EVALUATION OF PROPOSED EMBANKMENT DAM FOR DODOTA IRRIGATION PROJECT

MSc. THESIS

BY

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As members of the Examining Board of the Final MSc Open Defense, we certify that we have read and evaluated the thesis prepared by **Dawud Manza** entitled **Evaluation of Proposed Embankment Dam for Dodota Irrigation Project** and recommend that it be accepted as fulfilling the thesis requirement for the **degree of Master's** with specialization in **Dam Engineering** .

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To My Family

DECLARATION

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

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LIST OF ABBREVIATIONS

AC	asphalt concrete
ACCD	Asphalt concrete core rock fill dam
Av.area	Average area
CC	Clay core
D/S	Down Stream
E	Young modulus
ECRD	Earth core rock fill dam
EL	Elevation
FEM	Finite Element Modeling
FAO	Food and Agriculture Organization
FLAC	Fast lagrangian analysis of continua
FS	Factor of Safety
G	Gravity
G	Shear Modules
H	Horizontal
ICOLD	International Commissioning for Large Dams
IS	Indian Standard
KOICA	Korea International corporation Agency
LEA	Limit Equilibrium Analysis
PGA	Peak ground acceleration
MCE	Maximum Credible Earthquake

MCM	Million cubic meters
MDE	Maximum Design Earthquake
MER	Main Ethiopian Rift
MOWIE	Ministry of Water, Irrigation and Electric
NFL	Normal Flood Level
OBE	Operating Basis Earthquake
OIDA	Oromia Irrigation Development Authority
RCC	Reinforced concrete core
PGA	Peak Ground Acceleration
Q	Discharge per unit length
SEE	Safety Evaluation Earthquake
SEEP/W	Finite Element Software for Ground Water (seepage)Analysis
SIGMA/W	Limit Equilibrium Software for Stress-Strain Analysis
SLOPE/W	Limit Equilibrium Software for Slope Stability Analysis
Tr	Return Period
USACE	United States Army Corps of Engineered
USBR	United States Bureau of Reclamation
U/S	Upstream
USSD	United States Society on Dams
V	Vertical
WWDSE	Water Works Design and Super Vision Enterprise
α h	Horizontal Seismic Coefficient
α v	Vertical Seismic Coefficient

ABSTRACT

Design and construction of embankment dam is increasing from to time in our country to help the utilization of water for multipurpose. Evaluation of propose embankment dam for Dodota Irrigation project as alternative design by introducing asphaltic concrete core or clay core vital form the stand point of safety, controlling seepage and very important structure in fault and Earthquake area. This study was aimed to evaluate a proposed embankment dam as alternative design and analysis for Dodota Irrigation Project. Address this objective proposing an embankment dam with an impermeable asphalt concrete core and analyzes it for seepage static and dynamic stability using Geo Studio 2012 numerical computer program. Based on computation the flux through the dam and foundation for asphaltic concrete case has been found to be 0.000059 m^3/s and the flux through the dam and foundation for asphaltic concrete case has been found to be 0. 0.001334 m^3/s. The factory of safety of the propose embankment dam for the alternative design of embankment at different construction stage and with different loading condition satisfied the minimum requirement of (USAC,2003). The stress deformation observed was much lower than the expected bearing capacity of the foundation rock. The static deformation analysis computed for the propose embankment dam shows the horizontal and vertical deformation that the dam may subject to were in the tolerable limit. The dynamic deformation also analysis computed for the propose embankment dam during the time of shaking the maximum vertical and horizontal deformations within allowable limit. Generally, application of asphaltic concrete core rick fill and rock fill clay core dam in the project can fulfill the basic requirement and minimum factor of safety under all loading condition. Over all analysis of the thesis aim is indicate the possibility of constructing dam for the Dodota Irrigation Project.

KEY WORDS: Asphaltic Concrete Core, Geo-studio 2012, Seepage analysis

Slope stability, Static analysis and Dynamic analysis

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 than15m, which is referred as a high dam. Among these dams constructed all over the world, more than 70% are of embankments dams (ICOLD, 2005). ICOLD divide dams into embankment dams about 70% of the total number, concrete dams about 28 % and masonry dams about 2% (ICOLD, 1988).

In Ethiopia currently, the Government laid a policy of agricultural leading industry, and has seen improvements in this perspective. The water resource of our country starts to manage by construction of dam such as Tendaho, Kessem, Wolkayite, Gelgal Gibe, and Genale Dawa and so on.

Assessment of irrigation potential area and water resources is very important to identify the available quantity of water and suitability of land for the appropriate irrigation development. A fuller use of land and water resources by the development of irrigation facilities could lead to substantial increases in food production. According to FAO 1976 report Ethiopia has an estimated 3.7 million hectares of irrigable land and only about 5% is currently developed. On the other hand, the Ministry of Water, Irrigation and Electric (MOWIE) report indicated that the estimated national irrigation potential is about 5.3 million hectare. From 560 irrigation sites have been identified on the major river basins. Ethiopia in general has very large irrigation potential for agriculture development. The availability of abundant water resource in the country, the facility of irrigation has so far been only minimal leaving agriculture to be mainly dependent on the unreliable rainfall (OIDA, 2015).

To minimize the drought problem and improve the life standard of the community for our country; most of irrigation project is being constructed or under construction by using dam. In Oromia regional state most of irrigation schemes are diverted water by weir structure. Most of the project faces a shortage of water during the dry season due to increased interest of the community from time to time. Such problems were happened some of the existing projects: Fantale, Marti and Tibila which are among the large irrigation project in the region (OIDA, 2017).

According to KOICA , (2015) geological report of Dodota Irrigation Project, the dam site is designed to be placed on fault area, seismic condition of the area and besides the location of borrow area identified for the impervious zone is a little bit far from the dam site(>10km). The clay is found at the area which preserved by the community so that getting clay is going to be expensive. It was told that because of such limiting factors instead of a constructing dam, storage structure a diversion head work structure was recommended. While there are some alternatives like trying some dam type and with alternative construction materials that possibly reduce or even solve such mentioned failure risks it may not be wise to undermine them.

Because of these other material type must be used for the core and general dam zoning design with safety analysis is required. The option of embankment dam with asphaltic core design must be done assuring safety and less cost. Asphaltic core are mainly used in areas where natural impermeable material of sufficient quality or quantity are not available. Since the project area is in a seismic region and asphalt concrete core has viscoelastic-plastic properties can be tailored to local conditions and climate which makes asphalt core dams especially suited to seismically active areas.

Over the world, dams with asphalt concrete cores exist in different countries and these dams have good performance with respect seepage, which is challenging problem for any other rock-fills and earthen dam. Among asphalt concrete cores dams, the highest dam so far documented is Storglomvatn dam in Norway with height of 125 m, Langavatn dam in Norway is the least within height of 26 m next to Eberlaste Dam of Austria with height of 28 m (ICOLD, 2015).

