilling a lines of holes in to the foundation and injects cement slurry or chemicals grouts with recommended pressure in to the cracks, joints and fractures of ground formation. Conventional type of grouting are of two types: consolidation grouting and curtain grouting (Fell et al., 2005).

Consolidation grouting :- consolidation grouting is to strengthen the dam foundation so as able to reduce embankment deformation that might occur due to imposed load on the foundation. These type of grouting is mainly done for the top 10 m to 20 m Weaver and Bruce, (2007) depending on type of the dam as well as foundation condition.

Curtain grouting:- is a deep seated treatment employed to prevent water loss through the foundation of the dam by sealing up the pores in the foundation and cut down uplift pressure. This treatment forms main line of defence of leakage protection. The holes may have single or

multiple rows and the depth may extend to $0.75 \cdot H$ depending on the geology of the dam site and the type of dam supposed to be constructed (ICOLD, 2005).

The above two kinds of foundation treatments are well exercised in our country that the foundation of major embankment dams foundation across the country (Tendaho, Kesem, Ribb and Megech) has been treated in these techniques WWDSE (2008) and WWDSE (2010). But depending upon the complexity of the geology the above mentioned types of grouting may not be adequate due to the complexity or unique nature of the local geology and in such cases a special type of foundation treatment called jet grouting may be recommended.

According to Xanthakos et al., (1994) jet grouting was primarily developed in Japan and introduced to rest of the world in the mid 1980's. Presently jet grouting is a well exercised method all over the world as dam foundation treatment in weak rocks, karstic formation and alluvial soil Bruce (1994). Jet grouting is different from a conventional type of grouting that it is a mix of in place process where the ground is disturbed and remixed in-situ by dynamic actions of high speed jet liquid to make a series of interconnected soil cemented columns. The main challenge with jet grouting is the continuation of the column structures constructed and the geometry of jet grouting depends on radius of action, pressure applied, and geologic formation. In application of jet grouting there is uncertainty regarding the relative position and properties of treated foundation ICOLD (2005) as the construction takes place inside ground. This technique has never been exercised as in our country so far and the nature of demanding expensive cost as compare to other conventional type of treatment methods make it least preferable.

2.4 Alternative Dam Types

The presence of ample source of rock fill material, homogeneous characteristics of the quarries identified in the vicinity of the dam site, excavation of spillway and dam foundation makes earth rock fill dam the best alternative for Middle Awash multipurpose dam (ECDSWC, 2016). The choice between natural and artificial sealing is selected mainly in respect of the availability of material and the cost of its processing, hauling, dumping and compacting. Basically, natural or artificial sealing mechanisms are implemented in modern dam construction and dams with artificial sealing demand higher technical standards than dams with natural sealing.

Natural sealing mechanism has been widely used in construction of most embankment dams. Kesem, Tendaho, Ribb and Megech dam projects are well known embankment dams with natural sealing. But there are still few embankment dams with artificial sealing technique like Gilgel Gibe I, wolkaiyte and Genale Dawa dam projects (MoWIE, 2009). For dam sites like Middle Awash multipurpose, where it is found in remote area of no infrastructure to produce artificial sealing, natural sealing mechanism is more economical and preferable over artificial type of sealing mechanism. Embankment dams with a natural sealing has mainly two clay core geometrical arrangements: vertical or sloping core arrangements (Kutzner, 1997).

2.5 Clay Core Geometry

Impervious clay zone of an embankment dam has low shear strength than the shell zone so that the location, width and inclination of this zone have significant effect on stability, seepage control and deformation characteristics of the embankment (Khanna et al., 2014; Farzampour et al., 2014).

Clay core zone in embankment dams may have either vertical or inclined geometry depending up on the site condition. Vertical core is arranged to more or less symmetrical along the dam axis. This type of core arrangement in designing and construction of embankment dam is common that many dams with such core arrangement have been constructed across the world. In vertical clay core configuration, the core zone has relatively small effect on the stability of embankment slope as this zone is located deep inside the dam. Besides, seepage at the contact of the core and the foundation is expected to be less due to the nature of existing high contact pressure at this location. Far-Nurek Tajikistan, 300 m high), El-Infefiernillo (Mexico, 148 m high) and Gapatch (Australia 153 m) are among the highest dams constructed with vertical clay core arrangements (Tancev, 2005). This core geometry is common and widely used in our country that almost all embankment dams constructed recently all over the country have this type of core arrangement (WWDSE 2005; WWDSE 2008; ECDSWC 2016).

Depending on site specific and particular advantages have, core geometry might be designed to be sloping. Inclined core arrangement has particular advantage over vertical core regarding to shear stress concentration, plastic yield point of view and better resistant against hydraulic fracturing during sudden reservoir fill up (Nayebzadeh and Mohammadi, 2011). Embankment

dams with an inclined clay core geometry is common and widely used on other countries and dams with such geometrical arrangement have an advantage that smaller part of the dam body will be under water, majority downstream shell can be constructed as a homogeneous, foundation grouting can be performed parallel to the construction of downstream shell and has better performance or resistance to earth quake loading and so on (Tancev, 2005).

Tensay et al., (2013) and Kunitomo (2000) studied that embankment dams with inclined core are preferable for the conditions when the dam foundation has steep inclination along the river, there is relatively impervious and strong foundation upstream of the dam axis, different construction processes are required for the placement of core or shell material and the dam is being constructed in highly seismic zone. Orville dam (235 m high in USA) and Salto Santiago (80 m high in Brazil) are among the largest earth-rock fill embankment dams with an inclined core arrangement (Jansen, 1988 and Tancev, 2005). But except the coffer dam of Tendaho dam project WWDSE, (2007) no major embankment dam has been constructed in our country with such core geometry.

Many researchers have studied different issues associated with inclined core geometry and introduced the specific characteristics and their advantages. Tensay et al., (2013) has investigated numerically the performance of clay core arrangement on earthen dams. Typical embankment dam cross section has been used for investigation and used four different core arrangements (vertical clay core, 20° inclined to upstream, 40° inclined to upstream and clay blanket on the upstream slope face) for analysis purpose. A numerical model called FLAC 3D has been used for analysis and the influence of clay core geometry against settlement and development of pore water pressure has been investigated. Based on this particular study, the researchers concluded that the performance of earth dam increases with an increment of core slope towards the upstream and they recommend that clay core geometry has to be considered as major criteria during designing of embankment dams.

Nayebzadeh and Mohammadi, (2011) has studied also the effect of clay core shape on stability of embankment dam. These researchers undergo their study on Ghavoshan rock fill dam, found in western Iran. For the comparison purpose the same dam with vertical clay core, moderately sloping clay core and sloping clay core has been considered and a numerical model of SIGMA/W has been used for the analysis. Deformation, stress distribution and resistant to

hydraulic fracturing have been considered as comparative parameters. Based on the investigation, the researchers found that the dam with sloping core has better advantage and performances regarding to embankment deformation and resistance to hydraulic fracturing.

Apart from this finite element method has been applied by Zhang (2012) to analyze Hongxing earth dam in Taiwan, which has an inclined clay core. Stress distribution and dam deformation was the major parameters studied in this research and inclined core embankment has better performance regarding deformation. Besides the economic benefits of inclined core has been studied and he concluded that inclined clay core in earth dam of Hongxing dam is economical, the structure is safe, reliable and every index meets to the requirements of the design.

Querol and Moreta (2008) studied the performance of heterogeneous earth fill dams under earthquakes and the optimal location of the impervious core. Investigation was conducted on typical heterogeneous fill dam and a finite element numerical code has been used for analysis. Liquefaction potential, soil degradation and crest settlement was the main parameters investigated and based on the study the dam with higher core slope has better performance and the researcher recommended the location of core material should be considered as design criteria.

Thus, all the above studies conducted on the geometry and arrangement of clay core reinforces the idea that Middle Awash multi-purpose dam should be investigated by changing the geometry of the impervious zone. In addition, no major dam with such core arrangement has been designed or constructed in our country so far thus designers should consider this matter as a criterion during dam designing.

2.6 Embankment Dam Design

The basic objective of embankment dam design is to produce a satisfactory functional structure at a minimum total cost. To achieve minimum cost, the dam should be designed for maximum use of the most economical material available on the vicinity, including materials which might be excavated from the dam foundation and appurtenant structures (ICOLD, 1988 and USBR, 2012). According to ICOLD, (1988) and USBR, (2012) the main criterion of embankment design comprises:

- The embankment, the foundation, the abutments and reservoir rims should be stable under all static and dynamic loading conditions

- Seepage through the embankment and dam foundation should be controlled to prevent piping, instability, sloughing and material migration through rock joints and fractures
- Slope of the embankment should be stable during different loading conditions
- There should be sufficient freeboard for prevention of overtopping
- The embankment should incorporate camber for settlement allowance for the embankment and dam foundation
- Spillways and outlet structures should have sufficient capacity to prevent dam overtopping

2.6.1 Dam zoning

Large embankment dams should be zoned to use as much material as possible from required excavation and borrow areas with the shortest haul distances and the least waste. Embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells seepage control, and stability. The slopes of an embankment dam may vary widely depending on the characteristics of the materials available for construction, foundation conditions, and the height of the dam as well (USBR, 2012). According to Novak et al., (2001) zoned embankment dams led to more economical structures where there are a variety of soils readily available. Major advantages of using zoned embankments are:

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

2.6.1.1 Upstream and downstream embankment slopes

The slope of embankment dams roughly corresponds to the angle of repose of loose dumped rock and prevents raveling of the embankment slope. In recent practice a rock embankment of 1.3H:1.0V to 2.0H:1.0V is common. However, when gravel weak rock is used as an embankment material, flatter slopes are recommended. The upstream and downstream slopes of the dam depend also on the type and location of impervious zone. Rock fill dams having central or sloping earth fill cores usually have slopes of about 1.5H:1V to 2H:1V upstream and downstream, often depending on the location of the core. The upstream slope is generally flatter, particularly for upstream sloping cores (USBR, 2012).

2.6.1.2 Impervious clay core

According to USACE, (2004) the core width for an embankment should be established using seepage and piping considerations, types of material available for the core and shells, the filter design, and seismic considerations. In general, the width of the core at the base should not be less than 25 percent of the total hydraulic head. A core top width of 3 m is considered to be the minimum for construction equipment and the maximum core width will usually be controlled by stability and availability of impervious materials. For embankment dam of inclined clay core Kutzner, (1997) suggest that the upstream core slope can be reasonably flattened as 1V:1.4H and downstream slope 1V:0.5H as far as the embankment is stable for all loading condition.

2.6.1.3 Filter and transition zones

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

2.7 Dam Zoning Experience in Ethiopia

Construction of embankment dam in Ethiopia has become a common practice and many embankment dams are under construction across the country. Tendaho dam (55 m high) and Gidabo dam (24 m high) are of earthen embankment and Ribb dam (73.5 m high), Megech dam (> 91 m high), Kesem (91 m) and Welkayite dam are some of the rock fill embankment types (WWDSE, 2005; WWDSE, 2007 and ECDSWC, 2016). Experience of upstream and downstream embankment slopes of some of the rock fill dams designed and constructed in our country are considered as reference and presented in table 2.1 below:

Table 2.1: Embankment slopes experience for rock fill dams in Ethiopia

Name of the project	Upstream slope	Downstream slope	Remark
Ribb Rock Fill Dam	1V:2H	1V:1.75H - 1V:2H	Under construction
Megech Rock Fill Dam	1V:2H	1V:1.75H - 1V:2H	Under construction
Wolkayt Asphalt Core Rockfill Dam	1V:1.6H - 1V:2H	1V:1.7H	Under construction
Kesem Gravel Rock Fill	1V:2.25H	1V:2H	Completed
Jema Rock Fill Dam	1V:1.75H	1V:1.5H	Design completed

Similarly, the core geometry used in many projects across the country has been summarized below and as stated earlier most of them have similar clay core orientation regardless of the site condition. Table 2.2 presents the geometry and orientation of the clay core for some major projects in Ethiopia, which are under construction, design and completed recently.

Table 2.2: Core geometry and orientation experience for dams in Ethiopia

Name of the project	Core geometry	Slope		Remark
		upstream	downstream	
Ribb rock fill dam	vertical	1V:0.25H	1V:0.25H	Under construction
Megech rock fill dam	vertical	1V:0.5H	1V:0.5H	Under construction
Kesem gravel rock fill	vertical	1V:0.25H	1V:0.25H	Completed
Gilgel Abay rock fill dam	vertical	1V:0.5H	1V:0.5H	Design completed
Tendaho earth fill dam	vertical	1V:0.225H	1V:0.225H	Completed
Tendaho coffer dam	Inclined	1V:1H	1V:0.5H	Completed
Jema Rock fill dam	vertical	1V:0.5H	1V:0.5H	Design completed

2.8 Slope Stability

Slope stability is always a key issue in the analysis, assessment or design of most embankment dams. The prime objective of stability analysis is to ensure that the dam does not collapse or deform to the point that it causes unmanageable property damage and loss of human lives. Slope

stability analysis is common type of numerical analysis in geotechnical engineering and has become routinely used in practice (Khran, 2007). Earth and rock fill dams must sustain very different loading conditions that arise during construction and subsequent operations. Soil mechanics has developed methods to evaluate adequately safety against failure for the conditions during and immediately after construction, steady seepage, rapid drawdown and first filling of the reservoir Stematu, (2006) and the main challenges in slope stability analysis is determining the factor of safety (Nourani, 2012).

The principal methods for determination of this safety factors are the limit equilibrium and finite element methods. The limit equilibrium method is restrictive that it can only be used to determine a safety factor against sliding. However, the finite element method is more versatile and used for predicting deformation, stresses and time dependent consolidation analysis (SLOPE/W, 2008).

2.8.1 Limit equilibrium method

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

Different researchers have studied about LEM have been reached to the conclusion that LEM method has almost to that of other methods like finite element or finite difference methods (Azadmanesh and Arafati, 2012).

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

(SLOPE/W, 2008). Method available in limit equilibrium stability analysis, equations of statics satisfied, summary of the inter-slice forces included are presented in table 2.3 below.

Table 2.3: Features and limitation for limit equilibrium methods (SLOPE/W, 2008)

Method	Features and Limitation
Slope Stability Charts	<ul style="list-style-type: none"> - Accurate enough for many purposes. - Faster than detailed computer analysis.
Ordinary Method of	<ul style="list-style-type: none"> - Only for circular slip surfaces. - Satisfies moment equilibrium. - Does not satisfy horizontal or vertical force equilibrium.
Bishop's Modified Method	<ul style="list-style-type: none"> - Only for circular slip surfaces. - Satisfies moment equilibrium. - Satisfies vertical force equilibrium. - Does not satisfy horizontal force equilibrium.
Janbu's Generalized Procedure of Slices	<ul style="list-style-type: none"> - Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force locations to be varied. - More frequent numerical problems than some other methods.
Morgenstern and Price's Method	<ul style="list-style-type: none"> - Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force orientations to be varied.
Spencer's Method	<ul style="list-style-type: none"> - Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Side forces are assumed to be parallel.

2.8.2 Finite element method

With the development of modern personal computers, finite element method has been increasingly applied in slope stability analysis. The finite element method (FEM) is powerful alternative approach the analysis of slope stability. Zaman et al., (2000) studied that FEM is accurate, versatile and does not demand many assumption and it is also powerful in solving problems with irregular bodies and complex variation of flow lines. The advantage of finite element over limit equilibrium method is that no assumption needed in advance about the shape and location of the failure surface and this method gives information about the deformation at

working stress level Griffiths and Lane, (1999); Chollada and Tanan, (2013). In application of finite element methods stress-strain relationship will be considered in addition to force and moment equilibrium conditions and this will lead to much realistic result (SLOPE/W, 2008).

2.9 Loading Condition for Static Stability Analysis

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

2.9.1 End and during construction

End of construction condition of slope stability analysis is very important especially for non-free drain soils as there will be development of pore water pressure between the soil particles. Computations of stability during construction and at the end of construction are performed using drained strengths in free-draining materials and undrained strengths in materials that drain slowly. The stability of embankment slope during construction until end could be critical problem in the construction area where the weather condition is wet. This loading condition is critical for both upstream and downstream slopes USACE, (2003); Stematiu, (2006).

2.9.2 Steady state seepage conditions

Long term stability computations are performed for conditions that will exist for sufficient length of time after construction for steady state seepage. Stability computations are performed using shear strengths expressed in terms of effective stresses and with pore pressures appropriate for the long-term condition. The downstream slope of an embankment dam is more critical for this loading condition (USACE, 2003).

2.9.3 Sudden drawdown condition

Sudden drawdown condition is a classical scenario in slope stability, which arises when totally or partially submerged slopes experience a reduction of the external water level. Rapid drawdown

conditions have been extensively analyzed in the field of dam engineering because reservoir water levels fluctuate widely due to operational reasons or emergency condition.

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

2.10 Dynamic Stability Analysis

Earthquakes significantly affect the stability embankment of slopes. Dams in seismic region need to be evaluated to make sure that they can withstand earthquakes protecting public safety, life and property. 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).

To analyze the dynamic stability analysis of embankment dams horizontal and vertical acceleration-time histories (ATH) are key parameters. Therefore, site specific horizontal and vertical ATH for project area should be produced using the peak accelerations and records of actual earthquakes data. In the condition where there are no ATH records near the dam site, actual accelerographs recorded elsewhere have been used. Messle, (1996) suggests that in the area where there is no recorded ATH data, the following three ATH data could be considered for the analysis:

- The 1940 El Centro 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. In addition to this, during their study on the effect of canyon geometry and ground conditions on the seismic performance on Tendaho dam project Tenasy and wei, (2010) used El Centro (USA) acceleration time history as there is no any record of acceleration time history in / or around the area.

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, liquefaction, and differential settlement and so on. Therefore, the following key issues have to be analyzed for dams constructed in seismic zones:

2.10.1 Liquefaction

The evaluation of soil liquefaction phenomena and related ground failures associated with earthquake are the important aspects in geotechnical engineering practice. Liquefaction is the change of granular material from solid state to liquefied state due to the increase of pore water pressure until the effective stress reaches to zero (Marto and Choy, 2012).

All soils are not susceptible for liquefaction so that the first procedure in liquefaction assessment is that to determine the location where liquefaction may potentially happen (Kramer, 1996). According to Kramer if the soil at the particular site is not susceptible liquefaction hazard do not exist so that no need to evaluate the liquefaction hazard. According to Kramer, (1996), there are different methods to evaluate liquefaction susceptibility and some of them are presented below:

- **Historical criteria**

Post-earth quake investigation has shown that liquefaction recurs at the same location when the soil and ground water conditions are remaining same (Youd, 1984). Thus, case histories of previous earthquakes can be used to determine whether the site is susceptible for liquefaction or not and this may be applied more for general site condition.

- **Geologic criteria**

Geologic processes that sorts soils in to uniform grain size distribution and deposit them in loose state with produce soils which are susceptible for liquefaction. Depositional environments, hydrological condition and age of soil deposit affect the susceptibility for liquefaction (Youd and Hoose, 1977). Fluvial deposit, colluvial deposits and aeolina deposits are the kind of deposits which are susceptible to liquefy when they are saturated. Older alluvial soil deposited deposits are less susceptible for liquefaction as soil deposit consolidates with time (Kramer, 1996). Soil deposits made by humans requires great attention so that embankment fills like filter materials in side dam body should be compacted to the minimum requirement of relative density greater than 75% to avoid liquefaction.

- **Compositional criteria**

As liquefaction happens due to the development of excess pore pressure, it is therefore affected by the compositional characteristics that affect volume change of the soil mass. Compositional characteristics associated with high volume change are associates with high liquefaction susceptibility. This behavior has been studied extensively by researchers all around the world and liquefaction susceptibility identification studied by Wang (1979) is best known. However, in his study Wang did not included observation data for low plastic soil. Seed and Idriss, (1982) re-evaluates and conclude that soil which is susceptible for liquefaction must full fill the following criterion:

- The soil should contain less than 15% clay (finer than 0.005)
- Liquid limit should be less than 35%
- And water content should be higher than 90%

Later Finn, (1991) also modified this criterion suggested by Seed and Idriss, (1982) and he recommends the soil is susceptible for liquefaction in the condition that

- Percentage of finer is less than 10%
- Liquid limit should be less than 36%
- And water content should be higher than 92%

Graduation of soil particle and particle shape also affects the susceptibility of soil liquefaction. According to Kramer, (1996) well graded soils are generally less susceptible to liquefaction than poorly graded soil and soils having rounded particle shape densify easily than angular soils so that soil mass with rounded shape are susceptible for liquefaction as they have a tendency of considerable volume change with time.

- **State criteria**

Initial state of the soil (stress and density characteristics at the time of earthquake) is important criteria for the occurrence of liquefaction. Although a soil mass which has fulfilled all the above criteria may not lead to liquefaction as the state of condition is favorable. Been and Jefferies, (1985) has introduced a measure of the state of the soil. It is the difference between the current void ratio (e) and the void ratio at critical state (e_{cs}). The state parameter gives a description of the combined effects of the relative density and the confining stress. The critical state refers to the conditions that exist in the soil when it is sheared continuously and no further change in stress and volume is occurring (Idriss and Boulanger, 2008).

2.10.2 Post-earthquake slope stability

Following an earthquake, the stability of slope may be diminished because cyclic loading has reduced the shear strength of the soil. The reduction in shear strength is generally treated differently depending on whether or not liquefaction occurs. Earthquake motions can induce significant horizontal and vertical dynamic stresses in slopes. These stresses produce dynamic normal and shear stresses along potential failure surfaces within a slope. When superimposed upon previously existing static shear stresses, the dynamic shear stresses may exceed the available shear strength of the soil and produce inertial instability of the slope.

2.10.3 Permanent deformation

In dynamic analysis it is not sufficient to determine only the materials are potentially liquefiable or not rather it is important to check analyze the amount of permanent deformation that the dam supposed to experience due to earthquake loading is within tolerable limit or not (William et al., 2007).

3 MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location

Middle Awash Multi-Purpose Dam is located between Afar and Oromia national regional states, around Awash town, at about 220 km northeast of Addis Ababa on main Awash river about 10 km upstream of the Addis Ababa-Djibouti road bridge. The main asphalt road from Addis Ababa to Djibouti passes close to the dam site so that could be used to approach the proposed dam site (ECDSWC, 2016).

The project is located in Awash River basin with geographical location of 990686 Northing and 629786 Easting at the center of the dam site. The location of the project area is shown in Fig. 3.1 below.

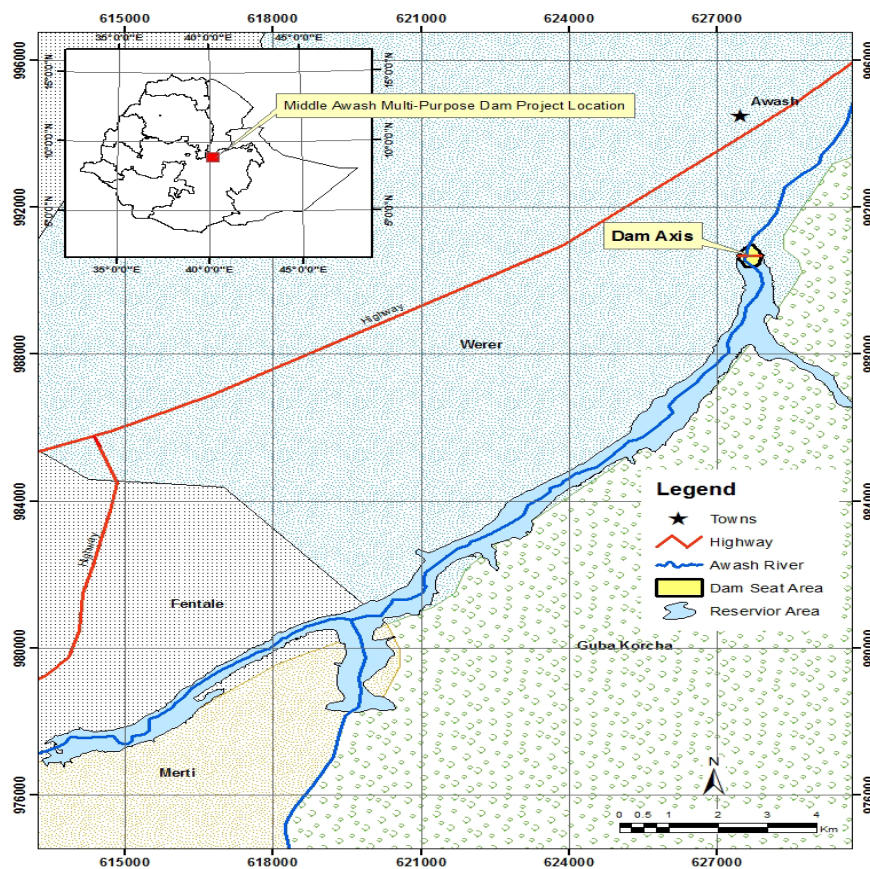


Figure 3.1: Location map of the project area

3.1.2 Climate

The climatic condition of the dam area is characterized as; the mean monthly temperature ranges between of 22.3⁰ C in December and 29.3⁰ C in June. The mean monthly rainfall varies from 9 mm in December to 117.8 mm in July. The mean monthly wind speed varies from 1.5 to 2.4 m/s. Sunshine hour ranges from 7.2 hr in September to 9.4 hr in November. Mean monthly humidity varies from 44% in May and June to 57% in August. The mean monthly evaporation rate varies from 189mm in August to 311 mm in June (ECDSWC, 2016).