Embankment dams are almost built with all types of the soils with central clay core of different shape and thickness, and the dam can be adapted to broad range of foundation conditions. But there were recognized benefits to use embankment dams with asphalt concrete core rather than clay core (Wang and Hoeg, 2011).

According to collaboration study result of three colleagues on two different dam section with clay core and asphalt core, they reported that, in the clay core dam the total seepage registered at maximum reservoir level is 42 liters per second. In the same case the total seepage registered at maximum reservoir level in the asphaltic core dam is 26 liter per second. The hydraulic gradient in clay core dam is equal to 0.68 and in the asphaltic core dam is equal to 0.29. The comparison of the results shows that the asphaltic core dam has the best performance against piping phenomenon with respect to clay core dam (Zomorodian et al, 2005)

1.2 Statement of the Problem

Dodota district is one of Oromia Regional state district which has a little mean yearly areal rainfall of 150 mm to 300 mm (KOICA, 2015). Due to the little rain falls the community of the district affected by drought. To solve this problem, Oromia Irrigation Development Authority (OIDA) proposes dam for

Dodota Irrigation Project in Keleta River with the sponsor of Korea International corporation Agency (KOICA).They recommended the diversion weir rather than constructing dam for Dodota Irrigation Project. As the recommended structure (diversion weir) relies on the base flow by its nature its supply capacity or the potential command area to be irrigated will be limited. There was an irrigable command area of 4000 hectare in the project site however with the recommended diversion head work structure only 420 hectare of the total will be irrigated (KOICA, 2015).

In general the proper and timely utilization of water resources remains one of the most vital contributions made to society. To apply the water resource in multi proposes area storing of water is very important. Dam is very important structure constructed to provide a storage reservoir for multipurpose function. Constructing dam highly important for developing country like our country Ethiopia in order to utilize high water resources potential for different function. In particular, in an area like Dodota where the annual rainfall is by far less than the minimum required to grow some cereal crops which are used to grown in the area the use of storage structure is the first and alternative to make use of the water resource of the basin.

In Ethiopia dams of different variety were designed and constructed in the past decades and others are under construction. Among those varieties of dam constructed so far in the country, embankment dam is the dominant types widely used. But, yet all of these dam has been constructed in such a way that, they utilize the same materials (clay) as the core, and as a result, these dam faces the problem of internal piping and seepage due to the fact that, weak strength of clay core in resisting differential settlement of embankment and foundation. The main problem of the site is the shortage of clay material and the clay material also far from dam site, the dam site is at earthquake area and fault plain.

This thesis also searched a solution of the above problem and indicating the possibility of constructing rock fill asphaltic concrete core for the project.

1.3 Research Question

The main questions of this study area:

- It that safe to construct an embankment dam with regards to seepage and stability problems for the Dodota irrigation project?
- Does the seismic loading affect the stability of an embankment in Dodota irrigation project?

1.4 Objective of the Study

1.4.1 General objective

The general objective of this study is to evaluate a proposed embankment dam as alternative design and analysis for Dodota Irrigation Project.

1.4.2 Specific objectives

The specific objectives of this particular study are.

- To investigate seepage analysis of propose embankment dam under steady state flow condition.
- To investigate the slope stability of an embankment dam during and at the end of construction and sudden drawdown condition at Dodota irrigation project.
- To investigate the stress deformation of an embankment dam under dynamic loading condition.

1.5 Scope of the Study

This study has been intended to evaluate propose embankment dam for Dodota Irrigation Project to indicating the possibility of constructing dam for the project as alternative option.

The intent of this study is to propose as alternative of embankment dam with asphaltic concrete core for the project. Thus, the study will include dam dimensioning for different zones, seepage modeling, and slope stability analysis for static and dynamic condition. Furthermore, this study will serve as a guideline for designers and professionals of this area in which construction material assessment, design, analysis and supervision of embankment works takes place.

1.6 Significance of the Study

The scope of this research will focus on design of Asphaltic concert core rock fill dam including dam dimensioning, zoning and fixing upstream and downstream slopes .In addition to this estimate the quantity of seepage through the body of the dam and foundation, static stability analysis under different loading condition, dynamical stability analysis using numerical model such as Geo-Studio 2012 were done. However, this study will not include hydrological analysis, design and analysis of other appurtenant structures.

2. LITERATURE REVIEW

2.1 Dam Type Selection

According to the standard manual provided by the International Commission on Large Dams (ICOLD, 2015), in which about 63 member countries are now associated, dams with the height of more than 15m are referred to as "high dams". To date about 14,000 high dams have been registered and more than 70 percent of these are embankment dams. A recent report on the construction of high dams has also noted that about 1,000 high dams constructed in recent two years, just 20 percent are concrete dams and the remainders are embankment dams.

There are factors, which must be considered in selection of dam type, such as topography, availability of construction materials, spillway requirement, diversion arrangements, experience of the contractor, geologic condition, etc. The availability of construction materials can limit the choice of dam type in economic wise, but provided the difference in cost is small, the choice between a concrete and an embankment dam is often influenced by previous and local practice and preference of the designer. The dam engineer is required to synthesize design solutions, which, without compromise on safety, represent the optimal balance between technical, economic and environmental considerations (Novak et al., 2007).

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).

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.

The presence or absence of construction material in the construction area has a significance influence on the type of the dam which has to be selected. If there is no earth material at economical distance it is normal to consider rock fill dam as a possible option and vice versa. The availability of critical construction material like cement, aggregate, sand and filter drainages also play significant role on the type of dam to be selected (Emiroglu, 2008).

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 dams type and high earth rock fill dam with artificial and natural sealing has been constructed all over the world (Emiroglu, 2008). Rogun 335 m high dam in Tajikistan, Manuel Torres 261 m high dam in Mexico, Tehri 261 m high dam in India, Oroville 230 m high dam in USA are among the biggest dam of this type across the world (Tancev, 2005).

Topography is also significant factor on the selection of dam as narrow gorges with U and V shapes are favorable for gravity or arch dams and wide and plain terrains are best for embankment dams (Stematiu, 2006).

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, 2015).

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 earth quake 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).

Embankment dam Constructed in Ethiopia such as Kessem and Tendaho dam projects are of good example for poor and complex geological conditions which take extended foundation treatment and Kessem and Tendaho dam projects located in center of rift valley near the Afar volcanic and earthquake centers. There also the study of multipurpose embankment dam at downstream of Awash River near to Metehara sugar factory which is Middle Awash Multipurpose Dam the same regional geological condition with Dodota Irrigation Project is also propose on poor and complex foundation with a height of 120m (WWDSE, 2016). All the three dam above are near to Afar depression and center to seismically active region with respect to the propose embankment dam for Dodota Irrigation project. In the country there also experience in constructing asphaltic concrete core is Wolkayt Asphaltic concrete core dam height of 136 m constructing under alluvial deposited (WWDSE, 2012).