3.1.3 Availability of construction material

Borrow for impervious dam core and alluvial deposits (for rock transition) material has identified some 15km further downstream of the dam site on the right side of Awash Rock quarries are identified on the left and right side of Awash River at the vicinity of the dam site, about 2.0 km from the right abutment and 1.5 km from the left abutment (WWDSE, 2015).

3.1.4 Geology

3.1.4.1 Regional Geology

The region is generally underlain by Tertiary Quaternary volcanic rocks and sedimentary rocks of the Nazret series (Nn), Afar series (Na) and Bofa basalts (Nb) and Quaternary volcanic rocks and sedimentary rocks of the Dino Formation (Qd) (ECDSWC, 2016).

The Afar series (Na) comprises a thick succession of stratified flood basalts and intercalated alkaline to peralkaline felsics (rhyolite flows, domes and ignimbrites) covering about 2/3 of the floor of the Afar depression. The basalts are porphyritic, vesicular and rarely display perfect columnar jointing. The Nazret series (Nn) comprises a thick succession of welded ignimbrites with fiamme, pumice, ash and rhyolite flows and domes with rare intercalation of basalt flows which occur in the main Ethiopian rift and adjacent plateaus. The name Bofa basalts was given to all the Pliocene rift floor basalts and these basalts are well developed in the northern and central part of the main Ethiopian rift forming a wedge between the Nazret series (Nn) and Dino Formation. The Bofa basalts are flood basalts mostly aphyric locally vesicular and fresh with several flows separated by scoracious horizons (ECDSWC, 2016).

3.1.4.2 Dam Site Geology

The rocks found in the proposed dam core foundation comprises fresh to slightly weathered jointed rhyolite with zones of fault breccia comprises materials of variable physical and mechanical properties. At the abutments, the core contact area comprises sequences of ignimbrites, tuff and basalt rock units, paleo-soils and volcanic ash layers are also found between the sequences. The lithological units of the right and left abutments are not exactly correlated, the basalt and ignimbrite unit underneath the basalt is not found on the right abutment.

According to the geotechnical investigation report of the site ECDSWC, (2016), the fault breccia comprising materials of variable physical and mechanical properties is located at the deepest section of the dam foundation; this is an extra ordinary material which requires special treatment to accept as dam core foundation material. It is assumed that this material will be qualified to the required physical and mechanical properties by consolidating using high pressure jet grouting.

The flatter slope of the right flank comprises thick older terrace alluvial deposit consisting of dense granular terrace alluvial deposit predominantly coarse gravel and coble sizes with minor fine proportions which extends for about 50 m right edge of the existing river. This terrace deposit has a thickness in the range of 13.0 m to 17.0 m as observed from drilled boreholes. During geotechnical subsurface field investigation eight test pits were excavated in the terrace deposit (right abutment) to investigate the geotechnical properties of the older alluvium foundation and its use as a general foundation of the dam. Most of the test pits located in the proximity of the active river channel was excavated to a refusal depth of about 3m using a 20-ton excavator with a 1.14 m wide bucket. It was not possible to excavate beyond 3 m depth (ECDSWC, 2016).

At the upper part of the right abutment sequences of ignimbrite rock units with intercalations of tuff layers overlying thick light grey Rhyolite rock unit which extends to the valley floor has been observed. According to the core drilling and geophysical investigation the left abutment is mainly composed of rock outcrops and colluviums. The dominant rock types are ignimbrites and basalts often separated by paleosoils and soft tuff layers (ECDSWC, 2016).

3.1.5 Topography

The topographic condition of middle Awash multipurpose dam show that the river bed is as wide as 30 m with average river bed level of around 822 m a.s.l, steep hilly escarpments on both sides. The topography of the dam site is partially shown in the picture below taken at the dam site (figure 3.1).



Figure 3.2: Pictures taken at dam site

3.2 Seismology

The Main Ethiopian rift (MER), which trends SSW from the Afar depression is one of the well-developed continental rift segments in East Africa that marks the boundary between Nubia and Somalia plates. Rifting is evident from topographic expression, geology, volcanism, seismicity and gravity.

The Middle Awash multipurpose dam project is located at the rift axis shown Fig.3.2 ECDSWC, (2016) just on top of the active rift floor. Hypo-central depths of well-constrained events are 5 to 7 km from modeled earthquakes in the main Ethiopian rift, which is the approximate elastic plate thickness in Afar and the main Ethiopian rift, possibly indicating the depth to the brittle–ductile transition zone in this part of the rift. The shallowness of the depth estimates agrees with the macro-seismic reports available from a wide area reported for Hosanna and Yirga Alem earthquakes in south Ethiopia. Potential future shallow crustal deformation may cause significant

loss of human life and damage to property unless measures are taken in improving building standards of houses, roads, bridges and dams (ECDSWC, 2016).

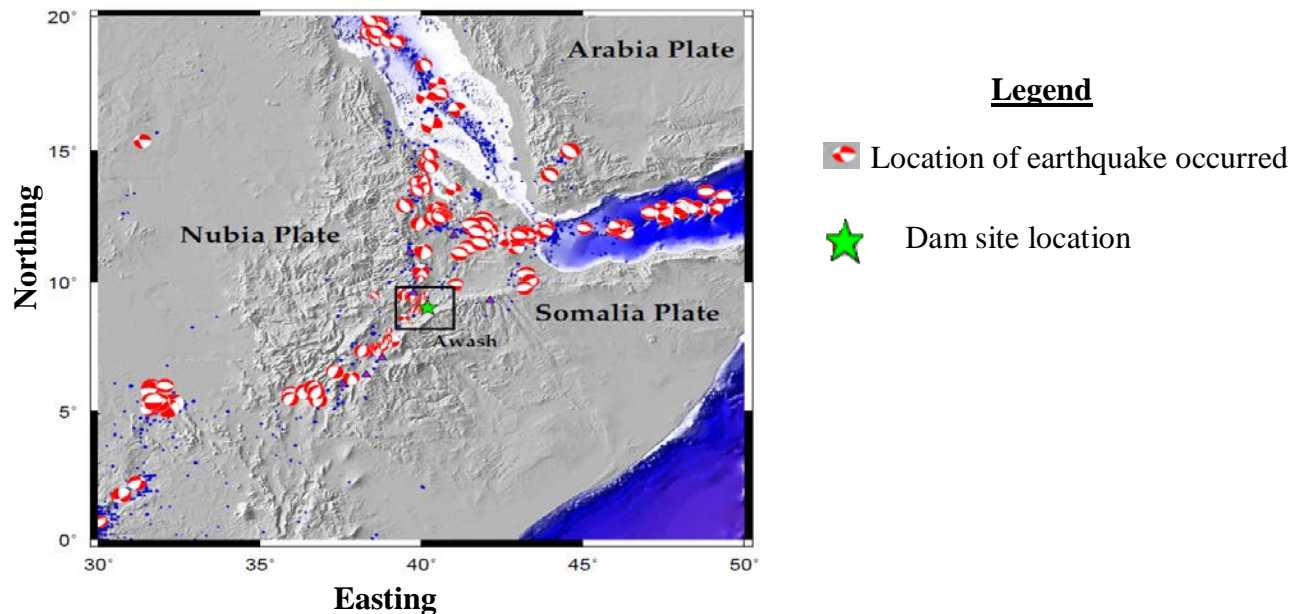


Figure 3.3: Seismicity data for the horn of Africa (ECDSWC, 2016)

3.2.1 Peak ground acceleration

According to FEMA, (2005) earthquakes used for the design or safety evaluation analysis of a dam have been specified by a variety of terms such as the maximum credible earthquake (MCE), the maximum design earthquake (MDE), or the Safety Evaluation Earthquake (SEE).

Maximum Credible Earthquake (MCE): is the largest earthquake magnitude that could occur along a recognized fault or within a particular seismotectonic province or source area under the current tectonic framework data collection

Maximum Design Earthquake (MDE) or Safety Evaluation Earthquake (SEE): is the earthquake that produces the maximum level of ground motion for which a structure is to be designed or evaluated. The MDE or SEE may be set equal to the MCE or to a design earthquake less than the MCE, depending on the circumstances.

Operating Basis Earthquake (OBE): is an earthquake that produces ground motions at the site that can reasonably be expected to occur within the service life of the project. The associated

performance requirement is that the project function with little or no damage, and without interruption of function.

Peak ground acceleration data corresponding to MCE and OBE estimated for middle Awash project area is adopted from the design report ECDSWC, (2016) and the dynamic analysis for revised cross section with an inclined core has been analyzed with this data. According to FEMA, (2005) the maximum vertical value has generally been taken as one-half to two-thirds of the maximum horizontal value and in this thesis vertical acceleration has been considered to be half of horizontal acceleration.

Table 3.1: Peak ground acceleration of project site (ECDSWC, 2016)

Description	Horizontal acceleration	Vertical acceleration
Maximum Credible Earthquake (MCE)	0.3715g	0.186g
Operating Basis Earthquake (OBE)	0.2135g	0.107g

3.3 Material and Parameters Used for Analysis

3.3.1 Primary data

Primarily data which are necessary for the purpose of this study has been collected from the site. These include physical observation of the site condition and gathering of necessary photographs.

3.3.2 Secondary data

For the purpose of geotechnical analysis of embankment dams, engineering properties of each embankment and foundation material are necessary. The geotechnical parameters used in the analysis are compiled from the final design report, geological and geotechnical investigation of middle Awash multipurpose dam and typical values has also been from design manuals and results of other researchers whenever necessary.

3.3.2.1 Parameters used for seepage analysis

- **Coefficient of Permeability**

Coefficient of permeability of the embankment and foundation materials has been adopted from final design report for middle Awash multipurpose dam and compiled as follow (ECDSWC, 2016).

Table 3.2: Permeability coefficients used in the analysis (ECDSWC, 2016)

Description	Permeability (m/s)
Core	4.76×10^{-8}
Fine filter	7.9×10^{-5}
Coarse filter	0.05
Inner shell	0.05
Rock fill	0.05
Old Alluvial	1.0×10^{-5}
Fault Breccia	1.33×10^{-5}
Foundation bed rock	1.0×10^{-9}

- **Volumetric water content (porosity)**

Saturated volumetric water content (SVWC) values for different embankment materials are adopted from different literatures and SEEP/W modeling engineering book sample functions (SEEP/W, 2008)

Table 3.3: Volumetric water content of embankment material (SEEP/W, 2008)

Description	Saturated Volumetric Water Content
Core	0.5
Fine filter	0.35
Coarse filter	0.28
Inner shell	0.27
Rock fill	0.25

3.3.2.2 Parameters used for slope stability analysis

- **Shear Strength Parameters**

For slope stability analysis the shear strength parameters of the dam and embankment materials have been adopted from the final design report of Middle Awash multipurpose dam (ECDSWC, 2016).

Table 3.4: Shear strength parameters used for analysis (ECDSWC, 2016)

Material	Unit weight (kN/m³)	C'(kPa)	φ'(degree)
Clay core	16.0	20	23
Fine filter	20.0	0.0	32
Coarse filter	20.0	0.0	34
Inner shell	20.0	0.0	34
Rock fill	22.0	0.0	42
Old Alluvial	20.0	5.0	34.0
Fault Breccia	17.0	10.0	34.0
Foundation bed rock	Bed rock	Bed rock	Bed rock

During computation of slope stability for stage construction and end of construction a pore-water pressure ratio (r_u) value has been used as factor which relates the overburden stress to pore water pressure. Table 3.5 shows r_u value used for non-free drained material adopted from design report of the project (ECDSWC, 2016).

Table 3.5: Pore water pressure ratio r_u values for construction conditions

Material	Pore-water pressure ratio (r_u) during construction	Pore-water pressure ratio (r_u) at end of construction
Clay core	0.45	0.40
Old Alluvial	0.35	0.3
Fault Breccia	0.35	0.3

3.3.2.3 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 3.5.

Table 3.6: Parameters used for stress deformation calculation (ECDSWC, 2016)

Description	E** Young's modulus in MPa	Poisson ratio (ν)**	C' (kPa)	Unit weight (kN/m ³)	ϕ° (degree)	k	n	Rf	Ko
Rock fill	100	0.3	0	22	42	-	-	-	-
Inner shell	80	0.3	0	20	34	-	-	-	-
Fine Filter	60	0.25	0	20	32	-	-	-	-
Coarse Filter	80	0.15	0	20	32	-	-	-	-
Clay core	-	0.3	20	16	23	150	0.35	0.7	0.6093
Older alluvial	90	0.31	0	19	34	-	-	-	-
Fault Breccia	-	0.35	5	17	28	-	-	-	-
Bedrock	1150	0.25	-	22	42	-	-	-	-

** Bowles et al., (1997)

3.3.2.4 Parameters used for dynamic analysis

The dynamic characteristics of the dam materials have not been investigated by means of dynamic triaxial tests. Therefore, the material properties required for the dynamic analysis was estimated with the help of the geotechnical literature (Messele and Hadush, 2006). Geotechnical parameters required for dynamic analysis has been presented below.

- **Soil stiffness**

The soil stiffness is generally a function of the stress state. As the confining stress increases, the soil stiffness increases. The equation presented below used to describe the relation between soil stiffness and with depth (QUAKE/W, 2008; Messele and Hadush, 2006).

$$G = K_G \times (\sigma'_m)^n \text{----- Equation 3.1}$$

Where G is the shear modulus, K_G is a soil modulus, σ'_m is the mean effective stress and n is a power exponent (generally n is taken as 0.5).

To determine G_{max} and the corresponding soil modulus K_G , the following widely used empirical equation developed by Seed and Idriss, (1970) can be used.

$$G_{\max} = 220 \times K_{2\max} \times (\sigma'_m)^{0.5} \text{-----Equation 3.2}$$

$K_{2\max}$ values for embankment material and dam foundation has been adopted from the design report of middle Awash ECDSWC, (2016) and compiled as follow.

Table 3.7: $K_{2\max}$ values adopted from design report (ECDSWC, 2016)

Function	$K_{2\max}$
Clay core	50
Fine filter	70
Inner shell	80
Rock fill	80
Fault Breccia	70
Old Alluvial	70
Foundation bed rock	E=1.64GPa, $\nu=0.18$ and $G_{\max}= 694915.25\text{kPa}$

- **Shear modulus reduction and damping ratio functions**

A soil subjected to dynamic stresses tends to soften in response to cyclic shear strain. This softening is described as a ratio relative to G_{\max} called a G-reduction function QUAKE/W, (2008). At the same time, the nonlinear response at higher dynamic strains leads to a higher rate of energy dissipation, which is represented by a damping ratio that increases at higher strain levels Messele and Hadush, (2006). The strain-dependent dynamic shear modulus and damping ratio values for the different soils and rock published by Sun et al., (1988); Schnabel et al., (1972), Idriss, (1990) and Seed et al., (1986) have been used taking the values from the following Fig 3.3.

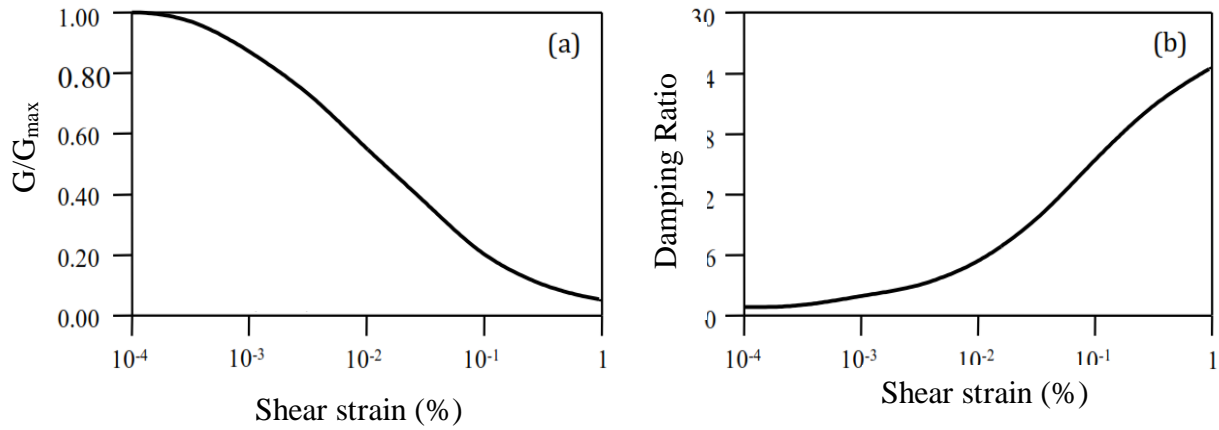


Figure 3.4: Dynamic characteristics of gravel soil (a) shear modulus (b) damping ratio Seed et al., (1986)

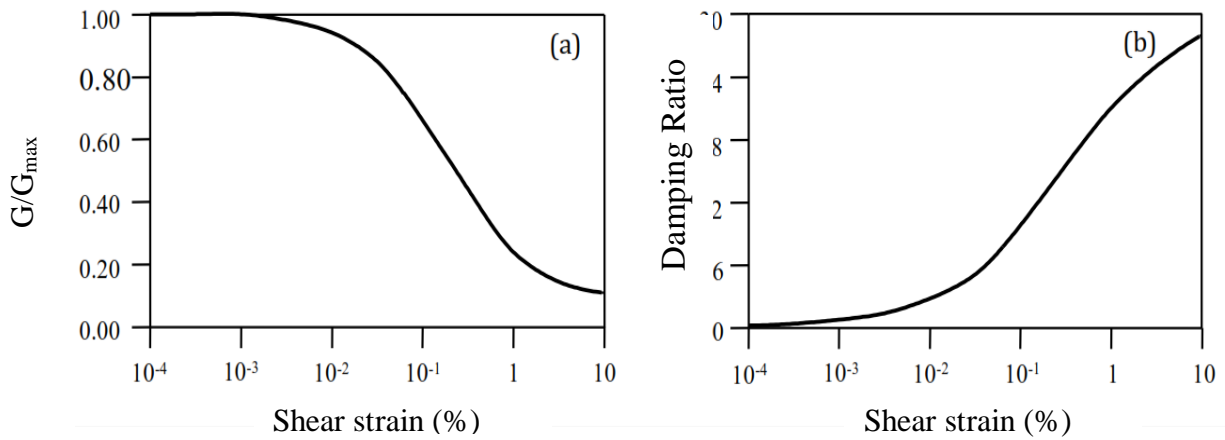


Figure 3.5: Dynamic characteristics of clay (a) shear modulus Sun et al., (1986) (b) damping ratio Idriss et al., (1990)

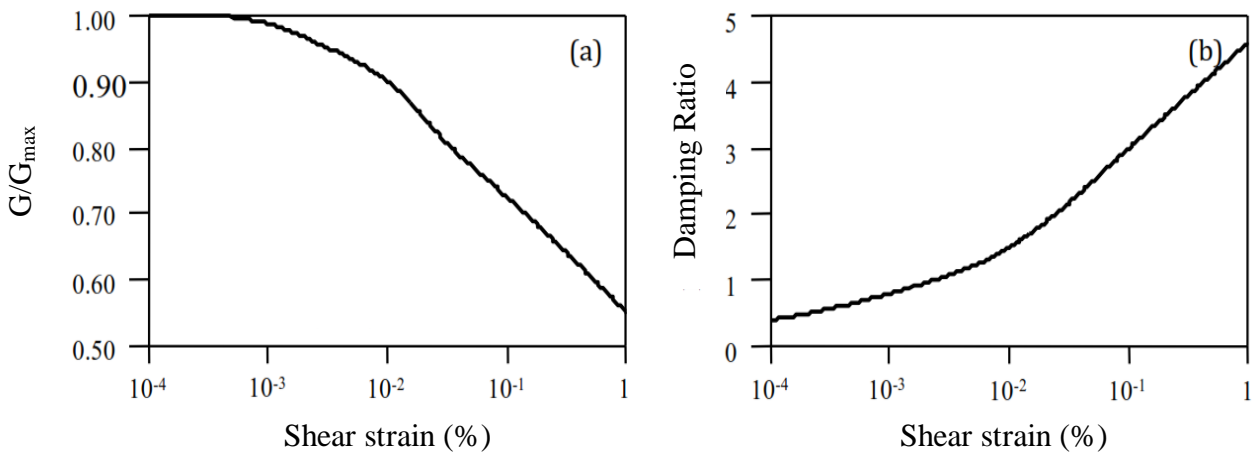


Figure 3.6: Dynamic characteristics of Rock (a) shear modulus (b) damping ratio Schnabel et al., (1972)

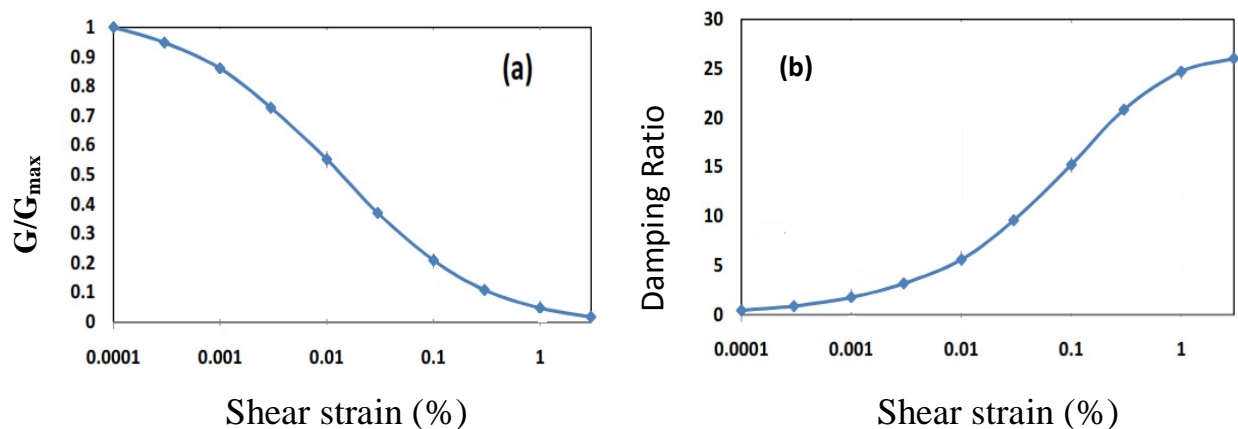


Figure 3.7: Dynamic characteristics of filter and drainage (a) shear modulus (b) damping ratio Seed et al., (1972)

3.3.2.5 Reservoir characteristics used for analysis

Reservoir characteristics of the dam used for analysis have been adopted from the final design of middle Awash multipurpose dam ECDSWC, (2016) as listed below:

Table 3.8: Reservoir characteristics (ECDSWC, 2016)

Description	Elevation(m) a.s.l
Maximum water level	938
Normal pool level	926
Minimum drawdown level	860

3.4 Description of Model Used

Numerical model is a simulation of a real physical process and it is purely mathematical. In this sense, it is very different from scaled physical modeling in the laboratory or full-scaled field modeling. Presently, stability and seepage problems can be accurately modeled analyzed using several geotechnical softwares. GEO-STUDIO is one of these numerical modeling softwares developed by Geo-Studio international based on limit equilibrium and finite element principles used specially for the analysis of seepage, stability and deformation of geotechnical structures. The package includes tools for seepage modeling (SEEP/W), slope stability modeling (SLOPE/W), stress and deformation modeling (SIGMA/W), dynamic modeling (QUAKE/W), thermal modeling (TEMP/W), contaminate modeling (CTRAN/W) and vadose zone modeling (VEDOSE/W) (Hassani et al., 2013).

For this study, Geo-Studio 2007 package has been used for analysis and the first four: SEEP/W, SLOPE/W, SIGMA/W and QUAKE/W have been applied used depending up on the type of problem to be analyzed.

3.5 Methodology

The intent of this study is to provide safe and reliable alternative design for middle Awash multipurpose dam and compare the alternative design (inclined clay core) with the original design (vertical clay core). Thus, the study will include dam dimensioning of different zones, seepage modeling and slope stability analysis for static and dynamic condition. And finally, comparison of inclined clay core geometry with vertical core will be carried out. The methods and material model used for each activity has been discussed as follow:

3.5.1 Dam dimensioning

For dam dimensioning, intensive literature review regarding to dam designing has been made. Dam dimensioning and zoning for the new alternative is fixed based on literatures and design manuals. During this process the experience used in our country has been reviewed and considered as base line. This section would include rearranging of the dam axis depending up on the topography, fixing of upstream and downstream rock slopes, fixing of upstream and downstream clay core slope, dimensioning of core width and other important parameters.

3.5.2 Seepage analysis

SEEP/W is a finite element based GEO-STUDIO component, used for seepage analysis. It is applied for modeling of movement of water and estimation of pore water distribution through a pores media of a soil and rocks. Its comprehensive formulation makes it possible to analyze both simple and complex seepage problems. This tool has great application in the analysis of geotechnical, civil, hydrological and mining engineering projects (SLOPE/W, 2008). SEEP/W is formulated based on the basic that the flow of water through both saturated and unsaturated soil follows the Darcy law which is expressed as:

$$q = k * i \text{ ----- Equation 3.3}$$

Where

q = specific discharge

k = the hydraulic conductivity of soil/rock media

i = the gradient of total hydraulic head

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

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

Geometry

Once dam dimension has been fixed based on guideline and standard, the geometry has been drawn on the SEEP/W window and will be discretized to small elements as finite element numerical models are based on the concepts of subdividing (discretize) a continuum into smaller pieces, describing the behavior or action of the individual pieces and the reconnecting all the pieces to represent the behavior of the continuum as a whole (SEEP/W, 2008).

Material Property and Model

SEEP/W has three material models: none model (used for removed part of a model), saturated/unsaturated and saturated model and for this study the latter two material models has been used. A summary of these models and the required soil properties are given below and a discussion of the individual parameters and functions are provided as follow (SEEP/W, 2008).

Saturated / Unsaturated model

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

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

Saturated only model

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

Boundary condition (BC)

To analyze seepage problems on SEEP/W boundary conditions has to be set as solutions are dependent on the boundary conditions assigned to a problem. The equation of finite element equation for seepage analysis would be given by (SEEP/W, 2008): -

$$[K] \{H\} = \{Q\} \text{----- Equation 3.4}$$

Where:

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

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

$\{Q\}$ = a vector of the flow quantities at the node.

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

$$H = \frac{u}{\gamma_w} + Y \text{-----Equation 3.5}$$

Where:

H = the total head (in meters)

u = the pore-water pressure (kPa)

γ_w = the unit weight of water (kN/m³)

Y = Datum elevation (meters)

Sudden draw down

In addition to what explained above, basically, two sets of boundary conditions are required to perform a transient seepage analysis: an initial boundary condition and a transient boundary condition (Huzjak et al., 2009).

Initial boundary condition is required to define pore water pressures throughout the model at the beginning of the transient analysis. For this analysis, the initial boundary condition is taken from SEEP/W result which is a total head on the upstream embankment slope equal to the reservoir level at the initiation of drawdown.

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

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

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

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

Procedure

- The maximum dam cross-section will be drawn on SEEP/W window
- The respective materials will be assigned to their regions

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

Results to Obtain

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

3.5.3 Slope stability analysis

For slope stability analysis SLOPE/W which is a component of GEO-SLOPE has been used which is designed and developed for analysis of stability of slopes. The package is based on limit equilibrium (LE) principle and it consists of many different methods inside. Among the different methods incorporated in this package, Morgenstern-Price method is used for this thesis as this method satisfies both force equilibrium and moment equilibrium. Morgenstern-Price method assumes that the shear force between slices is related to the normal forces (SLOPE/W, 2008) as:

$$X = \lambda f(x)E \text{ ----- Equation 3.6}$$

Where

X is inter-slice normal force

E is inter- slice shear force

λ scaling factor

$f(x)$ an assumed function that has prescribed value at each slice boundary

This package requires three input data depending up on the problem, and these input parameters are geometry, soil property and pore-water pressure. Providing all the necessary parameters limit equilibrium will compute a factor of safety with the equation Duncan and Wright, (2005) below:

$$F.S = \frac{\text{Shear strength } (\tau)}{\text{Equilibrium shear stress (S)}} \text{----- Equation 3.7}$$

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

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

$$S = C + \sigma_n \tan(\phi) \text{----- Equation 3.8}$$

Where,

C and ϕ are the cohesion and internal friction angle for the soil respectively,

σ_n is the total normal stress on the shear plane

Geometry

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

Material property and model

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

Procedure

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

Results to Obtain

At the end of this analysis the state of condition of embankment slope stability has been determined through a single number called factor of safety and interpretation and discussion has been made for all results computed during different loading conditions based on the criteria presented in Table 3.9 below. According to USACE (2003) the minimum factor of safety with respect to loading condition has been summarized as follow

Table 3.9: Minimum required factor of safety versus loading condition

Slope	Loading condition	Reservoir characteristics	Minimum FS
Upstream and downstream	End of construction	Reservoir empty	1.3
Downstream	Steady State seepage	Reservoir at level (full supply level)	1.5
Downstream	Maximum flood	Reservoir at maximum flood level	1.5
Upstream	Drawdown	Rapid drawdown to critical level	1.3
Upstream and downstream	Earthquake loading	Reservoir at maximum flood level	1.0

3.5.4 Stress deformation analysis

SIGMA/W, component of GEO-STUDIO has been applied for stress deformation analysis. This package based on finite element principle, dividing the whole problem domain to finite elements. To simulate the actual site condition principle of construction stages has been applied assuming that the embankment has been constructed in a sequence of many embankment lifts (Zomorodian and Chochi, 2012).

The basic finite element equation that SIGMA/W uses is expressed as in the equation below (SIGMA/W, 2008):

$$[K] \{d\} = \{F\} \text{----- Equation 3.9}$$

Where

[K] is system stiffness or characteristic matrix

$\{d\}$ = vector of nodal displacement (primary unknowns to be computed)

$\{F\}$ = vector of nodal action forces

Geometry

Similar procedure to that of SEEP/W and SLOPE/W has been used.

Material property and model

SIGMA/W is formulated for several elastic and elasto-plastic constitutive soil models all models may be applied to two-dimensional plane strain problems. Among the material models presented nonlinear elastic model for clay core, linear elastic model for the rest of embankment and foundation material has been used based on the recommendation given Khran, (2007) and Zomorodian and Chochi, (2012).

Young's Modulus, Poisson's Ratio, Cohesion and Friction angle of the soil and rock materials has been used as input parameters (SIGMA/W, 2008).

Boundary condition

Fundamentally, there are two types of boundary conditions that can be applied in a stress-deformation model: it is either force or displacement. And in this thesis a boundary condition of force has been given to compute the displacement of embankment. In general, it is common to fix the left and right sides of a problem along with the bottom edge (SIGMA/W, 2008).

Procedure

- Insitu stress of ground condition has been set up from SIGMA/W
- The dam cross section has been drawn with successive lifts to model the actual site condition
- Material properties has been assigned for each lift
- Boundary condition has been applied
- Water load has been applied depending up on loading condition
- Load deformation type of analysis has been done to compute static deformation

Results to Obtain

Major total stress, minor total stress and deformation have been found from this analysis and the stress and deformation computed during different loading condition has been interpreted and discussed.

3.5.4.1 Assessment of core cracking and hydro fracturing

Considering the water in the reservoir to be at its maximum level, results obtained from SIGMA/W will be used for the assessment of potential cracking and hydro fracturing in the dam clay core considering different stress and pore-water pressure at selected nodal points.

For the assessment of hydro fracturing, nodal points at different height will be considered and the maximum total stress and minimum total will be computed at these points and compared with the respective pore water pressure. To be more conservative, these nodal points have been selected near to the upstream edge of the clay core as the pore water pressure is relatively higher at these locations. Similarly, possibility of horizontal and vertical core cracking in the core will be analyzed by comparing the maximum horizontal and maximum vertical stresses against pore water pressure at same location.

3.5.5 Dynamic analysis

QUAKE/W is a part of Geo-Studio, which is fully integrated with other components like SEEP/W, SLOPE/W and SIGMA/W. QUAKE/W is finite element software product used for the dynamic analysis of earth structure subjected to earthquake shaking and other sudden impact loads like dynamiting and pile driving (QUAKE/W, 2008). As middle Awash dam project is located in highly seismic region, the dynamic response of the embankment has been studied with QUAKE/W. In the analyzing the dynamic stability of Middle Awash multipurpose dam, the case histories conducted on upper and lower San Fernando has been considered as these cases studied are famous in the history of geotechnical engineering (Seed et al., 1975).

Geometry

Similar procedure to that of SEEP/W, SLOPE/W & SIGMA/W has been used for analyzing the geometry.

Material property and model

There are four different material models to choose from when using QUAKE/W. These are none model, linear elastic, Equivalent linear and Non-linear model. Among these models equivalent linear model has been chosen for this analysis. The material properties required for equivalent linear method are Unit weight, Poisson's ratio, cohesion, internal angle of friction, damping ratio as constant or function, pore-water pressure function, G reduction function and G_{max} as constant or function

Boundary Condition

In QUAKE/W, all boundary conditions must be applied directly on geometry items such as region faces, region lines, free lines or free points. Nodal displacements are the most often type of boundary condition (QUAKE/W, 2008). In this analysis the boundary condition has been attached at the base and sideways of the foundation.

Procedure

The procedure or methods used for dynamic analysis of middle Awash has been presented as follow:

- In dynamic analysis the first procedure is that in-situ stress of the ground condition has to be computed and this initial stress can be computed with either SIGMA/W or QUAKE/W based on user preference (QUAKE/W, 2008). In this study the initial in-situ stress has been computed from SIGMA/W
- Pore-water pressure has been computed from SEEP/W and applied on this analysis
- The problem domain (geometry) has been drawn on QUAKE/W and concatenated to SEEP/W and SIGMA/W for consideration of pore water pressure and initial in-situ stresses

- Shaking type of analysis has been made with QUAKE/W by providing the site specific seismic data with equivalent linear type of model
- Once the shaking analysis has been made the change in stress computed on QUAKE/W analysis has been used in SLOPE/W to compute the factor of safety of embankment slope after the earthquake
- Once a safe embankment slope has been found after shaking, stress redistribution type of analysis has been made on SIGMA/W to compute the permanent deformation that the embankment exercised
- Finally, all the results computed has been analyzed and discussed

3.5.6 Comparison of inclined and vertical clay core geometry

After a safe and stable design with inclined core geometry has been found, comparison of inclined clay core with vertical clay core has been conducted. For comparison purpose the vertical clay core dam designed by ECDSWC has been used. Economy, static stability, static deformation and dynamic stability condition has been accounted as comparative parameters. And finally, based on the result found conclusion and recommendation has been provided in the thesis.

3.6 Project schematization

3.6.1 Dam Zoning and Geometry

It has been discussed earlier in this document that middle Awash multipurpose dam originally being designed a rock fill dam with central clay core which requires a special foundation treatment for the core foundation (ECDSWC, 2016). To solve this problem as an alternative the core geometry has been modified to inclined core instead of vertical core and all required analysis has been done for the new dam alternative.

Revision of the dam cross section has been done based on the basic criterion listed in section 2.6.1. The slope of the outer shell with 2H: 1V and 1.6H: 1V for upstream and downstream respectively has been chosen for the alternative design providing 6 downstream and 2 upstream berms with a width of 4m.

A core top width of 3m has been provided for utilization of construction equipment. And considering the actual site condition regarding to the location of the fault breccias zone and economic consideration, a clay core slope of 1V:0.8H and 1V:0.4H upstream and downstream respectively have been selected for the alternative design.

For prevention of internal erosion, piping and development of excessive pore pressure inside the dam a vertical transition and filter material of appropriate gradation is provided at both sides of the clay core. A horizontal drainage blanket is also incorporated to safely discharge the seepage water towards downstream of the dam and inner shell material is provided between the filter and rock fill zone to avoid the gradation gap between these materials.

The maximum dam cross-section of the original design (central vertical clay core) prepared by water works design and supervision enterprise ECDSWC (2016) (Fig.3.8 a) and the newly modified dam cross-section with an inclined clay core (Fig. 3.8 b) are shown below.

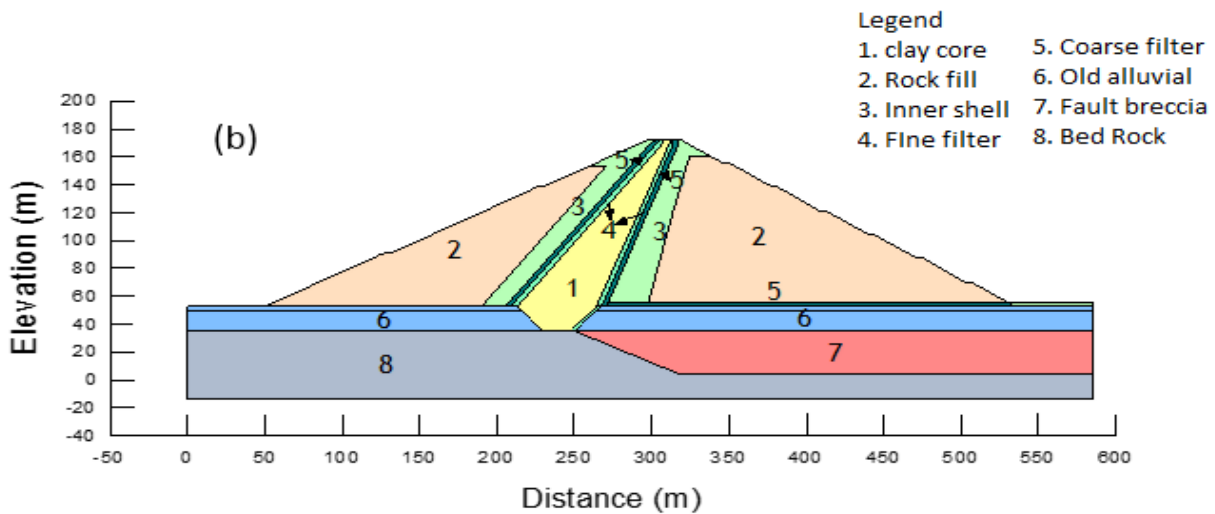
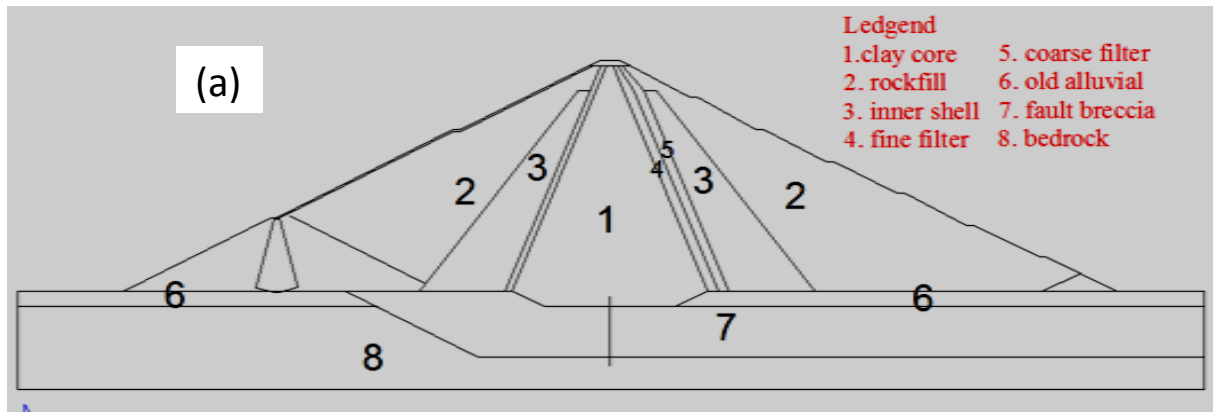


Figure 3.8: Maximum dam cross section a) original design b) alternative design

Some important geometrical parameters of the original design and alternative design considered in the design has been summarized and presented in the following table 3. 10.

Table 3.10: Geometrical parameters of dam cross section

Dam characteristics	Original Design	Alternative Design
Top width	12 m	12 m
Core top width	3 m	3 m
Number of upstream berm	2	2
Number of downstream berm	6	6
Berm width	4 m	4 m
Upstream slope		
Rock	1V:1.9H	1V:2H
Inner shell	1V:0.75H	1V:0.8H
Clay core	1V:0.4H	1V:0.8H
Downstream slope		
Rock	1V:1.75H to 1V:1.9H	1V:1.6H
Inner shell	1V:0.75H	1V:0.25H
Clay core	1V:0.4H	1V:0.4H

4 RESULTS AND DISCUSSIONS

4.1 Seepage Analysis

The seepage analysis for the new design alternative has been done and discretization of the whole domain in to finite elements was the first step in finite element method. A combination of quadrilaterals and triangular polygons are used as these polygons are best mesh shape for two dimensional analyses. The number of finite elements has been adjusted based on the recommendation given by SEEP/W (2008) engineering book where each discretized element is clearly visible on 100% zoom extent (Fig.4.1). In addition to domain discretization, boundary condition has been applied for upstream total head, downstream total head and zero pressure lines on downstream filter blanket (Fig. 4.1).

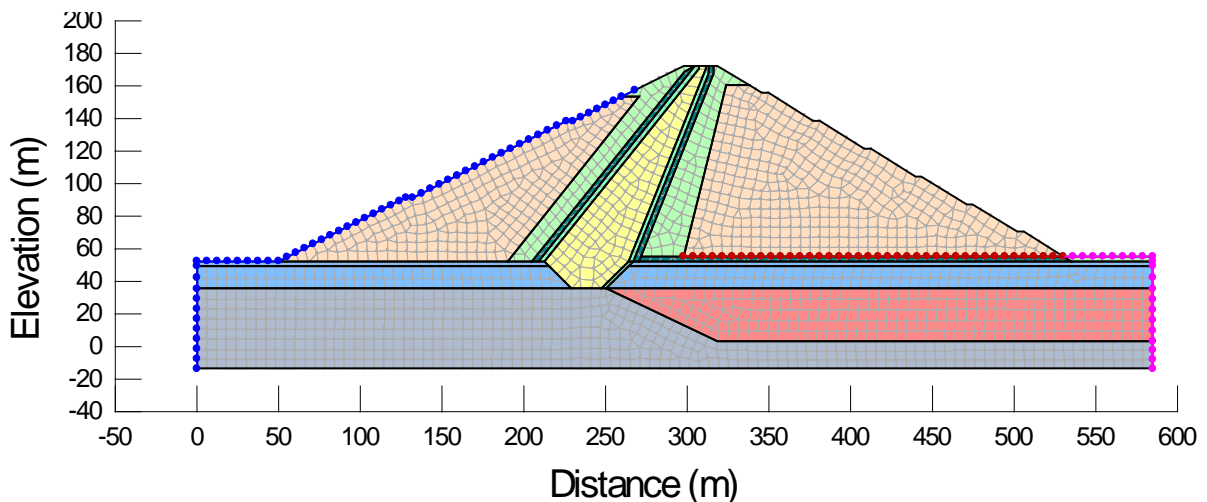


Figure 4.1: Finite element discretization and boundary condition applied for seepage analysis

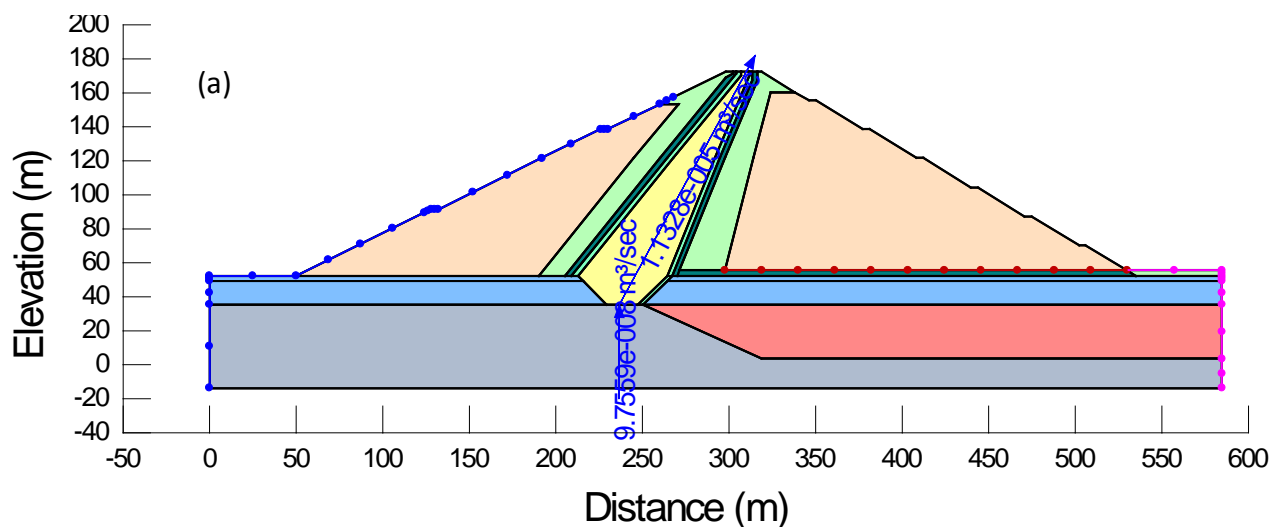
The flux through the dam body and the foundation has been computed and based on the computation, the total flux through the dam body and foundation is estimated to be $1.143 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m}$. Among the total flux found, a flux of $9.76 \times 10^{-8} \text{ m}^3/\text{sec}/\text{m}$ is through the dam foundation and $1.133 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m}$ is through the dam body (Fig. 4.2a). Considering the crest length of Awash multipurpose dam to be approximately 500 m, the total volume of water seeps through the dam body and the foundation is estimated to be $0.0057 \text{ m}^3/\text{sec}$.

The rate flow of water through dam body and foundation by itself does not lead to a conclusion regarding to dam safety problem. However, the quantity of seepage through the embankment

should be controlled to required level. A leakage of $0.03 \text{ m}^3/\text{sec}$ through the embankment and dam foundation is generally acceptable provided that proper filter material, drainage system and relief wells are incorporated (Jansen, 1988). In comparison to this recommendation the quantity of seepage loss through the dam body and foundation calculated in this research lies on the allowable range so that it is found the newly proposed dam cross section is satisfactory regarding to ability of water retaining.

The quantity of flux calculated on the original design prepared by water works design and supervision enterprise for the case of vertical clay core ECDSWC (2016) is $1.04 \times 10^{-5} \text{ m}^3/\text{sec}$ and $4.18 \times 10^{-6} \text{ m}^3/\text{sec}$ through the foundation and dam body respectively (Fig. 4.2b) so that the total volume of water seeps through the dam is expected to be $0.0073 \text{ m}^3/\text{sec}$.

The result of total flux the original design prepared by ECDSWC and alternative design proposed in this research different for the dam foundation as well as through the body of the dam. This is basically due to the reasons that in this research the core thickness is minimized to be economical as much as possible considering the scarcity of clay material within the reasonable hauling distance so that the flux trough the core is relatively higher however it does not violate the design requirements. The flux calculated through the foundation for the modified cross section (inclined clay core) considerably less as compare to the original design computed by ECDSWC because in modified section the core material has been placed on rock foundation and the direct contact of the reservoir water to the fault breccia has been intercepted with impervious material so that seepage loss trough the fault breccia is hugely controlled.



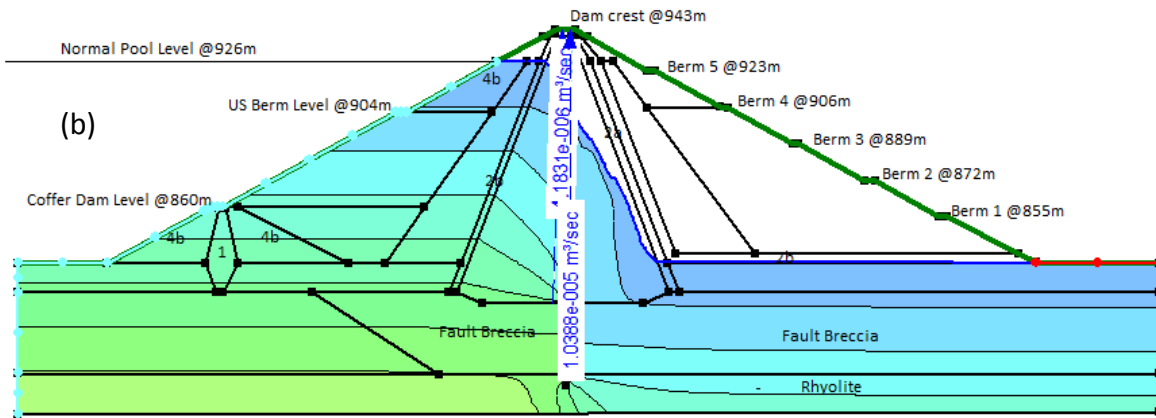
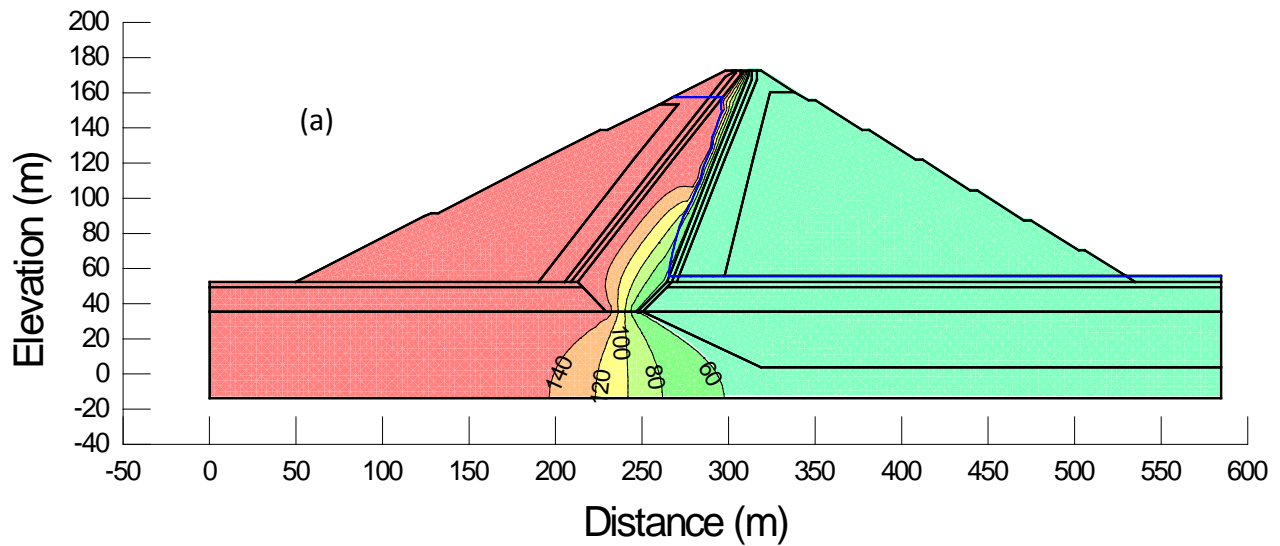


Figure 4.2: Flux through dam body and foundation for a) alternative design b) original design

The line of zero pressure and contours of total head has been shown in figure 4.3a. As it has been shown in figure that the line of zero pressure is horizontal until it reached to clay core because all the materials upstream of the core are assumed to be free drain, no energy is dissipated, and as it gets in to the clay core it is rapidly drawn. The distribution of total head has also been presented in the figure 4.3 a, where this total head is a function of pore pressure and elevation head.