2.2 Embankment Dams

The construction and deployment of embankment dam starts since the early civilization of mankind and some earthen dams with considerable heights, storage volume and crest level has been constructed before 530 B.C. Rock fill dams generally concedes more than a century. Today as that of the past, embankment dams continue to be the most common choice of dam type, principally   because its construction involves utilization of locally available material with minimum of stockpiling and processing cost (USBR, 1992).

Nowadays, embankment dams exist in excess of 300 meters high with volumes of many millions of cubic meters of fill. Development of soil mechanics, study of behavior of earth dams, and the development of better construction techniques have all been helpful in creating confidence to build higher dams with improved designs and more details that are ingenious. The result is that the highest dam in the world today is an earth dam. The highest earth fill and rock fill dams in the world are Roguni USSR (335 m) Nurek, USSR (300 m) Mica, Tehri India (260 m) Canada (244 m) and Oroville, USA (235 m). Thousands of embankment dams exceeding 20 meters in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction (USSD, 2011).

The material from excavating dam foundation, spillway, outlet works and other appurtenant structures would be used as soils, rock sand riprap and concrete aggregate (sand, gravel and crushed stone) for embankments. If suitable soils for an earth-fill dam can be found in nearby borrow pits, an earth dam may prove to be more economical. The availability of suitable rock may favor a rock-fill dam. The availability of suitable sand and gravel for concrete at a reasonable cost locally or on site is favorable to use for a concrete dam (Golze 1977; USBR 1987).

ICOLD (1996) defines an embankment dam as any dam constructed of natural excavated material or industrial waste material placed without the addition of binding materials other than those inherent in the natural material. In the past, and to some extent at present, embankment dams have been constructed from the most readily available materials such as loose rock, gravel, sand, silt, mine or industrial waste, rock flour and clay. The fill material is placed with sloping sides and with a length greater than its height. These dams are most suited in areas where the foundation material is of earth or sand, or where the materials for construction are so expensive that an embankment type of dam is more economical.

If suitable soils for an earth-fill dam can be found in nearby borrow pits, an earth dam may prove to be more economical. The availability of suitable rock may favor a rock-fill dam. The availability of suitable sand and gravel for concrete at a reasonable cost locally or on site is favorable to use for a concrete dam (Golze 1977, USBR, 1987).

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).

In general, the main problems associated with embankment dams are as follows ICOLD (2005):

- Seepage through the dam or through the foundation, causing pore water pressures in the fill and the foundation,
- Settlement of dam and/or foundation,

- Deformations due to internal and/or external stresses, and
- Slope stability (both upstream and downstream).

In fact the most dangerous process in embankment dams is piping (the progressive development of internal erosion by seepage, appearing downstream as a hole discharging water), which may be caused by cracking and fines entrainment. This may occur because of an inadequate downstream filter behind the core, or because the earth in a homogeneous dam does not possess the necessary healing properties. In many cases dam failure due to piping is reputed to be quick, but there have been cases where visible sand deposits at the toe of the embankment dam had shown that piping had already begun for some period of time (USBR, 2012).

2.3 Dam Design Alternatives

If suitable earth fill materials are available with in economic haul distance of the selected dam site, it is ideal to be selected for the construction of an embankment dam. If suitable earth fill is not available, the dam construct by rock fill material with appropriate impervious membrane. The concrete option fixed with a huge cost and foundation requirement. The embankment dam is possible construct on different foundation condition. For a given embankment dam project site, there are several different design options; a dam with: earth core; upstream facing of reinforced concrete or asphalt or synthetic geo membrane; or asphalt core (ICOLD, 2015).

Concrete faced rock-fill dam is the type of dam that has a dam body of either rock-fills or gravel fill that is compacted in layers and an anti-seepage system of concrete face slab. The concrete slab facing acts as an impervious layer while the rock fill body with a high permeability, gives support to the concrete face slab and the overall stability of the dam. The main advantage is utilization of less volume of rock fill consequently considerable economy is saved. Some of the disadvantages are design for leakage through open joints and tension cracks, large compression cracks can occur for considerable height in narrow valleys, needs competent foundation, and cannot provide storage during construction. As in the case of Dodota Irrigation Project dam site, the foundation condition shows that fresh and sound rock is located at a maximum depth of 9 m of which 2- 9m is over burden. Upstream faced concrete require removal of the foundation material from the dam site area and use of a non-compressible rock fill shell ICOLD (2011).

According to ICOLD, 2011, many places around the world, geo membranes are used as water barrier for temporary or permanent cofferdam and as a liner within main canals to reduce seepage losses. Some of the advantages of using such an impervious membrane: rock fill zone is unsaturated and downstream slope can be constructed with steeper slopes, plinth and grouting can take place independently, and membrane's flexibility to accommodate rock fills deformations. Whereas the following disadvantages play major role in their selection particularly in high dam; vulnerable to impacts, ice loads, sabotage, effects of weathering and aging, and this require partial or fully covered protective layer that increase cost, and cannot provide storage during construction.

Many research results showed that the asphalt concrete core in an embankment dam construction, especially in a seismic region can withstand very severe seismic shaking without cracking and loosing water tightness. In fact, the earthquake resistance of the dam will also depend on proper design and zoning of the embankment itself, considering the available fill materials and the foundation condition.

Embankment dams with impervious clay core induces lower stress on the foundation as compare to other ridged type of dams so that embankment dams are more preferable to be constructed on wide range of geological formation and foundation treatment has to be done to increase the bearing capacity as well as to minimize seepage through the foundation Conditions and the seismicity of the site (ICOLD, 2005).

Hoeg et al. (2007) presented the field performance observations for the Storglomvatn asphalt concrete core dam 125 m high and provided a general discussion of the relative merits of the different embankment dam design options. Different studies at several recent projects have shown the asphalt concrete core option to be very competitive both technically and economically.

Over the past few years fairly detailed cost comparisons have been made between asphalt concrete core dams and their alternatives at the design stage of projects. Several tenders have also opened for alternative bids. For the Urar dam, completed in 1997 in Norway, tenders were submitted for an RCC dam and a rock fill dam with asphalt core. When only considering contractor costs and additional spillway expenses, the asphalt core alternative turned out to be approximately 10% cheaper than the RCC option (ICOLD, 2015).

For the Greater Ceres dam, completed in 1998 in South Africa, three alternatives were compared at the design stage: design stage: RCC, concrete faced and asphalt concrete core dams. The asphalt concrete core was chosen due to cost and because the dam was located in an earthquake region on a poor sandstone foundation (ICOLD, 2015).

Asphaltic concrete core more important due to flexible nature, the capability of adjusting itself to the deformations of the embankment fill is very high during construction, impounding, operation, and earthquake loading. Weathering and aging are almost negligible as the core is imbedded inside the embankment and also the asphalt concrete is protected from the environment and is subjected to almost constant temperature after the end of dam construction and reservoir impounding (Høeg, 1997). Kramer (1988) reported that “the rock fill dam with an asphalt-diaphragm is an appropriate dam type for the very highest future dams.”