The distribution of pore water pressure through the dam body and the foundation at steady state condition is shown in Figure 4.3b and this result will be used as an initial boundary condition in SEEP/W, SLOPE/W and QUAKE/W for transient analysis during sudden drawdown and for downstream slope stability analysis and dynamic stability analysis.



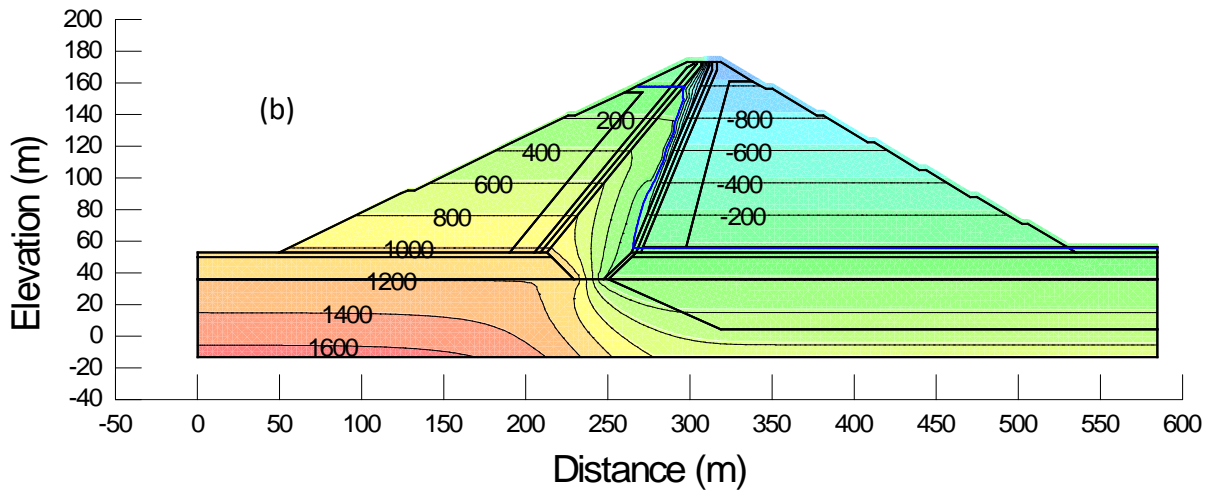


Figure 4.3: Contours of a) equipotential lines (total head in m) b) pore water pressure (in kPa)

The hydraulic gradient inside the dam body is associated with the possibility of material erosion and piping so that the hydraulic gradient at the contact of core to foundation contact and core to transition material has to be controlled. The hydraulic gradients are computed in the dam body and foundation for the alternative design and the result varies in the range of 0.5 to 3.5 (Fig. 4.4). For inclined core embankment dams, it is recommended that the hydraulic gradient should not exceed 4 (Kutzner, 1997). Therefore, the computed hydraulic gradient in this work does not violate the requirement. The vertical gradient inside the dam body and the foundation has also been computed and the result varies between the ranges of 1.81×10^{-7} to 3.15×10^{-7} which is much less than the allowable exit gradient of 1, so that the dam foundation will not be susceptible for erosion and soil boiling.

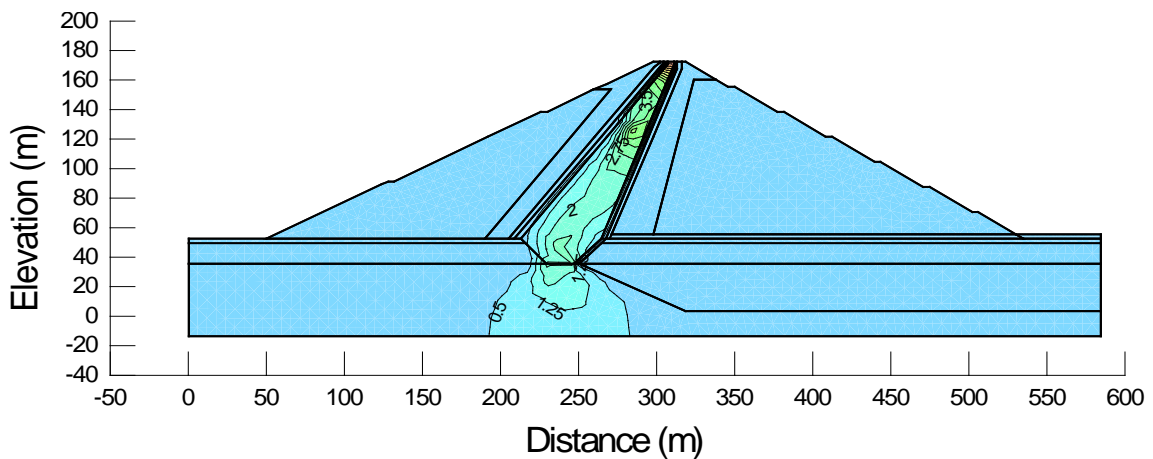


Figure 4.4: Contours of horizontal gradient through dam body and foundation

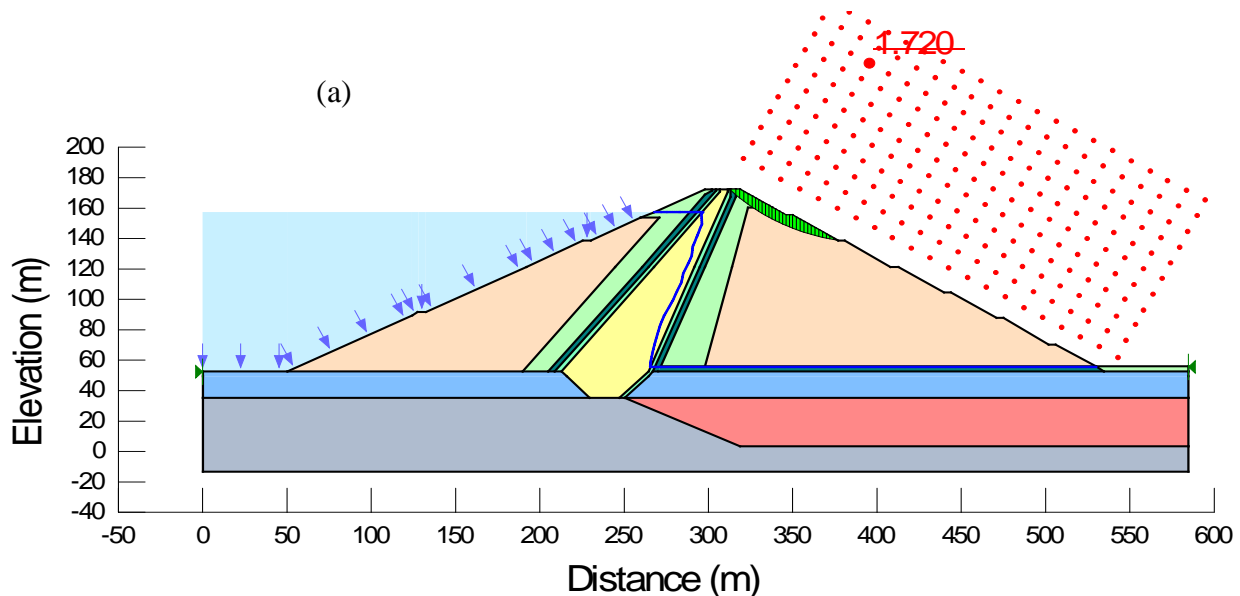
4.2 Static Slope Analysis

Slope stability analysis has been done based on the respective shear strength parameters of each material presented in section 3.3.2.2. Based on the analysis conducted, the newly revised section of the dam with an inclined clay core satisfies the minimum requirements in all loading condition. The results and discussion of stability analysis has been presented as follows based on each loading conditions.

4.2.1 Steady state condition

The downstream embankment slope has been analyzed for steady state condition when the reservoir is at normal pool level and a minimum factor of safety of 1.72 has been found (Fig.4.5a). Besides the mass which intended to fail at this scenario is so thin so it will not compromise the stability of the embankment. The minimum factor of safety required for this loading condition is 1.5, so that the embankment is safe during steady state condition.

The minimum factor of safety computed by water works design and supervision enterprise ECDSWC (2016) for steady state loading condition was 1.63 (Fig. 4.5b). The factor of safety has been improved for the alternative due to the reason that majority of the material with high shear strength is located in the downstream side due to the inclination of the core material towards the upstream.



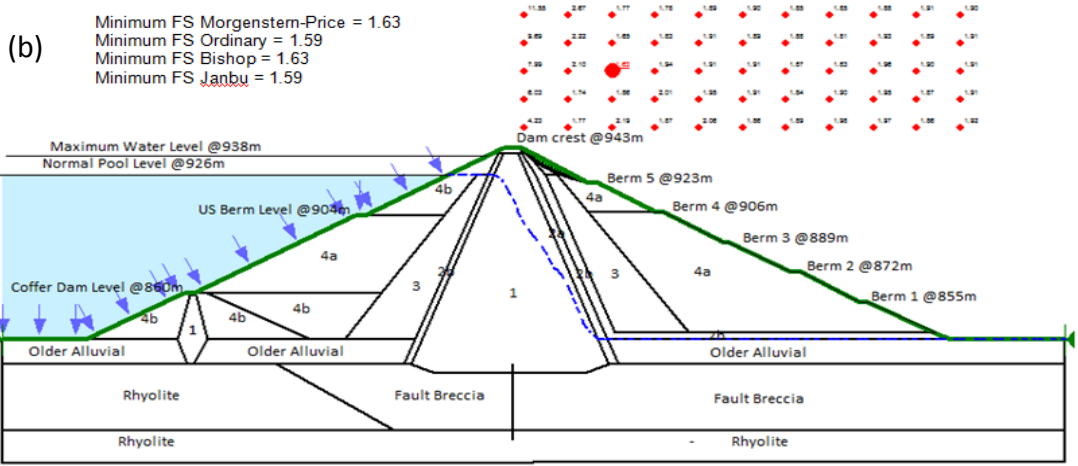
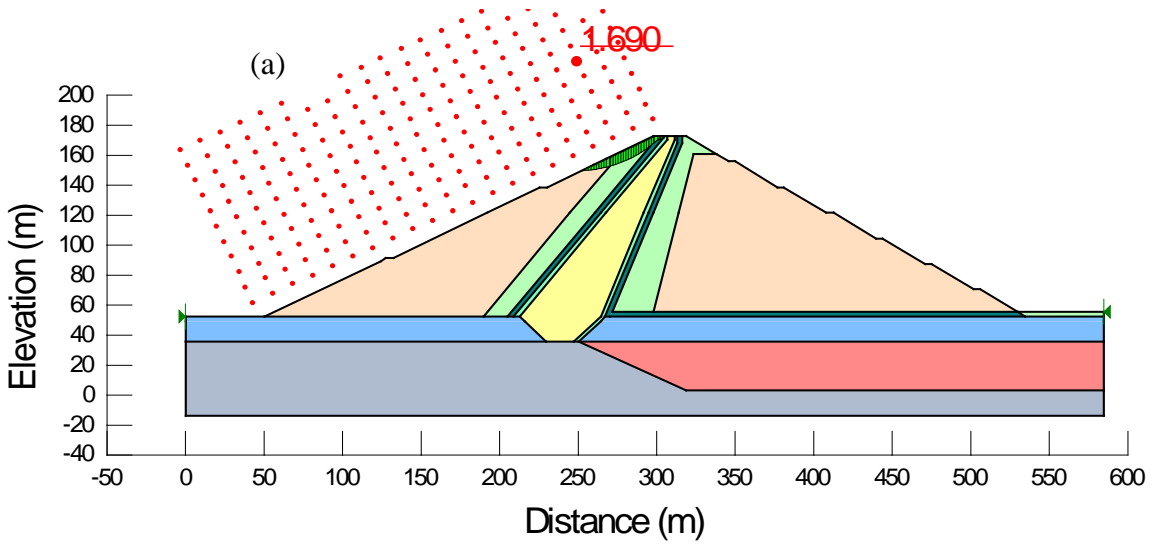


Figure 4.5: Factor of safety of downstream slope at steady state condition a) alternative design b) original design

4.2.2 End of construction

The stability of both upstream and downstream slopes during end of construction has been analyzed and a minimum factor of safety of 1.69 (Fig. 4.6a) and 1.531 (Fig. 4.6b) found for upstream and downstream slopes respectively. Despite the factor of safety for the downstream slope is less as compare to the original design 1.63 ECDSWC, (2016) but it satisfies the minimum requirement that the factor of safety during end of construction should not less than 1.30 (USACE, 2003).



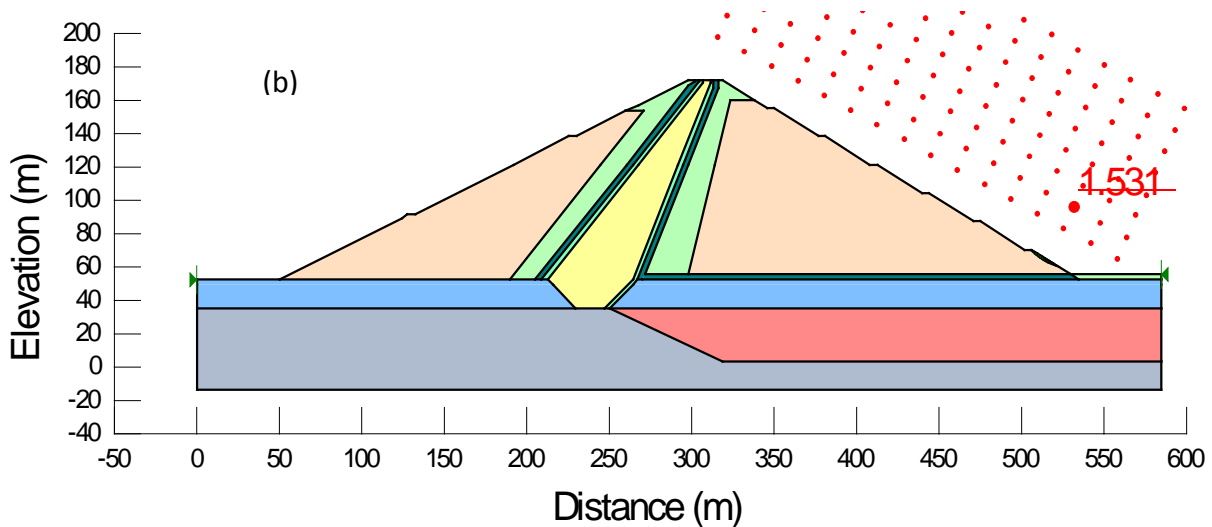


Figure 4.6: Factor of safety of downstream slope at steady state condition a) alternative design
b) original design

4.2.3 Construction stage

The stability of the embankment has also been analyzed during construction stages and the respective factor of safety has been computed. In this study for slope stability computation it has assumed that the embankment to be constructed in two stages and the upstream and downstream slopes has been analyzed for both conditions. Based on the computation, the minimum factor of safety during construction stage has been found to be 1.501 (Fig 4.7) and 1.587 (FigA.3) at the downstream and upstream respectively. According to USACE (2003) the minimum required factor of safety during construction stage is 1.3. Hence the revised section of the dam will be safe against structural failure during construction. The results computed for construction condition has been presented through Figure A.2 to Fig. A.4 of Appendix A for further reference.

The respective factor of safety computed in the original design was 1.630 and 1.570 for downstream and upstream slope respectively. The upstream slope for revised design has been found to have a slight improved factor of safety as compared to the original design and the downstream slope has a relatively less factor of safety as the downstream embankment slope becomes relatively steeper during revision. But the margin of safety computed for both upstream and downstream are still more than adequate as compared to the minimum requirement suggested by USACE, (2003). Besides the mass of embankment which the failure surface crosses for the downstream slope is so thin and manageable as shown (Fig. 4. 7).

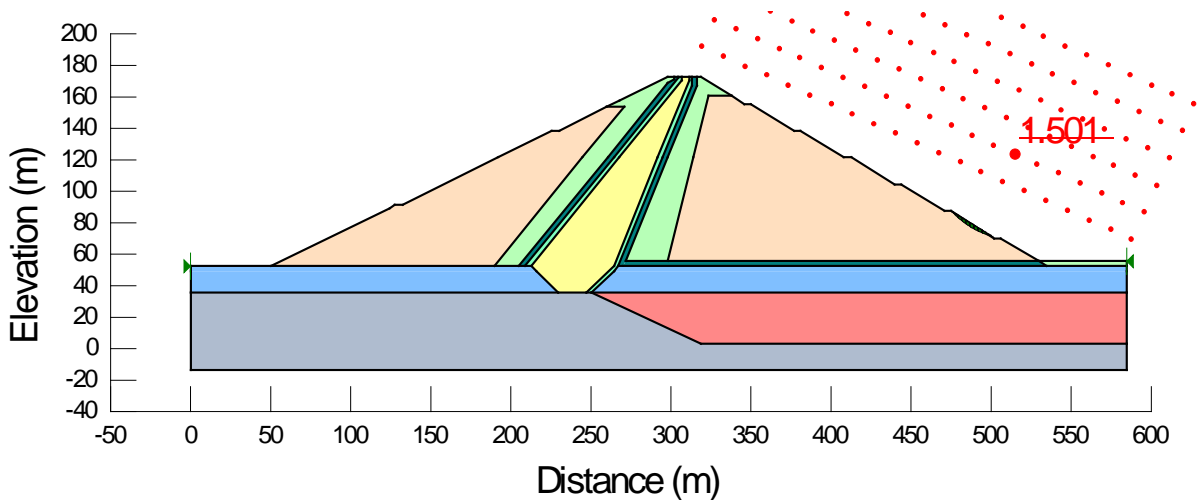


Figure 4.7: Factor of safety for downstream during second stage construction

4.2.4 Sudden draw down

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

Basically, two sets of boundary conditions are required to perform a transient seepage analysis: an initial boundary condition and a transient boundary condition (Huzjak et al., 2009). Initial boundary condition is required to define pore water pressures throughout the model at the beginning of the transient analysis. For this analysis, the initial boundary condition is taken from SEEP/W shown in Figure 4.1 which is a total head on the upstream embankment slope equal to the reservoir level at the initiation of drawdown.

The required initial pore water pressure just before sudden draw down has been taken from the steady state condition. The required boundary condition has been attached and the phreatic line of pore water dissipation has been analyzed (Fig. 4.8 and Fig. 4.9). The phreatic level drops with increasing time of analysis as shown in Figure 4.9, as pore water pressure get time to dissipate.

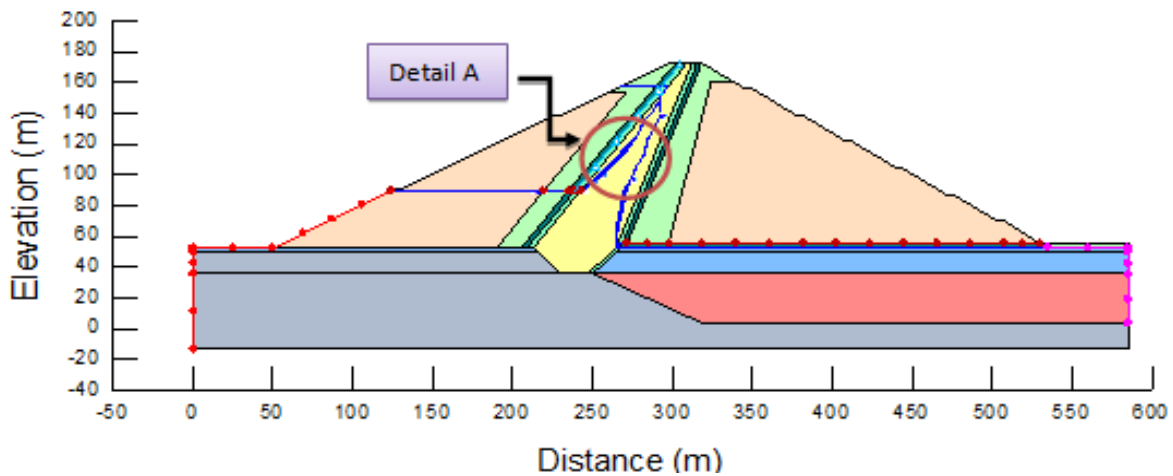


Figure 4.8: Pore-water dissipation of sudden drawdown during analysis period

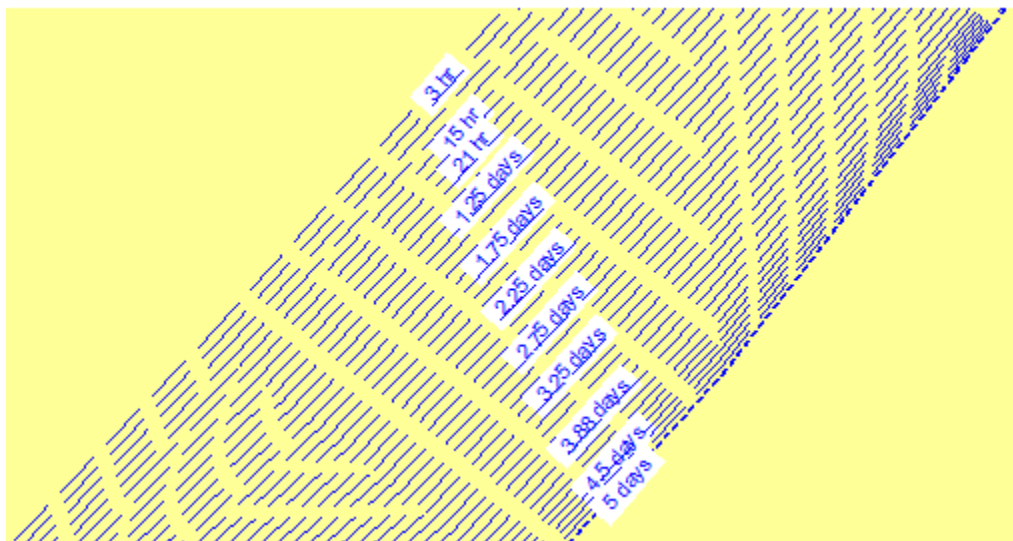


Figure 4.9: Detail A: - Changing positions of the piezometric line after instantaneous drawdown

The stability has been analyzed for 5 days since draw down with an interval 0.125 day (3 hr) and the factor of safety computed at the possible immediate time after the draw down has been found to be 1.464 (Fig. 4.10 a) which is more than adequate as compared to the minimum requirement 1.3 according to USACE (2003) for this loading condition.

The respective factor of safety calculated by ECDSWC for the case of vertical clay core is 1.540 (Fig.4.10b). The reason for slight reduction in factor of safety of upstream slope for the alternative design as compare to the original design is that the non-free draining material (clay

core) has been closer to the upstream slope that more pore water pressure is found in the sliding mass as compare to the vertical clay core (Khanna et al., 2014).

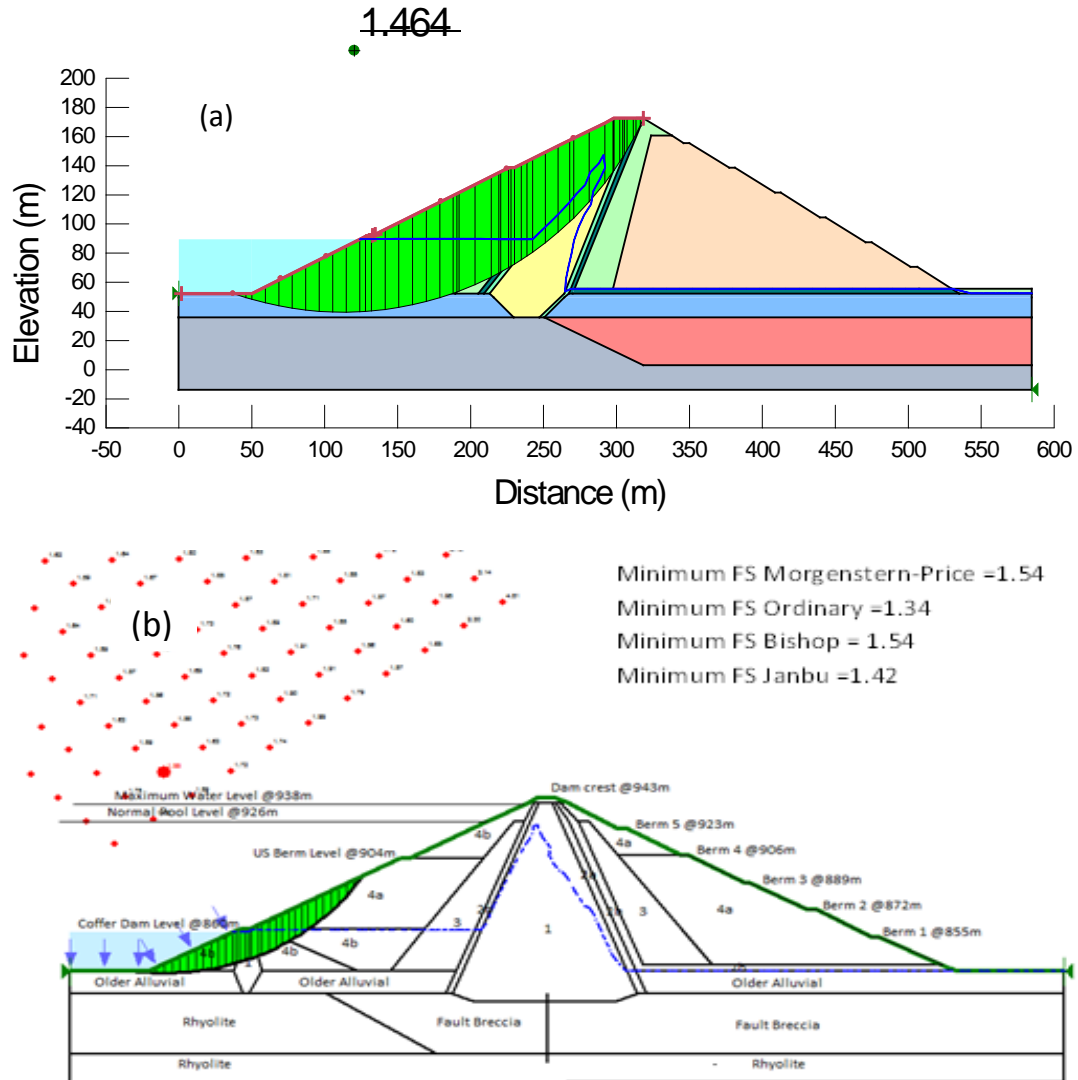


Figure 4.10: Factor of safety of upstream slope during sudden drawdown for a) alternative design b) original design

Although it was not critical loading condition, the stability of upstream slope was computed during steady state condition as reference and a factor of safety of 1.601 is found (Appendix, Fig. A.1). Evacuating the reservoir instantaneously caused the factor of safety to fall immediately to 1.464 and it recovers with time as excess pore water pressure dissipates (Fig. 4.11).

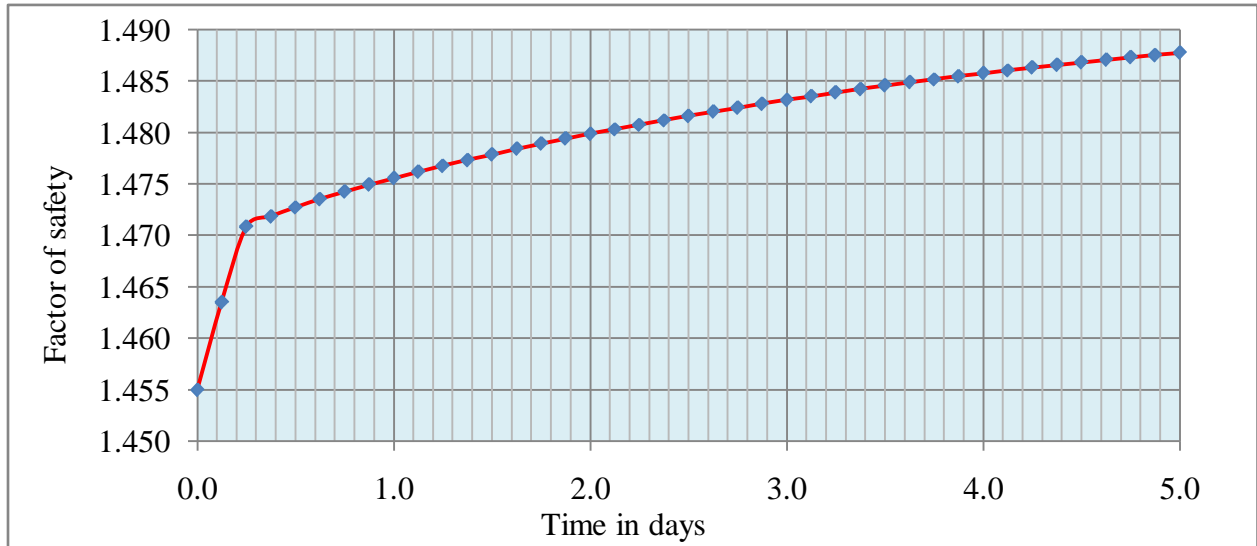


Figure 4.11: Factor of Safety versus time after instantaneous drawdown

4.3 Static Deformation Analysis

The static stress deformation analysis of middle Awash multipurpose dam with inclined clay core has been done. The results found from the analysis and the respective discussions of these results are presented below. The vertical deformation, horizontal deformation and stress state of the embankment during different loading conditions has been computed. Based on the result found the embankment satisfies all the requirements. Boundary condition and finite element mesh of the domain used for stress deformation analysis has been shown in figure 4.12 below.

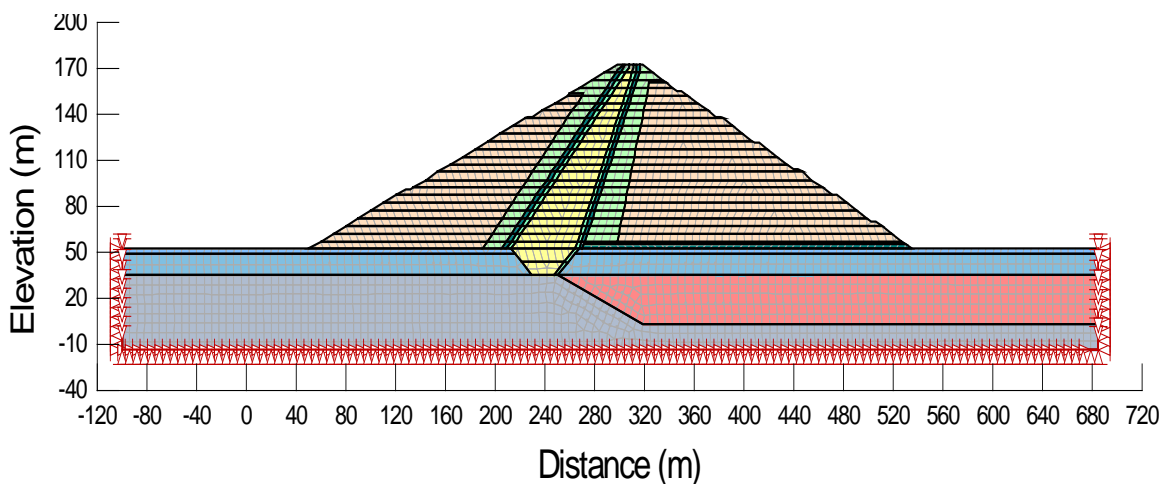
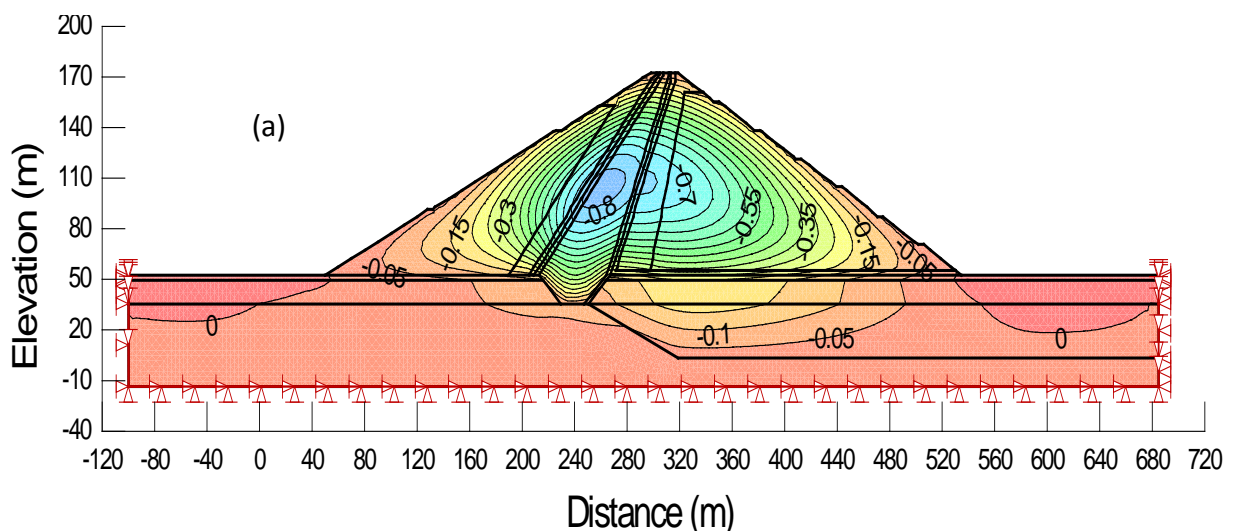


Figure 4.12: Boundary condition and finite element discretization of the domain

4.3.1 During construction / End of condition

The deformation and stress condition of the dam has been computed at the end of construction. At the beginning the dam was assumed to be constructed instantaneously so that the vertical deformation of 1.4 m has found to occur at the top of the dam but this result is unrealistic that the dam of such height and volume cannot be constructed instantaneously. Due to this fact, it is found necessary to exercise stress deformation analysis of such dams with construction stages as recommended by Tancev, (2005). Based on the recommendation the embankment has been analyzed with construction stages of 5 m lift height and the maximum vertical displacement found is 0.82 m inside the core as shown figure 4.13 (a) and (b), the horizontal displacement at the end of construction ranges from -0.14 m to 0.18 m at upstream and downstream respectively as shown figure 4.14. The negative sign on horizontal displacement shows that the direction of the deformation tends to towards upstream and positive value shows the deformation of the embankment to be downstream side.

It is always assumed that deformations are a function of dam height and vertical settlements are not likely to exceed 1% of the dam height and horizontal deformation should not be greater than 50% of the vertical deformation (Straubaa et al., 2009). According this standard the vertical settlements and horizontal deformation computed at the end of constructions are with in the allowable limit.



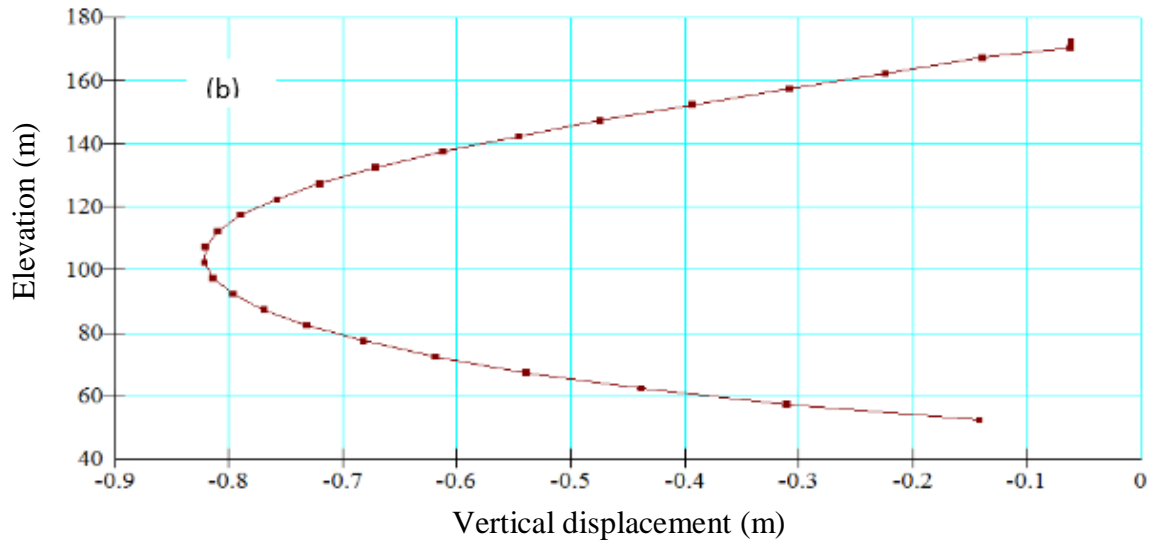


Figure 4.13: Vertical displacement at end of construction a) Contours of vertical displacement b) maximum displacement with dam height

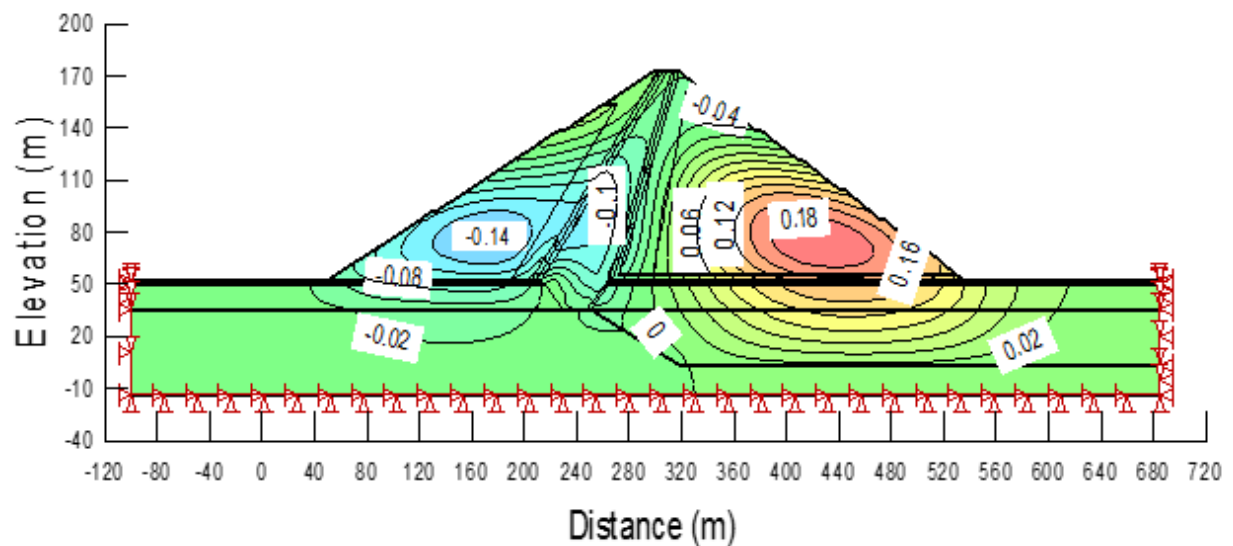


Figure 4.14: Contours of horizontal displacement for alternative design at end of construction

The stress distribution for end of construction loading condition inside the dam and foundation has been analyzed. The maximum total stress computed is 3000 kPa (Fig.4.15) on the base of the foundation, however this stress is less than the bearing capacity of the rock. Due to the modification of the core geometry the stress distribution has got changed and concentrated stress

at the base of the filter upstream and downstream material on the original design (Fig.4.16) has been minimized.

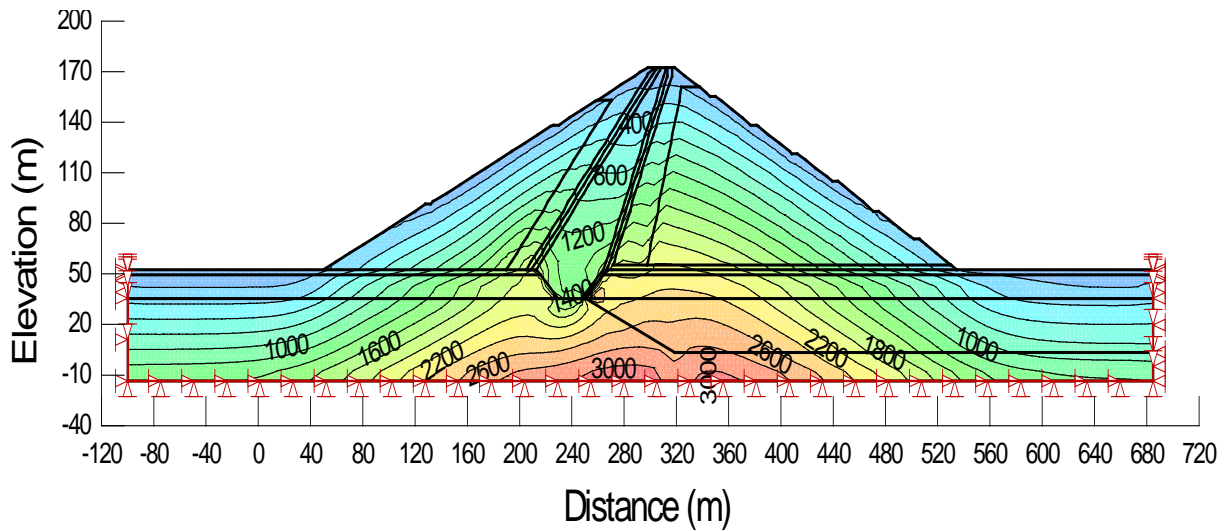


Figure 4.15: Contours of maximum total stress at end of construction for alternative design

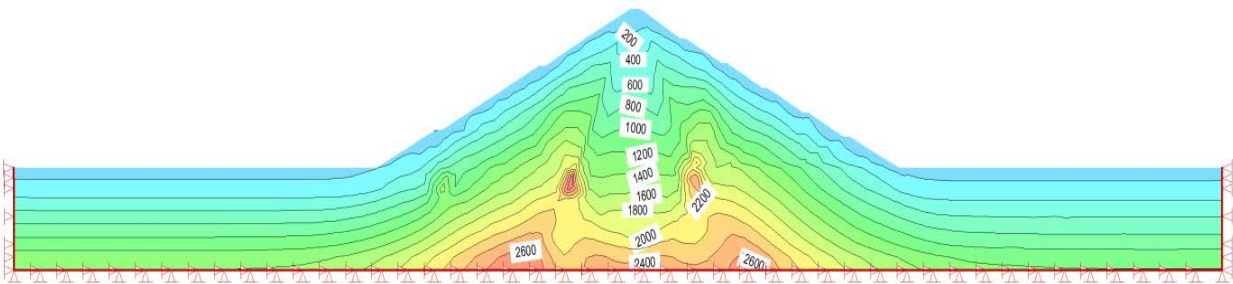


Figure 4.16: Contours of maximum total stress at end of construction for original design (ECDSWC, 2016)

The distribution of minor total stress during end of construction varies from 200 to 1200 kPa as shown in Fig. A. 5 of appendix A. Minor total stress inside the core ranges from 200 to 600 kPa, which is all positive, so that the clay is not subjected to tension which lead to core cracking.

4.3.2 Normal pool level condition

The deformation and stress distribution of modified section has been analyzed considering the effect of the upstream reservoir water at normal pool level. Based on the result of the analysis the deformation on upstream toe has been increased due to the water load subjected on the embankment. When the reservoir impounds water to its normal pool level settlement has shown

increment to 0.85 m as shown in Fig. 4.17. However, it is still within the allowable limit that no major problem is expected on the embankment.

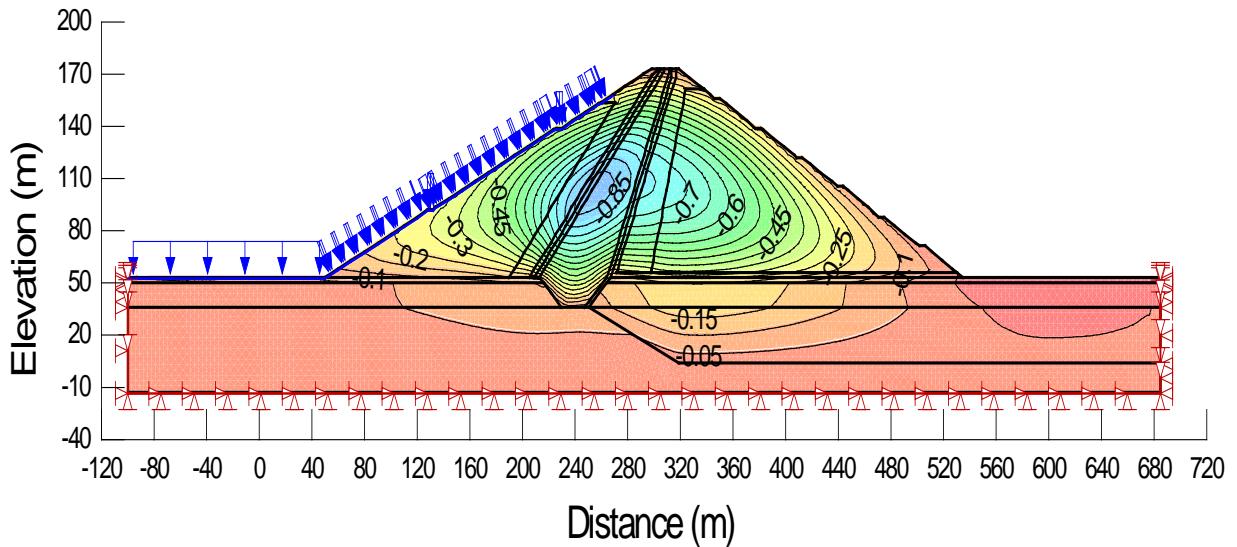


Figure 4.17: Contours of vertical displacement for alternative design at end of construction

The horizontal deformation of the dam at normal pool level has totally being modified due to the water force applied on upstream and the mode of deformation shown in figure 4.18 at a magnification scale of 30 times.

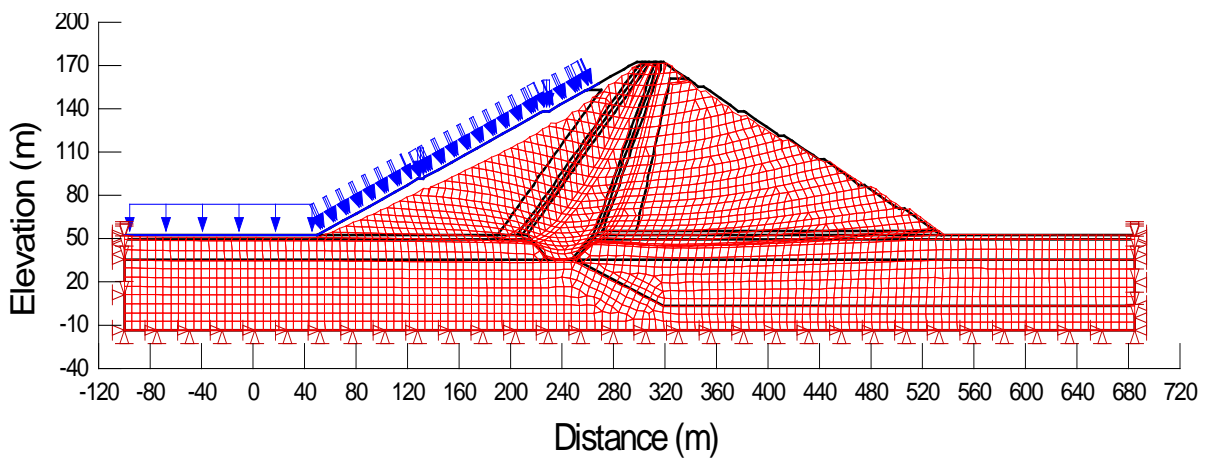


Figure 4.18: Deformed mesh of alternative design for NPL

The lateral deformation during normal full supply level ranges from 0.06 m at the upstream toe of the dam and 0.2 m at the top part of upstream (Fig. 4. 19). The deformation at downstream

also increased to 0.22 m to the load imposed by the reservoir. But the lateral deformations are all less than 0.5% of the dam height so that it is within acceptable limit.

Although the static deformation analysis with respect to reservoir impoundment has not been incorporated in the original design the deformation at the end of construction was estimated to be 0.83 m ECDSWC, (2016) which is almost similar to the result found in this work (0.8 m).

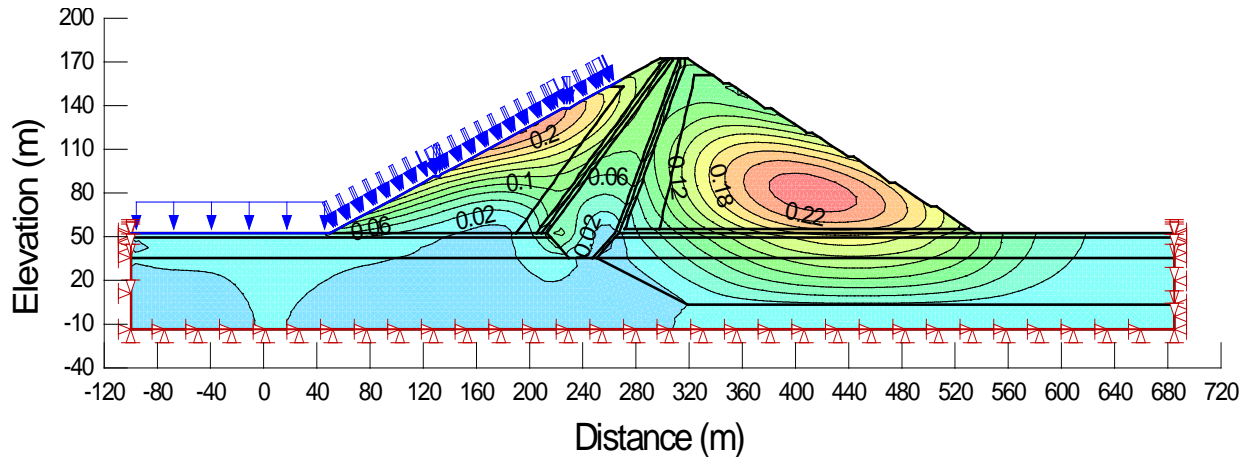


Figure 4.19: Contours of horizontal displacement for alternative at NPL

The maximum total stress on the foundation when the water at the reservoir is at normal pool level is 3000 kPa (Fig. 4.20) which is much less than the bearing capacity of the foundation. The minor total stress also varies from 200 kPa to 1600 kPa as shown in Figure A.6 (appendix A) so that no tensile stress inside the core.