Several dams of this type are currently under construction around the world. The recently built 125 m high, Yele Dam is founded on a deep and complex alluvial foundation in a high intensity seismic region (Wang 2008). Based on their experience, Chinese engineers have now designed and are building a 170-m-high asphalt concrete core rock fill dam (Quxue dam).

At the Eberlaste dam, completed in Austria in 1968, an embankment dam with an asphalt core was built on top of a deep compressible and inhomogeneous alluvial deposit. The 28m high dam and the flexible asphalt core were constructed on top of a cut-off slurry wall. During construction the foundations settled about 2.2m in the middle of the valley and in the following two years secondary settlements of 20cm were measured.

According to Høeg (1997) modern machinery for placing and compacting asphalt concrete with 6-7% bitumen content, and simultaneous placement of the transition zones, is the most economical way to ensure fast and reliable construction. Furthermore, this technique allows the best quality assurance and control of the asphalt core in situ. As demonstrated on the Svartisen project in northern Norway (1995-1997), progress on asphalt core dam, the dam is not affected by wet conditions when the appropriate equipment is used and necessary measures are taken. The core can be constructed in weather conditions suitable for construction of the rest of the embankment. Contractors’ standing and stoppage time is reduced, and so are the construction time and overall cost.

In relation to cost comparison, the option of asphaltic core embankment dam is selected from RCC dam and earth core embankment dam which is built in Norway completed in 1997 and China (three Gorge dam) respectively. The same case goes to south Africa among the three alternatives of RCC, concrete faced and asphalt core dams, the latter was chosen due to cost and because the dam was located in an earthquake region on a poor sandstone foundation (Hoeg, 1997).

Earth core rock fill dam (ECRD), upstream concrete facing rock fill dam (CFRD) and Asphalt concrete core rock fill dam (ACCD). To choose the most suitable among these alternatives emphasis was placed on costs, severe weather conditions during construction, earthquake resistance, and compatibility with the geological conditions which might cause significant differential settlements across the valley. After all aspects had been considered, the ACCD alternative was decided to be the most suitable (Wang and Höeg, 2010).

The dam type selection process has thoroughly analyzed the different options leading to confirm that the most suitable dam type options for the propose embankment dam for Dodota Irrigation Project is Rock fill with central asphalt concrete core. The Construction of Asphalt Concrete Core is relatively new technology in our country and demands an experience or qualified contractor as well as good quality control. However, Welkayit Dam is the best practice in our country. The research on ducted by the Norwegian Geotechnical Institute in cooperation with Kolo Veidekke, indicated that all dams with built-in asphalt cores are reported to have excellent field performance. Seepage through the core is claimed to be negligible and to date no repairs have been required. Therefore, for propose embankment dam for Dodota Irrigation Project Rock fill with central asphalt concrete core is preferable based on above listed facts under consideration.

2.4 Foundation Treatment

Foundations which consist of hard and erosion resistant bedrock are the most desirable. The use of foundations consisting of river gravels or rock fragments is acceptable under some circumstances, but a positive cutoff to rock is usually necessary. The foundation should be selected and treated so as to result in minimum settlement of the rock fill embankment. Any materials in fractures or deep excavations which may eventually erode into the rock fill, either from the foundation or from the abutment, should be protected with filters or removed, if necessary, and backfilled with concrete or suitable backfill .

To minimize the embankment dam failure due to foundation treated with different option. Foundation treatment increases the bearing capacity to minimize seepage loss and deformation. Foundation treatment of embankment dam should always need 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, strengthen and consolidation mechanisms (ICOLD, 2012).

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 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).

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, karastic formation and alluvial soil Bruce, (1994). Jet grouting is different from a conventional type of grouting that it a mix of in place process where the ground is disturbed and remixed insitu 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, (2015) as the construction takes place inside ground.

2.5 Embankment Dam Design

Earth and rock fill dams are constructed of all types of geologic materials, with the exception of organic soils and peats. Most embankment dam are designed to utilize the economically available on site materials for the bulk of construction. Special zones such as filters, drains and riprap, may come from offsite sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits, and from required excavation where suitable (USSD, 2011).

A great variety of rock types have been used in constructing rock fill dams. The types of rock used range from hard, durable, granite, and quartzite to weaker materials such as greywacke, sandstone, and slaty shale. In the past, designers thought that only rock fill material of the highest quality should be used; however, with the advent of thinner lifts and more efficient compaction techniques, rock of less desirable characteristics has been used within the embankment sections. A well-graded mixture of rounded gravels and cobbles are the most durable type of rock fill because more angular rock under high stress levels tends to break at the contact points of the rock particles or along fractures, causing more settlement and more deformation (USBR, 2012).

2.5.1 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).

Compared to a modified homogeneous dam, zoned dams are usually constructed in areas where several material types are available, such as clays, silts, sands, gravels, and rock. Zoned embankments take advantage of the availability of various materials by placing different materials in various zones so that their best properties are used most beneficially, and their poor properties are mitigated. A zoned earth fill dam typically has a central impervious core flanked by upstream transition zones, downstream filters and drains, and then outer zones or shells composed of gravel fill, rock fill, or random fill, which are considerably stronger than the core. Depending on the gradation of available materials, transition zones may not be necessary. The shells support and protect the impervious core, transition zones, filters, and drains; the upstream pervious zone provides strength for stability against rapid drawdown; and the downstream zone provides strength to buttress the core and filters so that steeper (more economical) slopes can be used. The upstream transition zone, if necessary because of a very pervious shell, provides protection against internal erosion or washout of the core during rapid drawdown (USBR, 2012).

2.5.2 Water barrier

An earthen core of low permeability is the common water barrier provided for a rock fill or zoned embankment dam. However, at many dam sites, particularly in high altitudes, impervious materials are very scarce, and those that are available are saturated. These facts can be complicated by a very short construction season. In these cases, an impervious earth core becomes impractical and an alternative solution is to design an asphalt concrete water barrier. This asphalt concrete diaphragm or membrane can be a vertical, or near vertical element situated in the interior of the dam, known as an asphalt concrete (AC) core (USSD, 2011).

The fact that the properties of asphaltic concrete can, within fairly wide limits, be tailored to satisfy specific design requirements, is an important aspect and advantage of the method of using bituminous cores in embankment dams. An earth-rock fill dam must be safe and stable during all phases of construction and operation of the reservoir (ICOLD, 2011).

AC cores, first developed in 1948, were sometimes constructed on a slope (upstream toward downstream, as the dam rises), but now are usually vertical in cross-section, and follow or parallel the axis of the dam, in plan. For high dams, the upper segment of the core may be sloped to maintain positive stresses on the core during reservoir operations. The modern AC core is placed right along with the rising embankment, keeping the fill crowned, with the core at the high spot so as not to be damaged by flooding during rainfall. The thickness of the AC diaphragm is often constant, and is 0.5m minimum, but not less than about 1 percent of the height of the dam. For high dams, the thickness can be reduced from the base to the crest, in steps. There are transition zones both upstream and downstream. The upstream transition, with a width of 1.5 m to 3 m, is grain-size compatible with the upstream shell, and has non-plastic fines that serve as a crack stopper for the AC core. The downstream transition, with a width usually ranging from 1.2 m to 2 m, functions as a chimney drain (USSD, 2011).