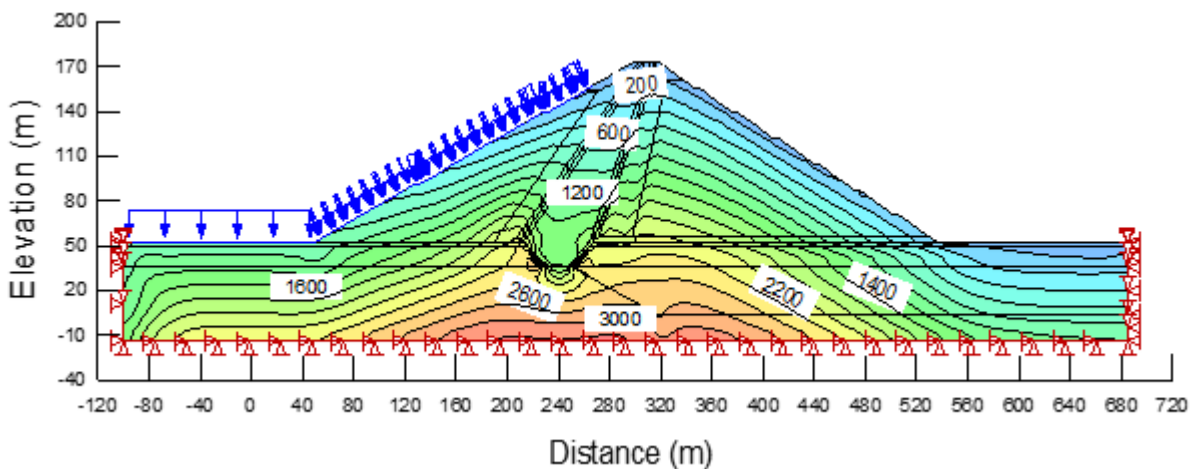


Figure 4.20: Contours of maximum total stress for alternative design at NPL

4.3.3 Maximum pool level condition

When the reservoir water reaches to its maximum level no major change in vertical settlement at the downstream part of the dam has been observed. However, there is an incremental of vertical settlement inside the core. The maximum vertical settlement for this loading condition is computed to be 0.90 m (Fig. 4.21) and vertical deformation has shown some increment at upstream side due to water load but it is acceptable as the deformation is less than 1% of the total dam height.

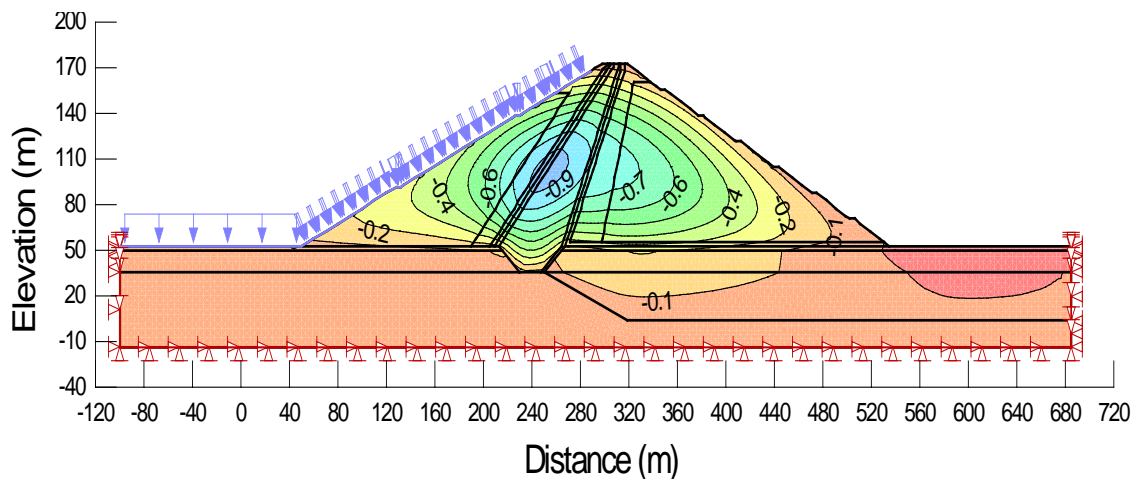


Figure 4.21: Contours of vertical displacement for alternative design for MWL

The increase of water load from normal pool level to maximum water level further deforms the embankment to lateral direction as shown figure 4.22, which is at 30 times magnification scale. A maximum horizontal deformation has been observed at the top part of upstream slope towards the downstream with a magnitude of 0.3 m. There is also a slight horizontal deformation increment in the core zone but it is in the allowable limit presented by Straubaar et al., (2009). The countours of horizontal deformation durig maximum reservoir level has been shown in figure 4.23 below.

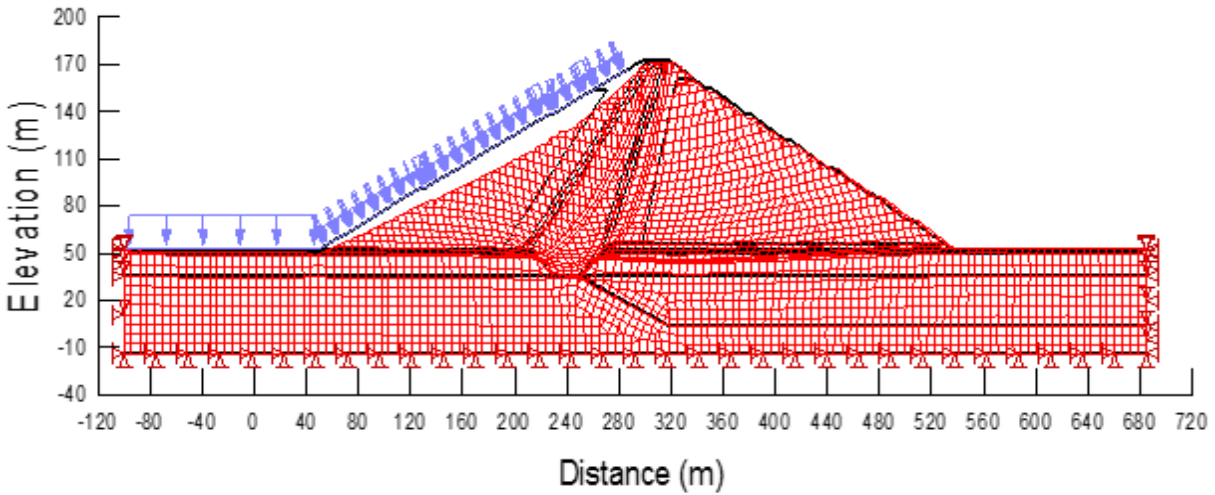


Figure 4.22: Deformed mesh for alternative design when the water level is at MWL

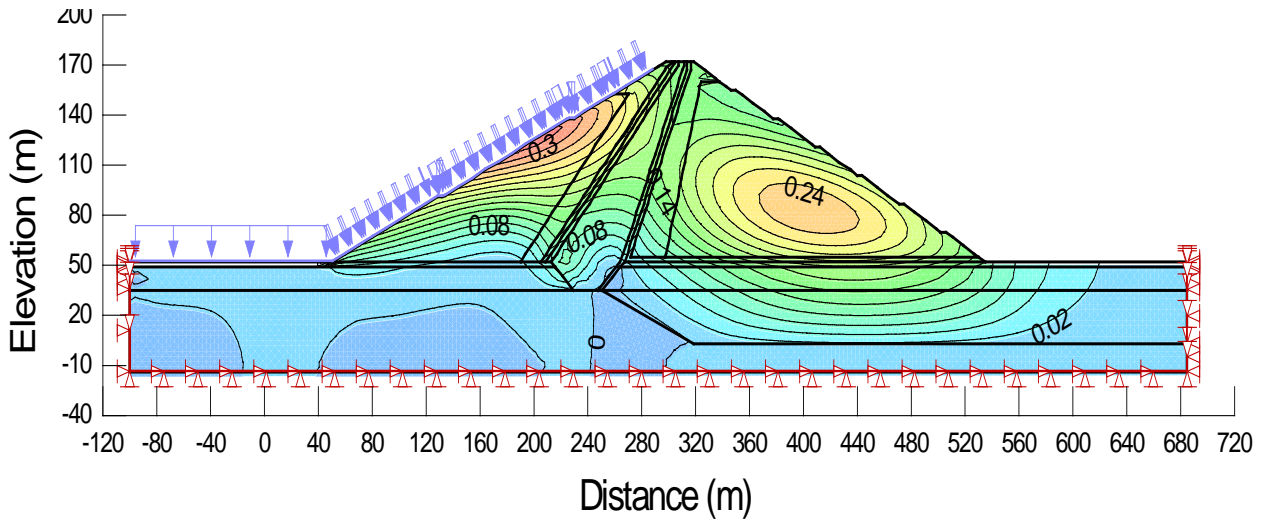


Figure 4.23: Contours of horizontal displacement for alternative section for MWL

The maximum and minimum total stress distribution during maximum reservoir level is shown in figure 4.24 below and figure A.7 (appendix A) respectively. The maximum total stress ranges from 200 to 3200 kPa and minimum total stress ranges from 200 to 1800 kPa have been found during this loading condition.

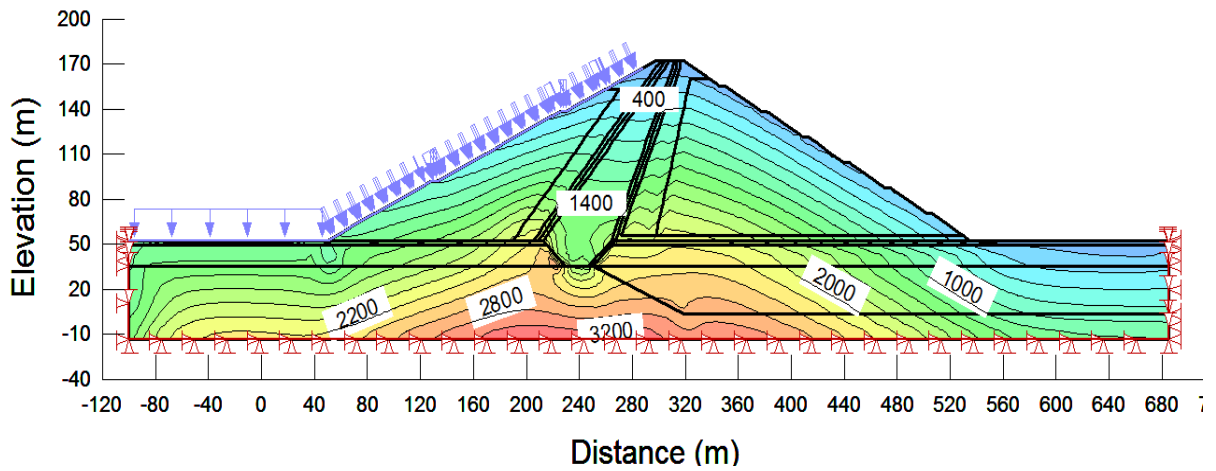


Figure 4.24: Contours of maximum total stress for alternative design for MWL

4.3.4 Hydraulic fracturing assessment

Apart from the criterion discussed in earlier section the stress in the dam body has been analyzed corresponding to whether the dam core is subjected to potential hydraulic fracturing or not. Osuji and Anyata, (2007) have studied that the impervious core material of an embankment dam is subjected to hydraulic fracturing when the minimum total stress inside the core at nodal point is less than the pore water pressure at same nodal point. Based on these criteria the minimum pore pressure inside the core is well greater than the pore pressure as shown in figure 4.25 below so that hydraulic fracturing is not expected.

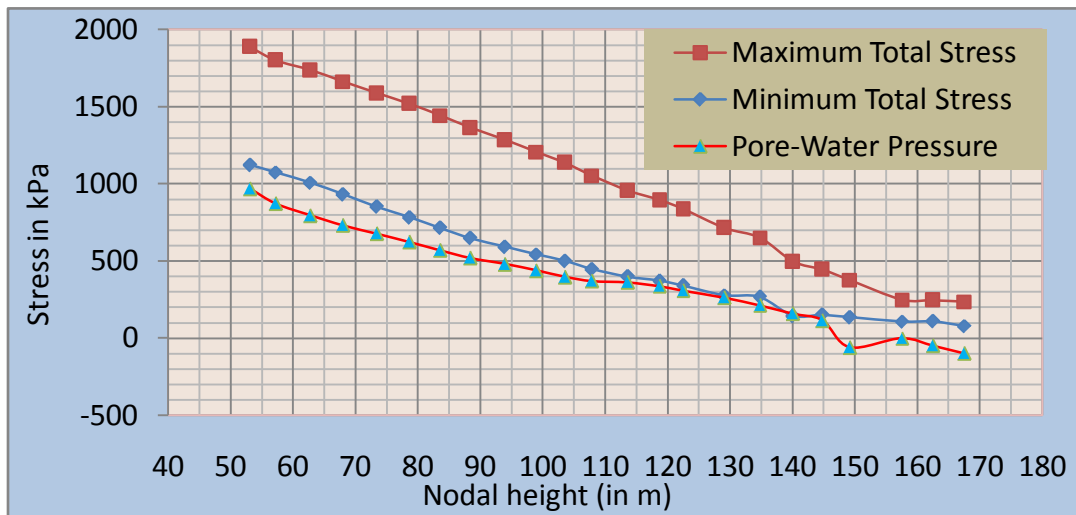


Figure 4.25: Stress distribution at the elements' nodal points in the dam core

Nayebzadeh and Mohammadi, (2011) have also studied on the possibility of hydraulic fracturing or core crack occurrence. Based on their study horizontal and vertical core crack may happen when the respective total horizontal and total vertical stress is less than the pore water pressure at same nodal point. Based on this the horizontal and vertical total stress has been analyzed with respect to the nodal pore water pressure and the pore water pressure is found less than the nodal vertical stress at all point (Fig. 4.26, a and b). Thus, both vertical and horizontal cracks are not expected.

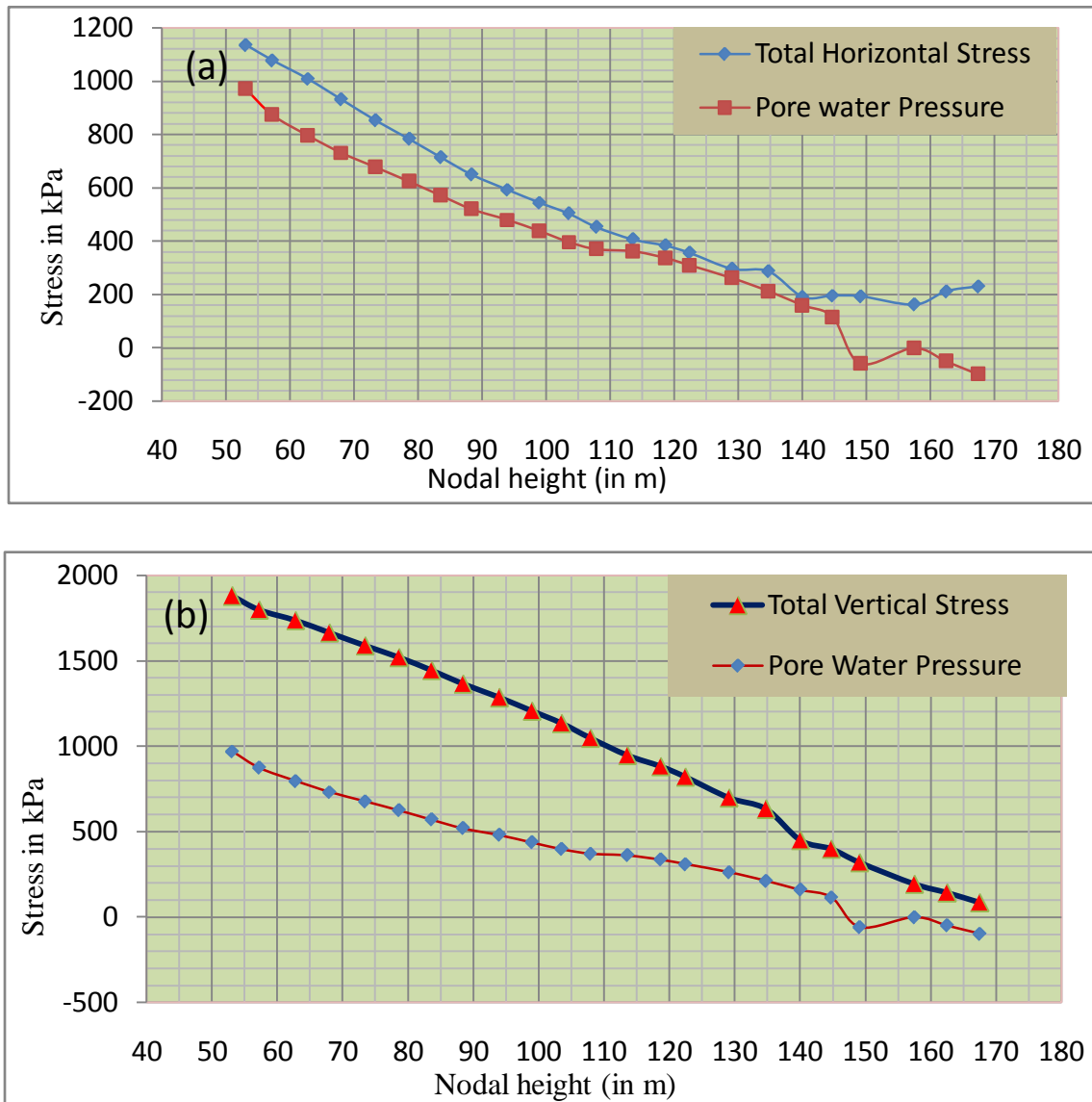


Figure 4.26: Evaluation of core cracking (a) horizontal cracking (b) vertical cracking

4.4 Dynamic Analysis

For the reason stated in section 2.10 the horizontal and vertical ATH data has been produced for middle Awash dam site location 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 ATH data have been scaled to PGA values of 0.3701g horizontal and 0.186g vertical corresponding for site specific MCE, and 0.2135g horizontal and 0.107g vertical corresponding to site specific OBE. Figure 4.27 and 4. 28 show the deconvolved ATH data used related MCE and OBE respectively for the project site.

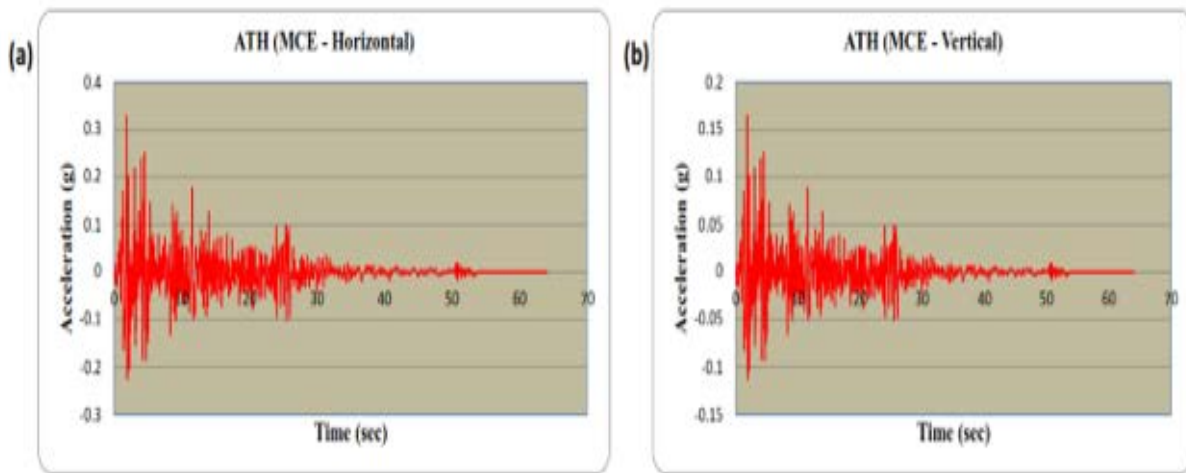


Figure 4.27: Deconvolved ATH of MCE a) horizontal b) vertical based on El Centro 1940

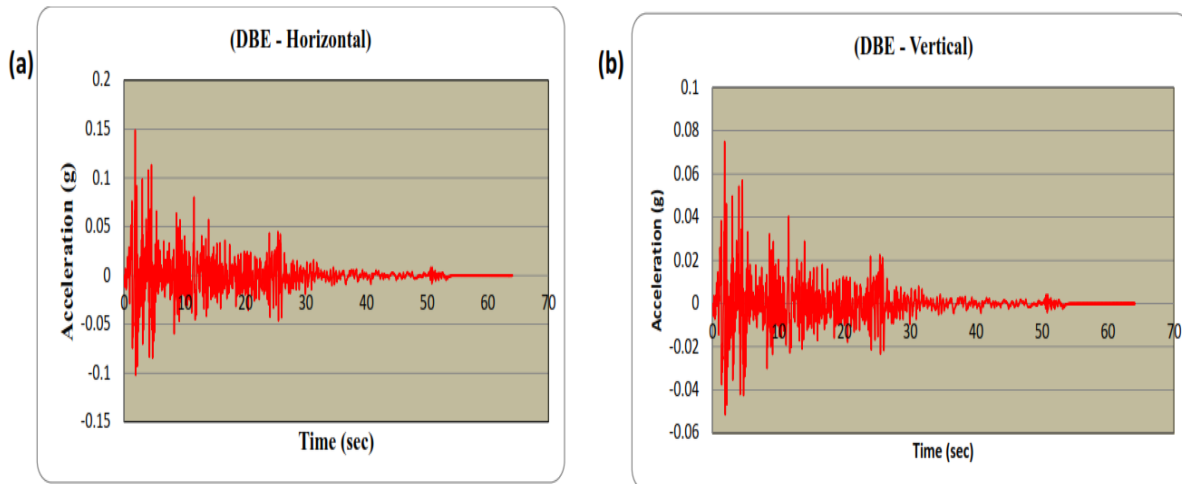


Figure 4.28: Deconvolved ATH of OBE a) horizontal b) vertical based on El Centro 1940

4.4.1 Post-earthquake stability

The stability of embankment slopes during and after the earth quake has been analyzed to evaluate the effect of the earthquake on stability base on records related to MCE and OBE. Based on the analysis the dam stays stable during earth quake shaking and after the earth quake has stopped.

- **Downstream slope stability**

The variation in factor of safety of the downstream slope during earthquake has been presented in figure 4.29 below and it varies between 1.225 to 2.431. The dynamic loading has sometimes stabilizing and also destabilizing effects on the embankment slope. Among the periods where the earthquake has destabilizing effect, a minimum factor of safety of 1.225 has been computed during earth quake (Fig. 4.30). The computed minimum factor of safety is greater than the minimum requirement suggested by Janson (1989) for unusual loading condition (i.e. > 1) which insures that the embankment is stable during earthquake shaking time. The minimum factor of safety that could be attained under the revised condition (i.e. inclined core) is even better than the minimum factor of safety for the original design (vertical clay core) ECDSWC is 1.05 (ECDSWC, 2016).