Many research result shows that the asphalt concrete core in an embankment dam construction, especially in a seismic region can withstand very severe seismic shaking without cracking and loosing water tightness. Among the existing large dams with compacted asphaltic concrete core, the minimum core width, in the top portion of the embankment, is 0.4 m. The maximum thickness so far used is in a 105 m high dam in Hong Kong where the bottom portion is 1.2 m wide.

The thickness of the asphaltic concrete core diaphragm is often constant, and is 0.5m minimum, but not less than about 1 percent of the height of the dam. For high dams, the thickness can be reduced from the base to the crest, in steps. There are transition zones both upstream and downstream. The upstream transition, with a width of 1.5 m to 3 m, is grain-size compatible with the upstream shell, and has non-plastic fines that serve as a crack stopper for the asphaltic concrete core. The downstream transition, with a width usually ranging from 1.2 m to 2 m, functions as a chimney drain (Hoeg, 1993).

From the literature and construction of experience for Dodota Irrigation project Asphaltic concrete core width variable between 1m to 0.5 m from the base plinth up to the dam crest elevation, will constitute the primary barrier against water leakage inducing most of the water head loss. This material will be effectively confined by Filter-Transition Zone material, while undergoing deformations of the dam body subjected to the external loads. Filter-Transition Zone material will constitute a second safety barrier against water leakage, in case of any potential local leakage through the asphalt core. The Asphalt Concrete is constituted by bitumen, aggregates and filler materials.

A transition zone of fine filter on each side of the asphalt core is 2.5 m upstream and downstream. This material, at direct contact with the core obtained as natural alluvial gravel or crushed rock, is a well graded sand and gravel to be compacted simultaneously with the asphalt core in 0.20 m thick layers. It exerts the effective confinement of the asphalt core while undergoing deformations of the dam body subjected to the external loads. In case of temporary cracking of the asphalt core it will constitute a second safety barrier against water leakage inducing significant water head loss while being protected against erosion by the downstream transition zone.

A second transition zone of crushed rock adjacent to the first transition on both sides of the core is 4 m and 5 m upstream and downstream respectively. This zone is a well graded mix of sand, gravel, cobbles and boulders up to 200 mm size, to be compacted in 0.40 m thick layers. It operates as a transition between the filter-transition zone and the rock fill material zone and exerts the effective transfer of load between the two zones.

A well compacted rock fill zone of quarried adjacent to the second transition zone on upstream sides of the core and downstream side of the fill is designed on the bases of optimization to utilize most economical fill materials of the shell as random fill surrounded by alluvial gravel and rock fill which are quite good in their free draining nature. The material will be obtained by quarries or alluvial deposits and must be free-draining. The material has to be compacted in 1.20 m thick layers (after compaction) with vibratory roller of minimum weight of 15 tons, a number of 6 passes if confirmed by the results gathered from trial embankment operations in order to achieve the specified dry density.

Rockfill materials may not be ideally suited for embankment slope protection because they typically contain significant sand and gravel particle sizes between the larger rock fragments, which can be easily scoured by wave action. If the coarser rock fill is not concentrated on outer parts of the slopes, the embankment may still require a designed slope protection consisting of riprap and bedding. Adequate slope protection must be provided for all earth and rock-fill dams to protect against wind and wave erosion. Damped riprap stone in the upstream slope protection must consist of sound rocks with sufficient weight to withstand the action of waves (USBR, 2012).

The tolerable damage riprap layer thickness of 0.56 m and zero damage of 0.68m are found as per the design standard. For the safety consideration of riprap provision of 1.00 m is used for propose embankment dam for Dodota Irrigation Project. For the downstream slope the existing gravel fill surface is considered to be sufficient for protection there also berm and plant cover is minimize the risk (USBR, 2012).

The objective of filters and drains used as seepage control measures for embankments is to efficiently control the movement of water within and about the embankment. In order to meet this objective, filters and drains must, for the project life and with minimum maintenance, retain the protected materials, allow relatively free movement of water, and have sufficient discharge capacity (USSD, 2011).

2.5.3 Upstream and downstream slopes

Rock fill dams having central or sloping earth fill cores usually have slopes of about 1.5H:1 V to 2H:1V upstream and downstream, often depend on the location of the core. The upstream slope is generally flatter, particularly for upstream sloping cores (USBR, 2012). Inclination of downstream slope at earth rock fill dams with central core in some cases would have to be more moderate V:H (1:1.6 to 1:1.7) and upstream slope is usually dictated by its convenience for construction and in seismically active regions often adopted as V:H (1:1.8 to 1:1.9) most practice in japan (Tanchev, Ljubomir, 1945).

Embankment slopes used for rock fill dams have evolved from very steep slopes, usually 0.5 to 0.75: 1 (H:V) which were used on early rock fill dams, to the flatter slopes of 1.3:1 to 2.0:1 used in current practice (USBR, No.13, 1986). The recommended values of side slopes as given by Terzaghi (Terzaghi, 1943) are given in table 2.1 below

Table 2.1: Embankment dam slope recommended by Terzaghi, 1943

Type of material	Upstream slope (H:V)	Downstream slope (H:V)
Homogeneous well graded	2.5:1	2.5:1
Homogeneous course silt	3:1	2.5:1
Homogeneous silt clay		
1.Height less than 15 m	2.5:1	2:1
2.Height more than 15m	3:1	2.5:1
Sand or sand and gravel with central clay core	3:1	2.5:1
Sand or sand and gravel with R.C diaphragm	2.5:1	2:1
Rock fill	1.6 to 2:1	1.6 to 2:1

The side slopes of the dam is determined by Stability Analysis Program, Geo-studio 2012, accounting for each of the different construction materials and considering all design criteria conditions. The side slopes depend upon various factors such as the type and nature of the dam, foundation materials, height of dam and economic requirement. From the literature and the constructing project in our country such as Welkayit dam is designed as zoned rock fill dam with central asphalt concrete with upstream slope (H: V) =2.0:1 and downstream slope of (H: V) = 1.8:1. Inclination of downstream slope at earth rock fill dams with central core in some cases would have to be more moderate V:H (1:1.6 to 1:1.7) and upstream slope is usually dictated by its convenience for construction and in seismically active regions often adopted as V:H (1:1.8 to 1:1.9) most practice in japan (Tanchev, Ljubomir, 1995).

From the geometry of dam profile the propose embankment crest length 250m , the dam height 60m and from the stability analysis result determine upstream and downstream slope for proposed Dodota Irrigation Project slope is 1:1.8 (V: H) and 1:1.6 (V: H) respectively .

2.5.4 Dam crest width

The top width of an earth or rock-fill dam within conventional limits has little effect on stability. The crest width is often governed by construction procedure and the access required. Depending upon the height of the dam, the minimum top width, according to USACE, is between 7.5 and 12.0 m. The width of dam at crest as per BIS 8826 - 1978 "Guide lines for design of large earth and rock fill dam" should be fixed according to the working space required at top and the crest width should not be less than 6.0 m. The Japanese code 1957 specifies crest width (W) in terms of dam height (H) as follows.

$$W=3.6 * \sqrt[3]{H} -3 \text{ (meters)}$$

Table 2.2: Embankment dam crest width propose by different code or equation

Code	Unit	Criteria
BIS 8826-1978	m	>6
USACE	m	7.5 to 12
Japanese (1957)	m	14.5

The crest width of the dam is used 10 m for design taken from the standard manual in Japan and (USACE, 2011).Freeboard must be sufficient to prevent overtopping by waves and include an allowance settlement of the foundation and embankment as well as for seismic effects where applicable (USACE, 1993).

2.5.4 Provision of berms

Berm used in dam for providing level surface for construction and maintenance of the dam section and used for reducing the surface erosion in case of downstream slope and breaking the continuity of the slope (USBR, 2012). Berms of 4 m width have been provided at downstream faces of the dam different elevations as indicated below. The berm will be inclined at 2% slope towards the fill and the elevation of Berm 1=1685 m, Berm 2= 1700 m and Berm 3=1715m and the providing berm upstream side at elevation of 1695 m.

2.6 Software Used for Model

Over the years there has been an increase in construction and infrastructure projects and consequently a growth in the requirements for dam design, excavation, footings and road design. Engineers must take into account all geotechnical aspects affecting their design including soil material properties, slope stability and possible natural disasters which can have devastating social and economic impacts. Incorporating the analysis of slope stability within the design will help in the prevention of any geotechnical failures throughout construction and the life of the design (Fell, 2005).

Plaxis is a numerical program based on the finite element method and is very useful for calculation of loads, deformations and water flow in soil and for geotechnical constructions. It was developed in the end of the 1980 at Delft University in the Netherlands. During the 1990 it was further developed for commercial purposes and is since 1998 available in Windows environment.

Plaxis is described as a FE package for geotechnical analysis that can utilize both two-dimensional and three-dimensional analysis in determining the stability and deformation experienced by slopes. Plaxis lends itself to modeling more complex geotechnical scenarios as it has the capabilities to simulate inhomogeneous soil properties and time-dependent scenarios (Brinkgreve, 2012; Gouw, 2012).

The models produced by Plaxis can be considered a qualitative representation of the soil's behavior. However the models "simulation of reality remains an approximation, which implicitly involves some inevitable numerical and modeling errors" (Gouw, 2012).

Geo Studio is composed of eight software products that enable everything from simple-to-complex analyses. When integrated, the products offer a broader analytical environment that offers significantly more power and capabilities. The fully-integrated software suite includes limit equilibrium stability analysis and seven finite element applications for modeling geotechnical and earth science problems.

SLOPE/W Slope stability analysis, SEEP/W Groundwater seepage analysis, SIGMA/W Stress-deformation analysis, QUAKE/W Dynamic earthquake analysis, TEMP/W Thermal analysis, CTRAN/W Contaminant transport analysis, VADOSE/W Vadose zone and soil cover analysis, and AIR/W Air flow analysis.

The limit equilibrium method establishes the required soil properties; slope geometry and then using the Mohr-Coulomb criterion calculates the stability of the slope by comparing the forces causing failure against the resisting forces. Throughout this procedure an FOS is computed using the equations of static equilibrium. “The fundamental assumption…is that failure occurs through sliding of a block or mass along a slip surface” and in order to compute the appropriate FOS a number of slip surfaces need to be postulated to find the critical slip surface (Duncan & Wright 2005; Hammouri et al. 2008; Cheng & Liu 1990, Das 2010).

The Finite element method is a numerical technique for solving differential equations or boundary value problems in science and engineering. The FE method has been adapted for geotechnical engineering; however there is a perception by professionals in the geotechnical industry that the FE method is too complex and there is criticism in its necessity compared to the simpler LE method considering the poor quality of materials properties often used in the analysis (Das, 2010).

FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation. This program simulates the behavior of a structure built of soil, rock, or other materials that may undergo plastic flow when their yields limits are reached. FLAC finds the static solutions for a problem using the two-dimensional plane-strain model. However, the dynamic equations of motion are included in the formulation to help model the stable and unstable forces within the model; this ensures that the scenario of a sudden collapse within the model is accounted for (Gouw, 2012).

The equations of motion are utilized to derive the velocities and displacements; then the new stresses and strain rates are calculated and so forth until failure is achieved. Note that a relatively small time step is chosen to ensure that the stress changes of each element do not influence its neighbor’s.

From the above software Plaxis, FLAC and Geo Studio conclude to use Geo Studio software for my thesis because of the following condition.

1. The solving process used in Plaxis is more complex compared to Geo Studio requiring more time to input the necessary parameters and using the correct procedure to perform the analysis.

2. The models produced by Plaxis can be considered a qualitative representation of the soil's behavior. However the models "simulation of reality remains an approximation, which implicitly involves some inevitable numerical and modeling errors".
3. The factor of safety due to earthquake using Geo-Studio and Plaxis the Geo-studio result more accurate than Plaxis (Das, 2010).
4. The full license tool of Plaxis and FLAC is not available easily.

3. MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location

Dodota Irrigation Project is located in Oromia Regional State, Arsi Zone, Dodota District, 135 km southeast from the capital, Addis Ababa. The command area is located in the north of Dera-Sire road, near Dera town (Figure 3.1) and close to the eastern margin of the Ethiopian Rift System (Figure 3.2).The approximate geographic coordinates attributed to the central parts of the dam site location is 8.3°E and 39.4°N.