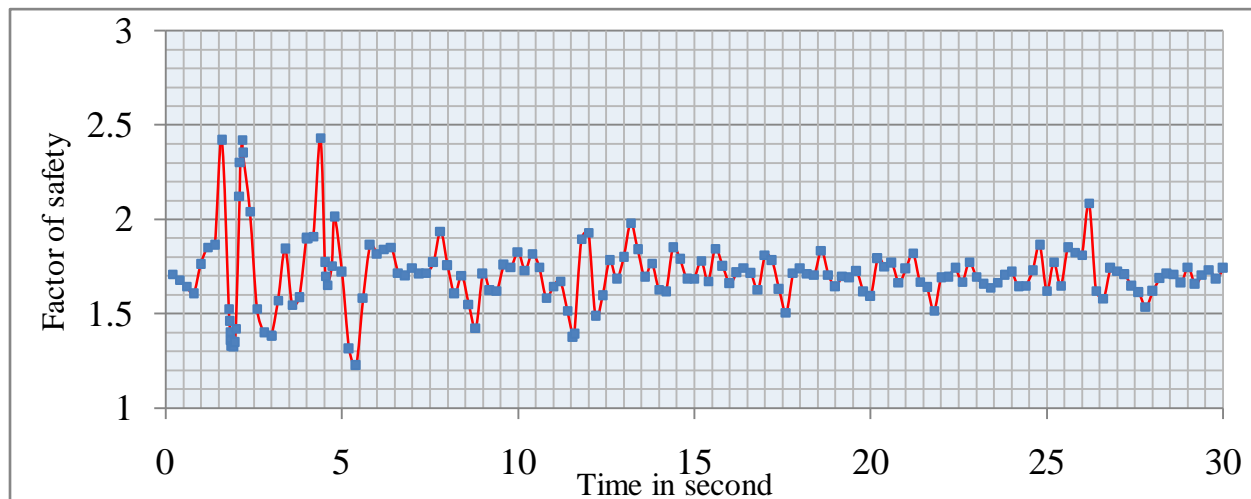


Figure 4.29: Variation of minimum factor of safety for downstream slope during earthquake

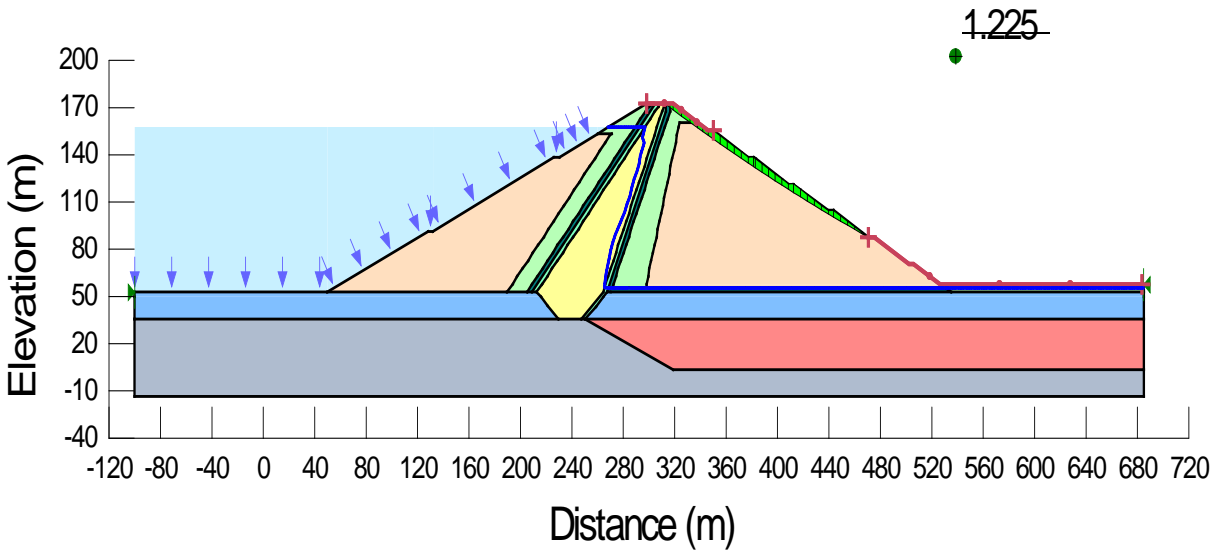


Figure 4.30: Minimum factor of safety of downstream slope during earth quake

- **Upstream slope stability**

Similarly, the upstream slope has also been analyzed during earth quake and a minimum factor of safety of 1.104 has been found as shown in figure 4.32. It is still greater than 1.0 so that the embankment is expected to be stable during earthquake. The variation of upstream slope during earthquake has been presented graphically below in figure 4.31.

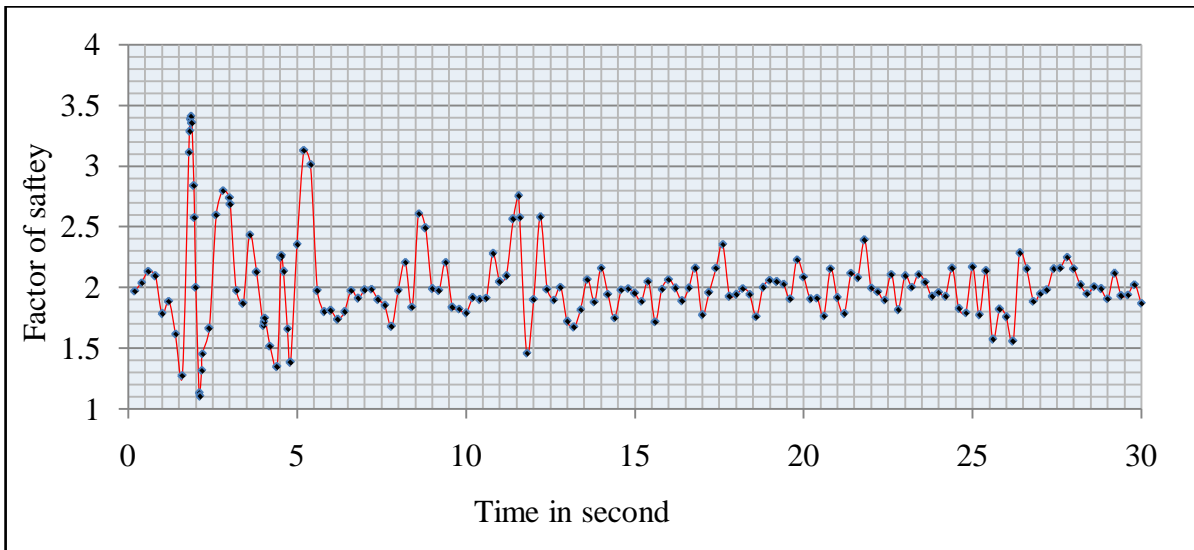


Figure 4.31: Variation of minimum factor of safety for upstream slope during earthquake

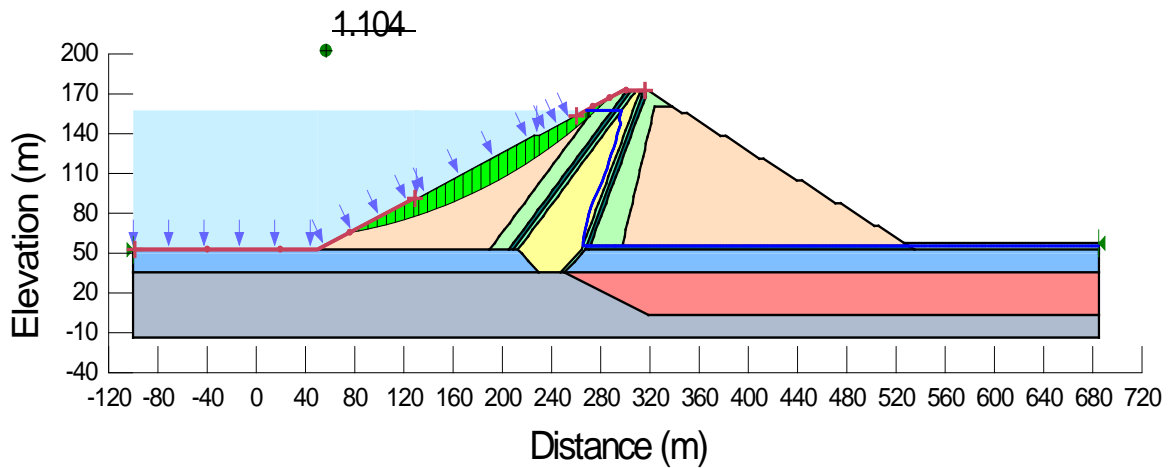


Figure 4.32: Minimum factor of safety of upstream slope during earthquake

Moreover, the stability of upstream and downstream embankment slope for earthquake related to OBE has been computed and a minimum factor of safety of 1.154 and 1.245 has been found for upstream and downstream respectively and shown in figure A. 8 and figure A. 9 of appendix A.

4.4.2 Permanent deformation

The alternative design (rock fill dam with inclined clay core) after the earthquake shaking has been examined to estimate the permanent deformation that the dam experience. Based on the analysis the permanent deformation has been computed to be 0.803 m (Fig. 4.33). And the computed deformation is much less than the freeboard supposed to be provided (5 m above from maximum water level ECDSWC, (2016) so that the serviceability of the embankment will not come to question after earth quake.

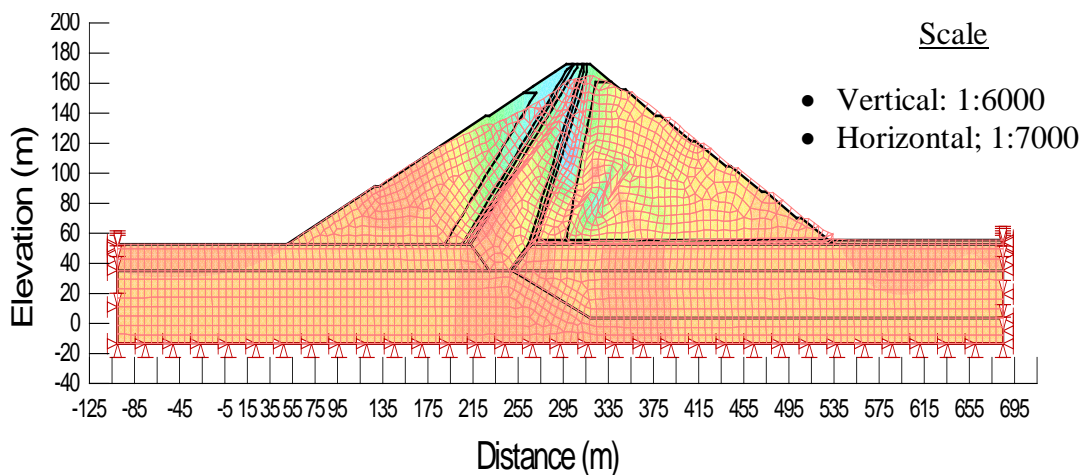


Figure 4.33: Exaggerated deformation at a 15 times scale of permanent deformation

4.4.3 Liquefaction analysis

Liquefaction of saturated granular soils has been the cause of most geotechnical hazards during earthquakes. Development of excess pore pressures in saturated soils when subjected to cyclic loading caused liquefaction. As it has been explained in section 2.10.1 soils are susceptible for liquefaction based on historic criteria, geologic criteria, compositional criteria and state condition of the soil.

Tsuchida, (1970) summarized the results of grain size analyses performed on a number of alluvial and diluvial soils that were known to have liquefied or not to have liquefied during earthquakes. He proposed a gradation envelope for liquefiable and non-liquefiable as shown in Figure 4.34. Based on the proposed curve the soil with grain size that lies between the two inner curves represent silty sand and sand termed as highly liquefiable soil and a soil with a gradation curve falling in the zones between the outer and inner curves is less likely to liquefy. However, case histories indicating that liquefaction have occurred in loose gravelly soils too although their gradation falls outside the curve and during severe ground shaking (Ishihara, 1985).

Based on the geological investigation report of middle Awash multipurpose dam WWDSE (2015) eight test pits were excavated in the terrace deposit (right abutment) to investigate the geotechnical properties of the older alluvium foundation shown in figure B. 1(appendix B) to use it as general foundation.

Most of the test pits located in the proximity of the active river channel was excavated to a depth of 3 m using a 20-ton excavator and the alluvium refuse to be excavated beyond 3 m depth. From this we can conclude that this older alluvium is consolidated. The grain size of the samples taken from the top 3 m has been analyzed and 90% of the grain size lies in liquefiable zone suggested by Tsuchida (1970) as shown in figure 4.35 and this makes the top 3 m of the older alluvial found in the foundation of middle Awash multipurpose dam is suspected for liquefaction.

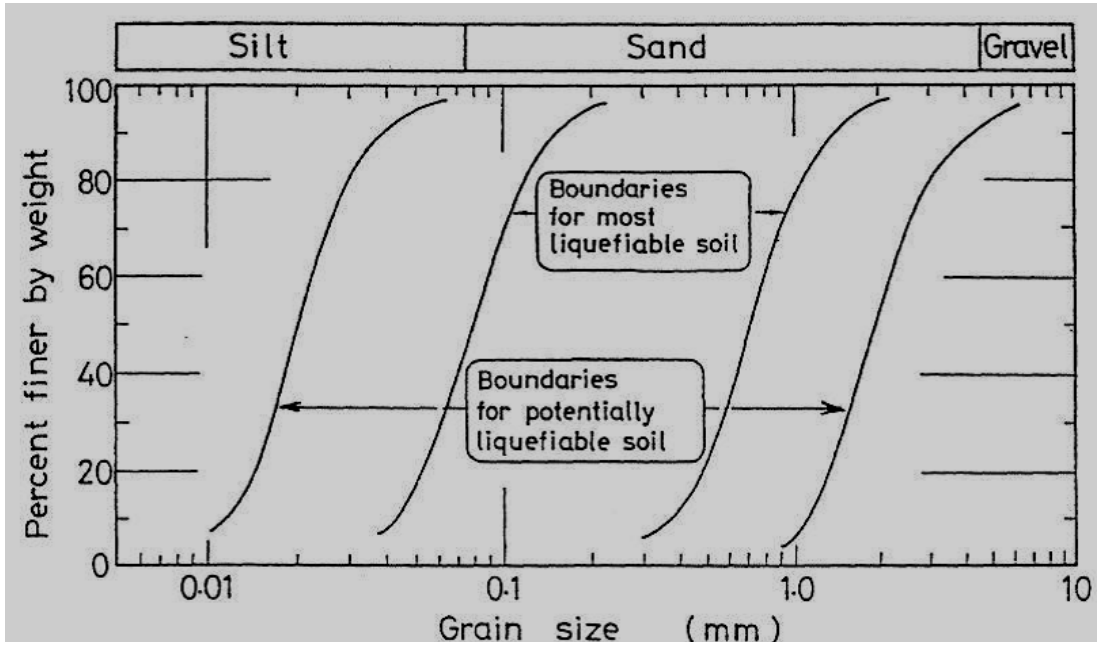


Figure 4.34: Limits of particle size curves separating liquefiable and non-liquefiable soils
Tsuchida, (1970)

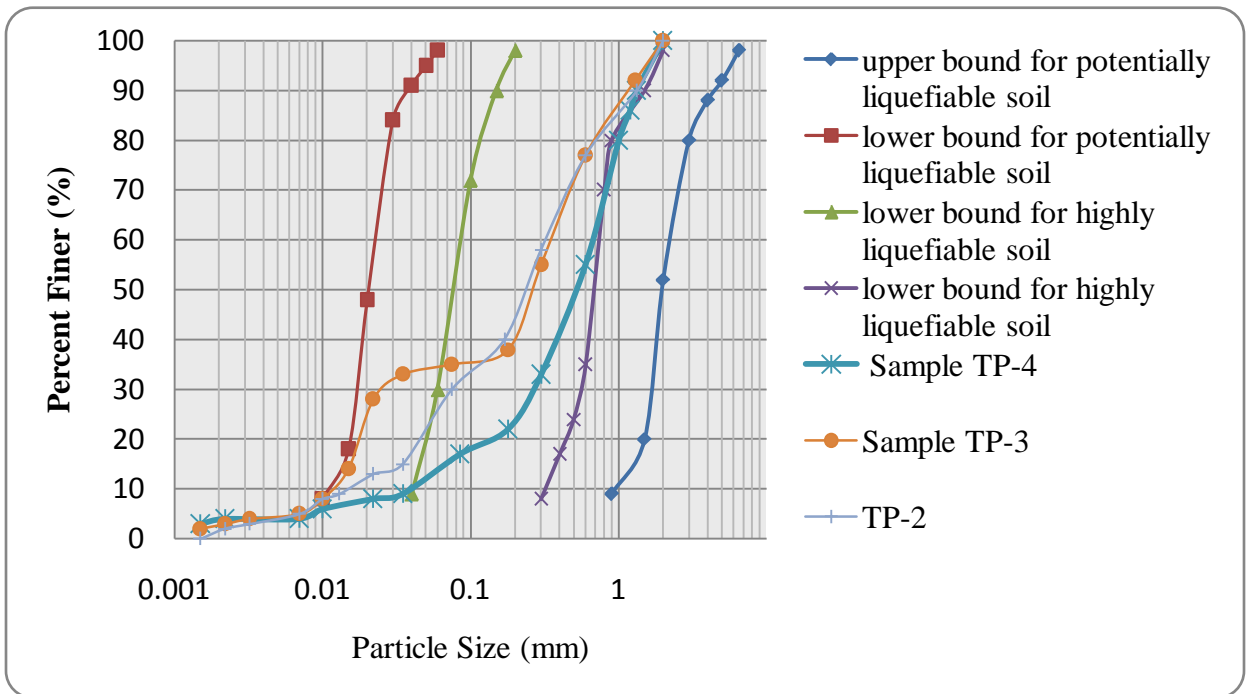


Figure 4.35: Grain size distribution of samples from top 3m old alluvial in boundaries of liquefiable soils

Apart from majority of the gradation of the old alluvial deposit fall in potentially liquefiable zone subjected by Tsuchida (1970), the dam is supposed to be constructed in seismic prone area and the top 3m exist in loose state that can easily be excavated during field investigation. Thus, all these reasons lead to the conclusion that there is high risk of liquefaction susceptibility for the top 3m thick old alluvial during dynamic loading.

However, in this study it is not found necessary to incorporate the liquefaction analysis for this particular zone because it is normal procedure that any uncertain foundation material found especially on the surface has to be excavated and removed to minimize any risk regarding to dam stability.

4.5 Economical Comparison

Economical comparison of the major activities of Middle Awash multipurpose dam for the original design and for the alternative design has been estimated. The revised design with an inclined clay core geometry found to be economical over the vertical core geometry as the volume of the clay core and inner shell fill material decreases. In addition, it is become possible that special treatment work recommended in the core foundation (fault breccias) has been abandoned due to the revision of the core arrangement. The quantity estimation of major activities for both design alternatives has been shown in Table B.1 and Table B.2 of Appendix B.

Based on the quantity estimated, the volume of rock fill zone has brought an additional cost of 308,007,380.04 birr however, the cost of impervious material, inner shell and special treatment works on core foundation could be reduced by an amount of 1,132,332,552.57 birr. From that the newly revised design (inclined clay core) has brought an advantage of saving 824,325,172.53 birr on the project as the detail has been shown in Table 4.1 below.

Table 4.1: Variation of activities due to design change

Activity No.	Description	Unit	Rate (in birr)	Quantity Variation		Amount in birr
				Addition	Omission	
1	Foundation treatment with jet grouting					
1.1	Mobilization and demobilization	LS	10,170,000		-1	(10,170,000)
1.2	Full scale trial test	LS	2,825,000		-1	(2,825,000)
1.3	Jet grouting treatment	m ³	10,170		-87500	(889,875,000)
2	Embankment work					
2.1	Compacted Rock fill	m ³	466.67	660011.1		308,007,380.04
2.2	Compacted inner shell	m ³	178.32		-205936.8	(36,722,646.61)
2.3	Compacted clay core	m ³	213.2		-904033.3	(192,739,905.96)

Net 824,325,172.53

Regarding to construction difficulty and workmanship issues, constructing embankment dam with include clay core does not demand any different skill than the vertical core. In has to also be noted that although the clay core is designed to be vertical but the upstream filter materials and finer shell has similar geometrical arrangement as the alternative design proposed in this thesis. In addition, there is an experience construction inclined clay core embankment dams on the coffer dam of Tendaho project WWDSE (2005) so that construction and workmanship will not be as such a big issue.

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From static analysis, based on the analysis conducted on middle Awash multipurpose dam by revising the core geometry from vertical to inclined the following conclusions has been made

- The quantity of seepage through dam body and dam foundation is found to be tolerable as compare to standards and the quantity of flux computed for inclined core without special treatment on the fault breccia is less than the quantity computed for vertical core
- The downstream slope has been found stable for all loading conditions i.e. steady state condition; end of construction and during construction. The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and the factor of safety of downstream slope during steady state computed, 1.720 is even better and improve than the one computed for vertical core, 1.63.
- Similarly, the upstream slope has been found stable for all loading conditions i.e sudden drawdown; end of construction and during construction. The factor of safety for all loading condition satisfies the minimum requirement suggested by standard and guideline.
- The static deformation analysis computed for the alternative design shows that the vertical and lateral deformation that the dam may subjected is within tolerable limit. The maximum static deformation computed 0.9 m vertically and 0.3 m horizontal when the water in the reservoir is at maximum pool level and the deformation for both vertically and horizontally are within the allowable limit.
- The alternative design has been analyzed for the possibility of core cracking and hydraulic fracturing however the inclined core has been found safe so that there is no possibility of core cracking and hydraulic fracturing

Once the alternative design had been found safe stable for all requirements during static loading conditions the same cross section has been used for dynamic analysis and the following conclusions has been made from dynamic analysis result.

- The upstream and the downstream slope of the embankment have been found stable during and after the earth quake shaking. The minimum factor of safety computed for alternative design during earth quake shaking is 1.225 which is better than the respective result computed for vertical clay core, 1.05. The minimum factor of safety computed for upstream slope during shaking time, 1.104 is almost equivalent to the vertical core arrangement, 1.15.
- The permanent deformation computed after the earthquake is 0.803 m which is less than the freeboard provided so that the serviceability of the dam during and after the earthquake shaking will not be affected.
- Liquefaction is expected for the top 3 m of old alluvial deposit for both design scenarios however this thickness will be excavated and there will be no risk of liquefaction.

Moreover, based on the economic analysis examined on the main activities of the project the alternative design (i.e. inclined core geometry) has a significant advantage over vertical core for the case of Middle Awash multipurpose dam.

5.2 Recommendation

From the analysis conducted on middle Awash project the following recommendation has been given

- During the design stage of any dam project it is recommended to consider the possible types of dam alternatives to come up with the most economical and structurally stable design
- Acknowledging all the designs which has been made so far in the country, the author of this paper recommends consideration of embankment dam with an inclined as a possible alternative under such specific condition which could minimize construction cost and reduce foundation treatment complications
- The top 3 m of old alluvial is susceptible for liquefaction so that it is advisable to excavate and remove this layer to minimize risk of liquefaction failure
- Apart from the jet grouting conventional types of grouting works (i.e. consolidation and curtain) is recommended on the dam foundation to seal joints and fractures so as able to minimize seepage and foundation deformation