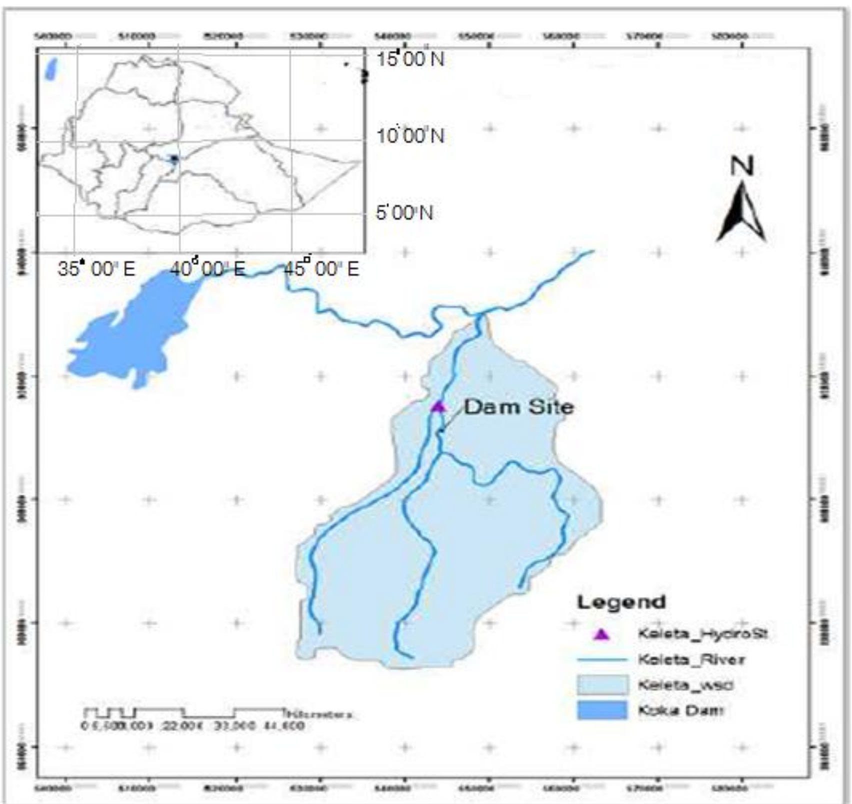

Figure 3. 1: Location of project area

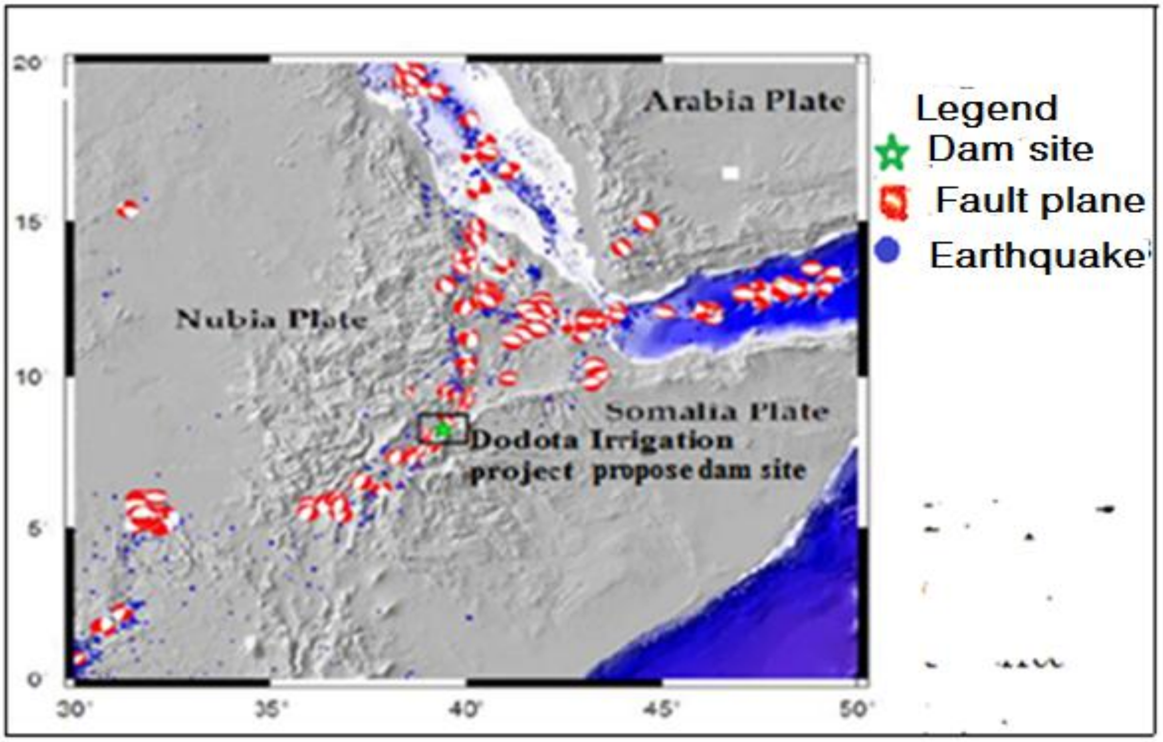

Figure 3. 2; Ethiopian Rift systems

3.1.2 Climate

According to the meteorological report of the Dodota Irrigation Project the annual average of daily maximum temperate of the study area is 29°C and daily minimum temperature is 13.7°C.The rainfall in main rainy season (from June to September) is 637 mm accounting for 70% of total rainfall. However, the rainfall in dry season (from October to February) is as low as 105 mm. The rainfall in small rainy season (from March to May) is 178 mm .The average annual rainy days for 10 years is 93 days. The rainy days in main rainy season (from June to September) is 60 days accounting for 64.5% of total rainy days in years. The rainy days in dry season (from October to February) is 13 days accounting for 14% of total rainy days in years. The average annual hours of sunshine is 8.3 hour/day for 10 years. The daily 2.2 hours of sun shine in dry season (from October to February) is 1.1 hours longer than average annual sunshine time and 2.2 hours longer than sunshine time in main rainy season (from June to September) .The average annual wind speed is 2.5 m/sec. The average wind speed in dry season is 2.8 m/sec and average wind speed in small rainy season (from March to May) is 2.4 m/sec.

3.1.3 Availability of construction material

Borrow for impervious dam core and alluvial deposits (for rock transition) material has been identified as some 5km downstream of the dam site on the right side of Keleta River Rock quarries are identified on the left and right side of Keleta River at the vicinity of the dam site, about 2.0 km from the right abutment and 1.5 km from the left abutment.

3.1.4 Geology

According to the regional geological investigation (KOICA, 2015) the study area is located at the eastern margin of the main Ethiopian rift bounded to the east by the main rift escarpment fault. The Keleta River generally flows to the North and North east following the general fault orientations, and exposing the underlying geological units along deep gorges. The Eastern Margin Unit is mainly exposed by the major escarpment faults east of the Keleta River and lies deep beneath the Keleta River bed within the rift. This unit forms up to 200 m thick sequence of weathered usually coarse-grained densely welded varicoloured ignimbrites containing vitrophyric fiamme (elongated, pumice clasts) and lithic fragments with associated lava dome and flows, interleaved with subaphyric to porphyritic basalts and occasional palaeosol layers. The age of this unit is set to early Quaternary (1.8 Million year ago).The Keleta and Boru Rivers expose the Keleta Unit in the vicinity of the study area.

The study area is generally overlain by a thin sequence of unwelded ignimbrites (the lower subunit of the Dera-Sodere Unit) forming the flat plains west of the Keleta and Boru Rivers, underlain by a thick sequence of variously welded and vari-coloured ignimbrite layers intercalated with basalt and palaeosol layers (the Keleta Unit) exposed by the Keleta and Boru Rivers. The upper subunits of the Eastern Margin Unit composed of coarse-grained, loosely welded ignimbrite intercalated with basalts, trachy basalts and basaltic agglomerates are also present at deeper levels. The foundation and abutments of the proposed dam sites and the tunnels all fall within the Keleta Unit. The propose embankment dam site axis, the river is approximately 45m wide flat bottom forming V shaped valley with the slopes approximately 1V:1H on both sides of the river. The bed rock of dam axis, at the river center and on both sides of flatbed areas is covered by cobbles, gravels and silt sand deposited by means of colluvial and alluvial actions. On the basis of the field investigations, the overburden is found to be shallow with an estimated thickness ranging 4 -5m at the river bed, 2m to 9m on both slopes top of the abutments. The soil at the river bank, along the river course consists of boulders, cobbles and sandy silt with no or a thin layer of clay overlaying this alluvial deposit. This alluvial deposit has also contained clay and gravel size particles with boulders and cobbles are being the dominant particle size. The thickness of the layer and gravel content of the residual soil seem to decrease with the increase of distance from the river center.