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7 APPENDIXES

7.1 Appendix A

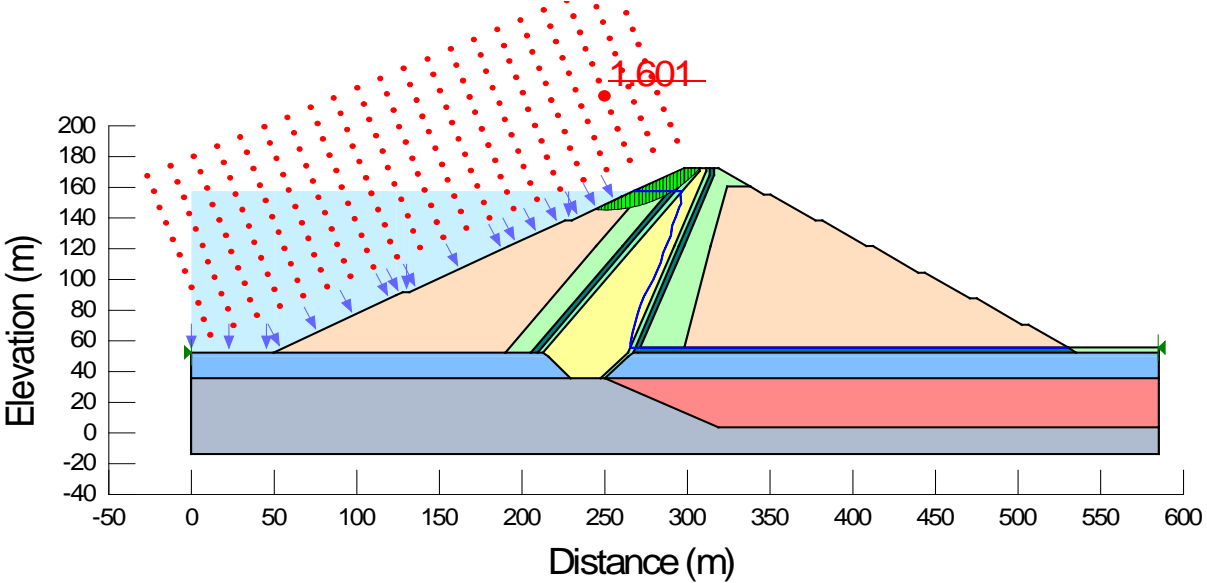


Figure A. 1: Minimum factor of safety of upstream slope during steady state

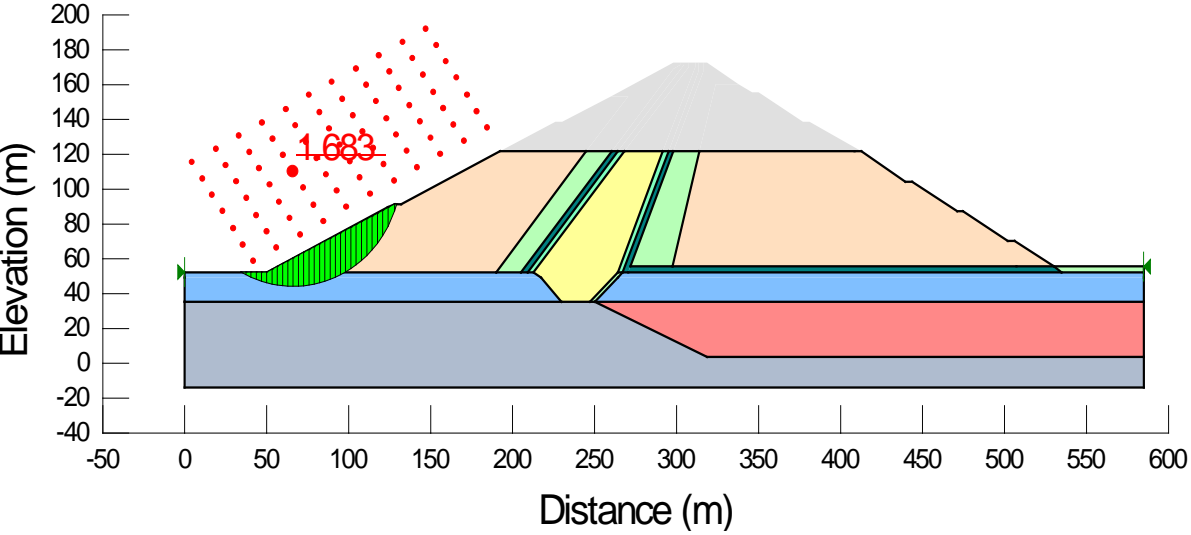


Figure A. 2: Factor of safety of upstream slope during first stage construction

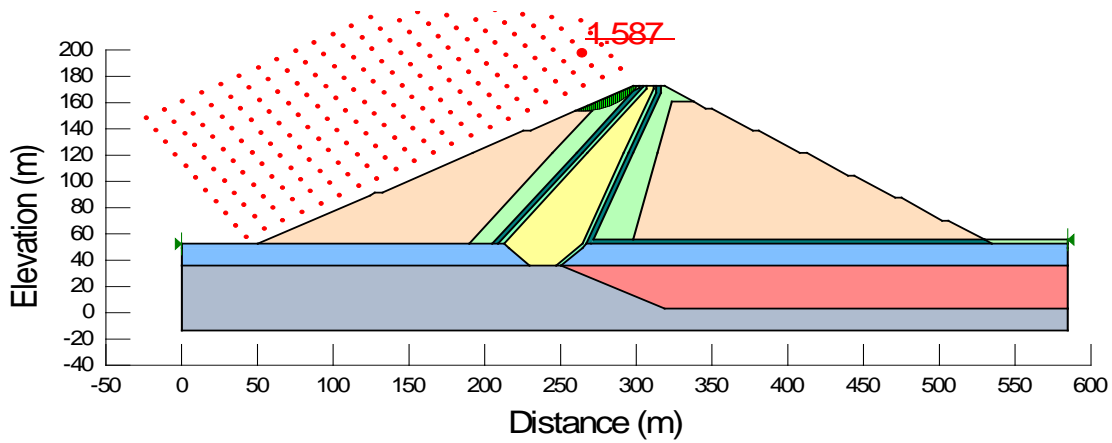


Figure A. 3: Factor of safety of upstream slope during second stage construction

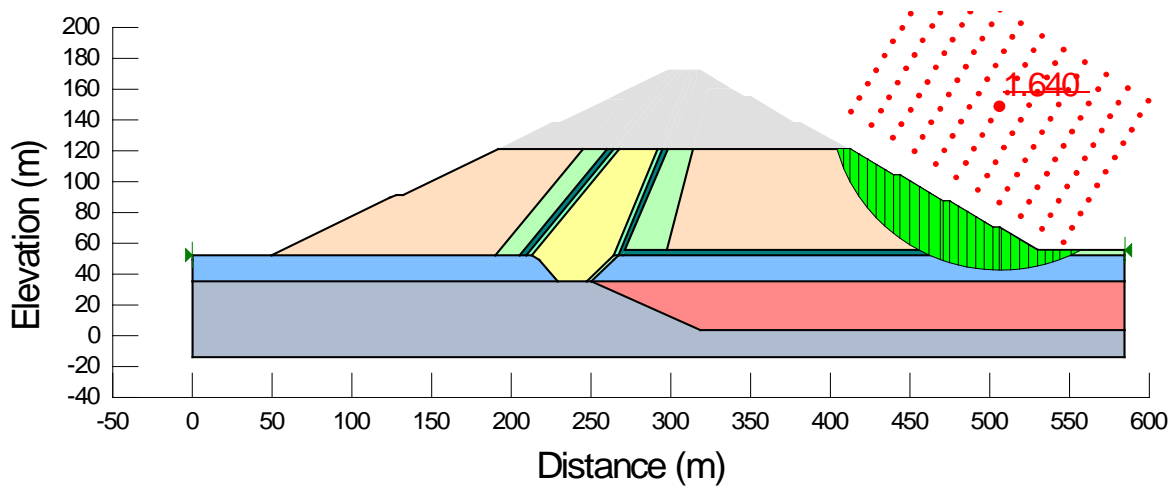


Figure A. 4: Factor of safety of downstream slope during first stage construction

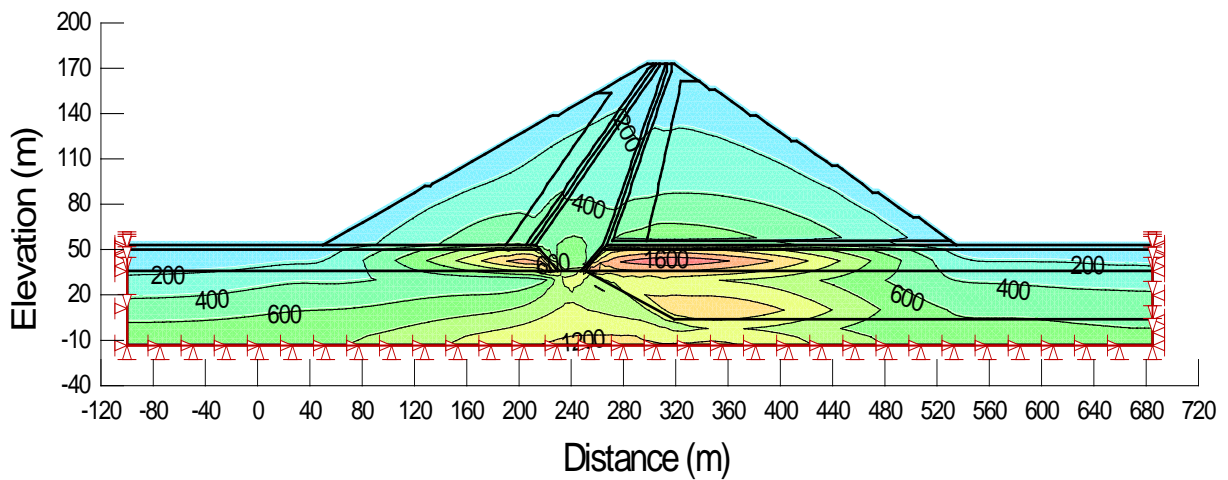


Figure A. 5: Contours of minimum total stress at end of construction for alternative design

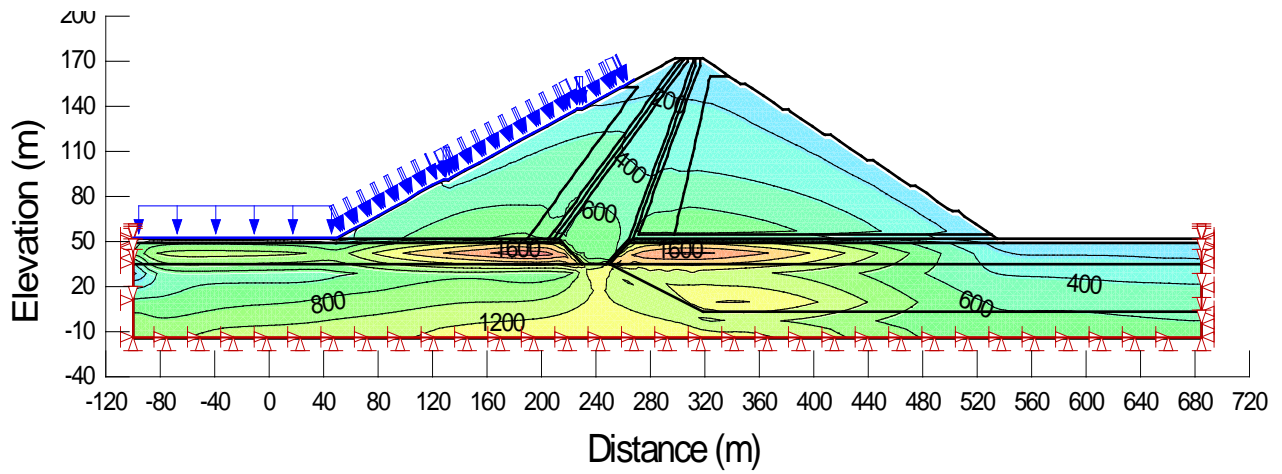


Figure A. 6: Contours of minimum total stress for alternative design at NPL

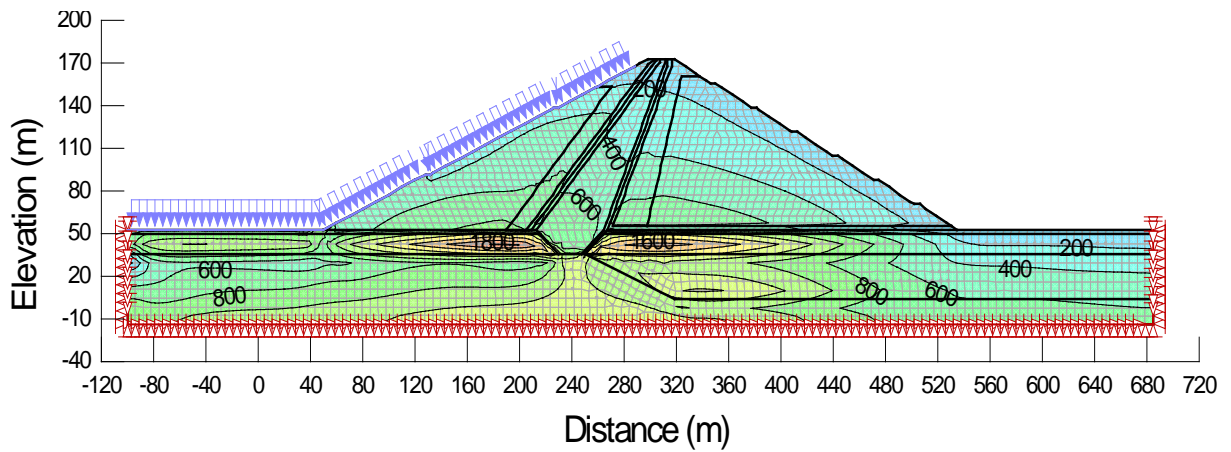


Figure A. 7: Contours of minimum total stress for alternative design at MWL

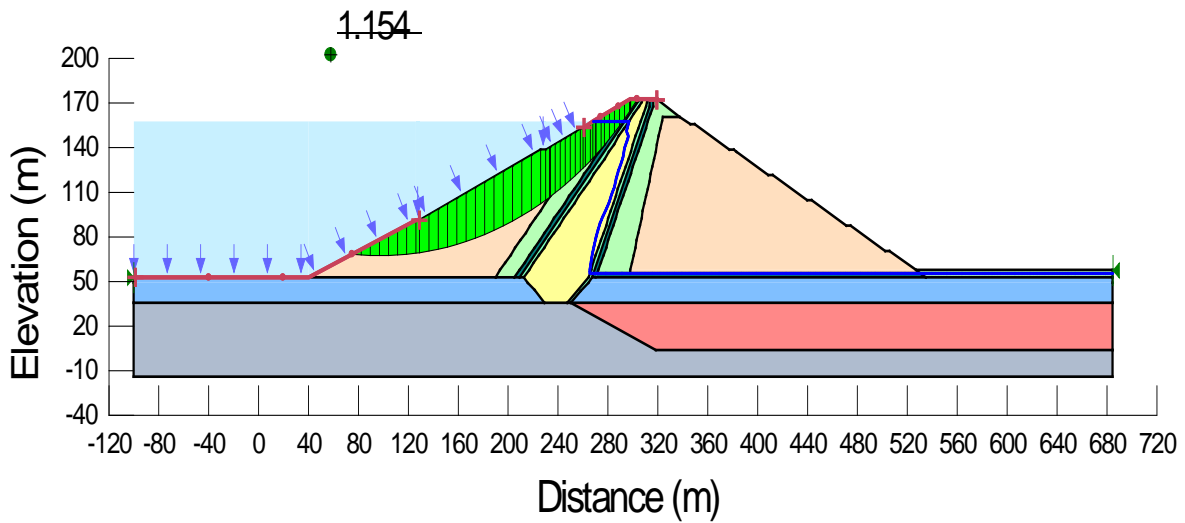


Figure A. 8: Minimum factor of safety for upstream slope for earthquake corresponds to OBE

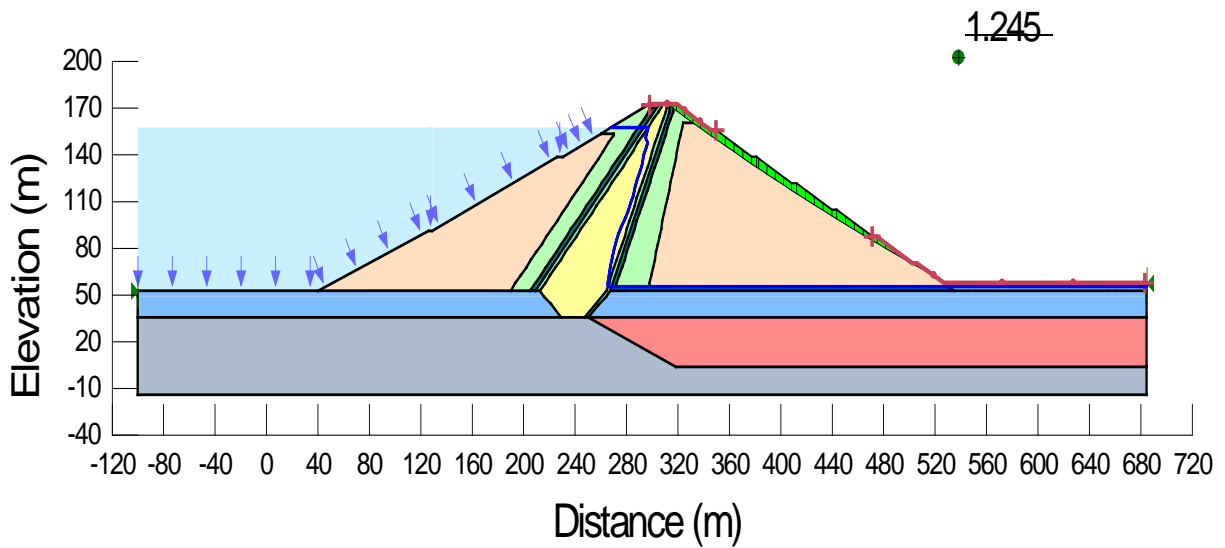


Figure A. 9: Minimum factor of safety for downstream slope for earthquake corresponds to OBE

7.2 Appendix B

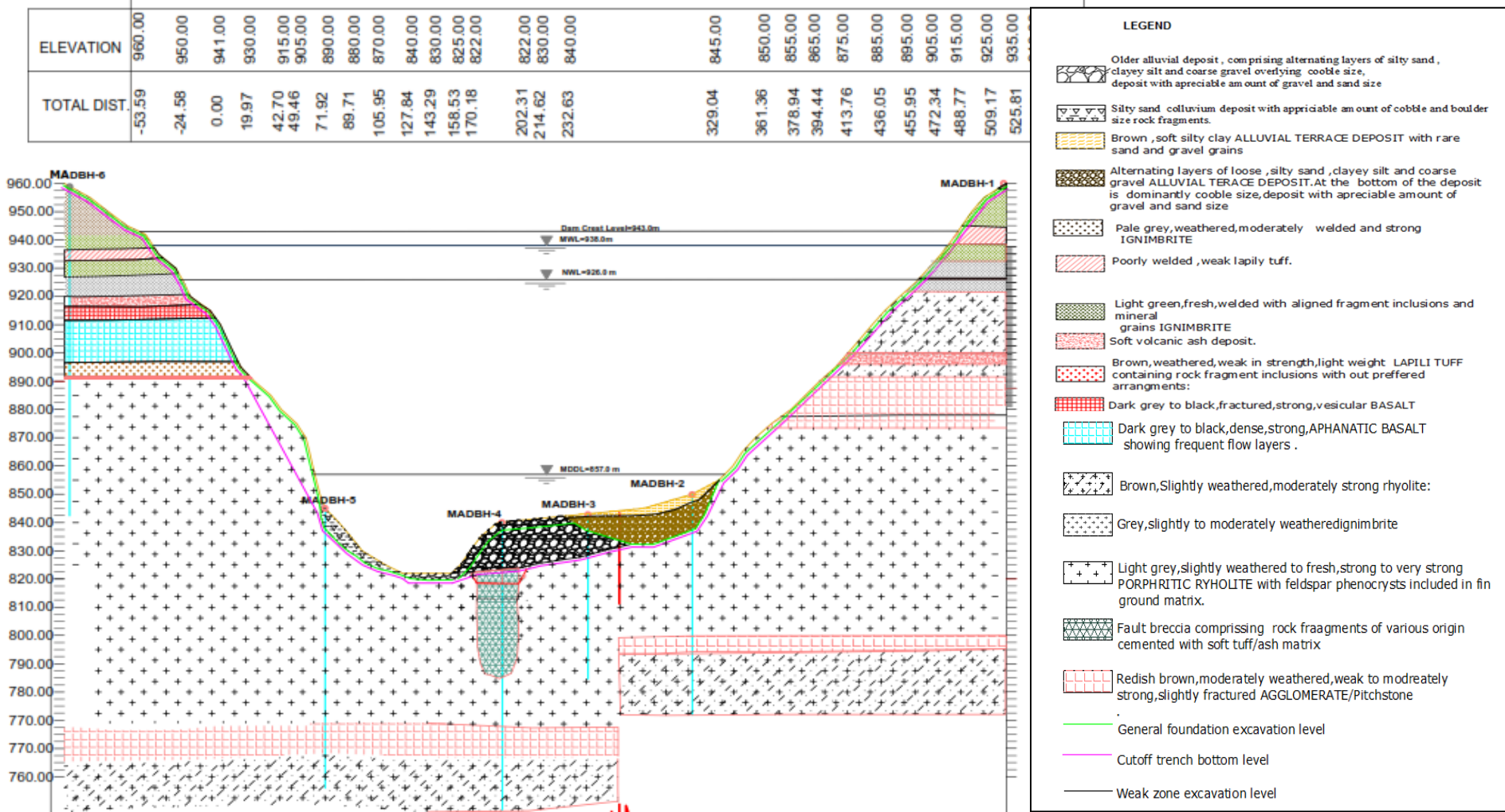


Fig. B. 1: Geological section of middle Awash multipurpose dam (WWDSE, 2016)

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

1	2	3	4	5	6	7	8	9	10	11
Chainage	Partial distance	Inner shell			Clay core			Rock fill		
		Area	Avg. area	Volume	Area	Avg. area	Volume	Area	Avg. area	Volume
			$(3_i+3_{i+1})*0.5$	(2_i*3_i)		$(6_i+6_{i+1})*0.5$	(2_i*7_i)		$(9_i+9_{i+1})*0.5$	(2_i*10_i)
0	0	0	0	0	0	0	0	0	0	0
20	20	458.858	229.429	4588.58	134.402	67.201	1344.02	192.02	96.01	1920.2
40	20	426.139	442.4985	8849.97	168.686	151.544	3030.88	350.363	271.1915	5423.83
60	20	854.394	640.2665	12805.33	246.567	207.6265	4152.53	1327.431	838.897	16777.94
80	20	1323.132	1088.763	21775.26	501.423	373.995	7479.9	2837.856	2082.6435	41652.87
100	20	1761.526	1542.329	30846.58	996.615	749.019	14980.38	6010.388	4424.122	88482.44
120	20	1763.486	1762.506	35250.12	995.797	996.206	19924.12	5961.827	5986.1075	119722.15
140	20	3556.477	2659.9815	53199.63	2668.4018	1832.0994	36641.988	15816.369	10889.098	217781.96
160	20	3674.502	3615.4895	72309.79	2918.899	2793.6504	55873.008	16837.762	16327.0655	326541.31
180	20	3949.271	3811.8865	76237.73	3275.33	3097.1145	61942.29	19660.211	18248.9865	364979.73
200	20	3917.553	3933.412	78668.24	3878.635	3576.9825	71539.65	20361.237	20010.724	400214.48
220	20	3901.5	3909.5265	78190.53	3878.635	3878.635	77572.7	20065.661	20213.449	404268.98
240	20	3556.477	3728.9885	74579.77	2679.1	3278.8675	65577.35	15461.643	17763.652	355273.04
260	20	3738.956	3647.7165	72954.33	2912.934	2796.017	55920.34	17006.736	16234.1895	324683.79
280	20	3908.563	3823.7595	76475.19	3405.327	3159.1305	63182.61	20248.086	18627.411	372548.22
300	20	3342.607	3625.585	72511.7	2524.142	2964.7345	59294.69	16182.852	18215.469	364309.38
320	20	3439.32	3390.9635	67819.27	2675.67	2599.906	51998.12	20752.678	18467.765	369355.3
340	20	3394.444	3416.882	68337.64	3878.635	3277.1525	65543.05	17382.179	19067.4285	381348.57
360	20	3058.258	3226.351	64527.02	2225.507	3052.071	61041.42	18580.766	17981.4725	359629.45
380	20	2372.676	2715.467	54309.34	1541.019	1883.263	37665.26	10011.365	14296.0655	285921.31
400	20	1864.627	2118.6515	42373.03	1131.468	1336.2435	26724.87	7432.226	8721.7955	174435.91
420	20	1864.728	1864.6775	37293.55	1131.468	1131.468	22629.36	7449.674	7440.95	148819
440	20	1541.982	1703.355	34067.1	858.904	995.186	19903.72	5533.56	6491.617	129832.34
460	20	1202.185	1372.0835	27441.67	585.081	721.9925	14439.85	3263.22	4398.39	87967.8
480	20	900.174	1051.1795	21023.59	338.063	461.572	9231.44	1628.426	2445.823	48916.46
500	20	657.971	779.0725	15581.45	193.053	265.558	5311.16	499.382	1063.904	21278.08
520	20	341.631	499.801	9996.02	73.365	133.209	2664.18	0	249.691	4993.82
Total volume				1,212,012.43	915,608.89			5,417,078.36		

Table B. 2: Work volume estimated for major activities of the original design

1	2	3	4	5	6	7	8	9	10	11		
Chaiage	Partial distance	Inner shell			Clay core			Rock fill				
		Area	Avg. area	Volume	Area	Avg. area	Volume	Area	Avg. area	Volume		
			$(3_i+3_{i+1})*0.5$	(2_i*3_i)		$(6_i+6_{i+1})*0.5$	(2_i*7_i)		$(9_i+9_{i+1})*0.5$	(2_i*10_i)		
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
20	20	84.6	42.3	846.4	134.6	67.3	1346.3	537.4	268.7	5374.3		
40	20	48.7	66.7	1333.0	264.6	199.6	3992.1	680.6	609.0	12180.4		
60	20	385.6	217.1	4342.8	657.5	461.0	9220.8	1406.7	1043.7	20873.3		
80	20	785.2	585.4	11708.3	1287.2	972.3	19446.5	2732.4	2069.5	41390.8		
100	20	1766.4	1275.8	25515.7	2339.5	1813.3	36266.1	5221.8	3977.1	79541.9		
120	20	3353.6	2560.0	51199.6	4206.3	3272.9	65457.3	10112.2	7667.0	153340.2		
140	20	4359.1	3856.3	77127.0	5408.4	4807.3	96146.5	12480.6	11296.4	225928.4		
160	20	4606.7	4482.9	89657.5	5867.3	5637.8	112756.3	13732.8	13106.7	262134.1		
180	20	5229.6	4918.1	98362.7	6499.9	6183.6	123671.9	15935.0	14833.9	296677.2		
200	20	5295.5	5262.5	105250.9	6875.1	6687.5	133750.8	17229.2	16582.1	331641.0		
220	20	5050.0	5172.8	103455.2	6258.7	6566.9	131338.7	17586.1	17407.6	348152.6		
240	20	4857.0	4953.5	99070.5	6323.0	6290.8	125816.9	16759.9	17173.0	343460.3		
260	20	4857.0	4857.0	97140.0	6323.0	6323.0	126459.3	16759.9	16759.9	335198.5		
280	20	4045.8	4451.4	89028.0	5862.1	6092.6	121851.1	13692.7	15226.3	304526.4		
300	20	4070.0	4057.9	81157.7	5664.9	5763.5	115270.5	13680.8	13686.8	273735.1		
320	20	4265.8	4167.9	83357.5	5141.7	5403.3	108066.4	14727.6	14204.2	284083.5		
340	20	4293.7	4279.7	85594.8	5209.2	5175.4	103509.0	14621.5	14674.5	293490.7		
360	20	3814.2	4054.0	81079.1	4702.8	4956.0	99119.4	12727.4	13674.5	273489.3		
380	20	2532.3	3173.3	63465.2	3405.8	4054.3	81085.5	8506.9	10617.1	212342.8		
400	20	2405.4	2468.9	49377.3	2796.0	3100.9	62017.3	8621.9	8564.4	171287.3		
420	20	1846.4	2125.9	42518.0	2147.2	2471.6	49431.5	7524.3	8073.1	161461.6		
440	20	1344.1	1595.2	31904.6	1549.7	1848.5	36969.1	5675.8	6600.0	132001.0		
460	20	874.2	1109.1	22182.4	1051.7	1300.7	26014.6	3515.2	4595.5	91910.1		
480	20	495.9	685.0	13700.3	630.9	841.3	16826.4	2048.6	2781.9	55637.7		
500	20	220.6	358.2	7164.2	316.8	473.9	9477.3	1088.3	1568.5	31369.1		
520	20	20.5	120.5	2410.5	116.6	216.7	4334.8	495.6	792.0	15839.7		
		Total volume			1,417,949.21				1,819,642.22			4,757,067.26