The other dominant rocks encountered in the boreholes drilled along the dam axis are Trachybasalt and slightly weathered strong Ignimbrite. Both rocks have been encountered in both boreholes and have relatively good rock quality. The combination of the Trachybasalt and Ignimbrite rock competency and shallow overburden of the dam site leads to the conclusion that the dam structure could be founded on the competent bed rock (KOICA, 2015).

The first site basic design study (BDS) dam site is located in Keleta River that is about 10km far from Huruta town. Here are two alternative at downstream stream of the basic design site ; the alternative number one dam site is at 2.1km downstream of dam site in basic design site and alternative number two dam site is at 0.8km downstream of alternative number one dam site. The dam site is selected in consideration of the topographical and geological conditions through site investigation. Weir construction is proposed at alternative number two dam site and the proposed embankment dam for this study is selected at alternative number one dam site. The alternate sate of those three sites and the canal layout is shown on Figure 3.3 and the photo of alternative one is also shown by Figure 3.4. Refer the geological section of the propose embankment dam site at appendix Figure B.1 for dam geometry.

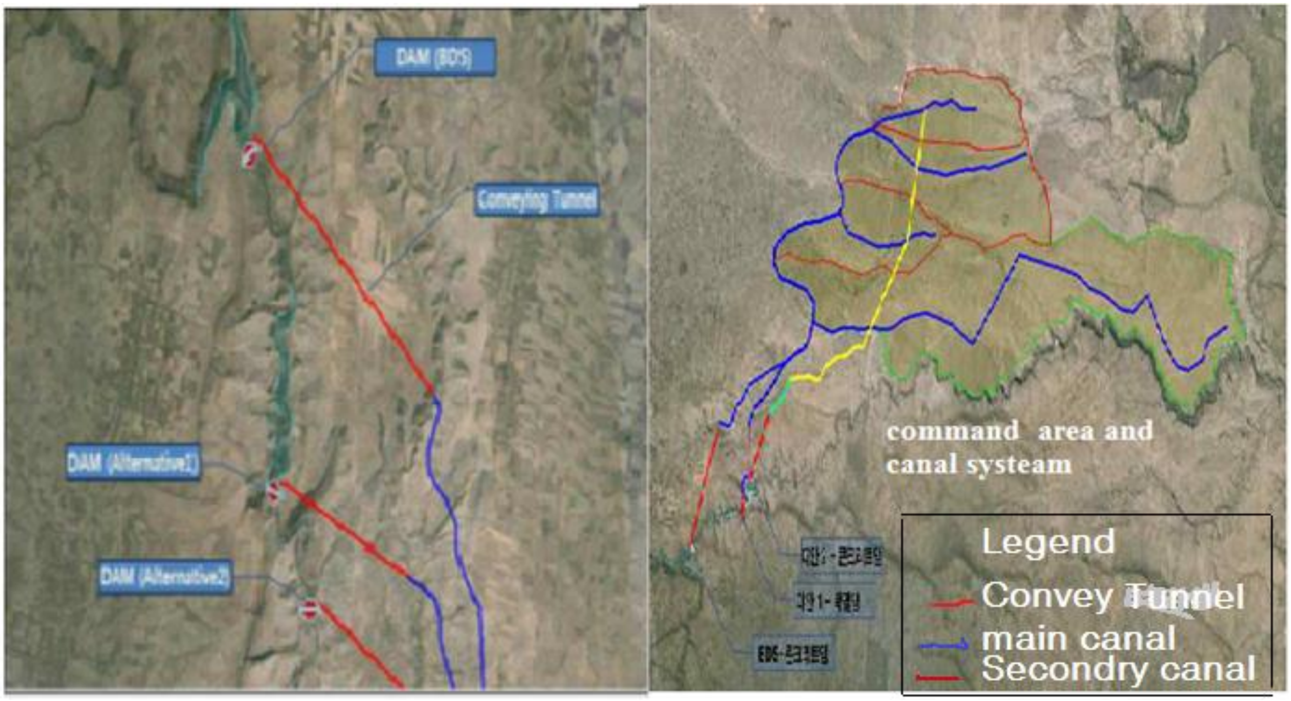

Figure 3. 3: Dam alternative site and command area (KOICA, 2015)

Figure 3. 4: Picture of propose dam site at alternative site

3.1.5 Topography

The Keleta River is formed in the deep gorge below the flat plateau of EL. 1,750 m that has been used as farmland. There is about 70 m to 100 m of altitude difference between the flat plateau and the Keleta River.

3.1.6 Hydrology

According to the hydrological report (KOICA, 2015) the flood potential of Keleta River is very high during the rainy season and very small during the dry season. The base flow of Keleta River is 10.3 m^3/s during the rainy season and the base flow during dry season is 0.77 m^3/s. The daily stream flow observation data is available from 1967 to 2011 for Keleta gauging station near Sire which is 8.3 km downstream of the proposed dam site. Mean runoff of observed data of Keleta River shows at appendix Figure B.2.The flood frequency analysis was done using the daily maximum river discharge of Keleta River by year. The station is located around the downstream area of the proposed dam site and in the mainstream of the Keleta River. The flood frequency analysis of daily maximum discharge from daily maximum rain falls with 100 years 344.5 m^3/s. In this project the estimated amount of flood from daily maximum discharge is 474.0 m^3/s computed for 1000 years of return period. The expected annual inflow from the watershed area estimated is 158.560 million m^3 per year. From these hydrological report no problem of water supply for the reservoir capacity of the propose dam. The capacity of the reservoir on dam site is determine from the contour map of the area. The reservoir capacity of proposed embankment dam for Dodota Irrigation Project is calculated by using ArcGIS software with counter of five meter interval. The capacity of the reservoir is 10 million m^3 at full supply level of 1715 m and the dead storage level at 1680 m with storage of 0.7 million m^3 the result shown on the appendix C table.1 and appendix figure B.3.

From the hydrological reporter of the study document given above there is high water from flood during rainy season and there also small water source during dry time. From this concept the project face a problem of shortage of water during dry time and fixe the irrigated command area, therefor to save the available water resource and increase the irrigated command area constructing dam for the site is an alternative choice.

3.2 The Dam Zoning and Geometric

Keleta River is a tributary of the Awash River and joins the Awash River in about 20 km downstream of dam site. Keleta River originates at the Horasa Mountain and Chilalo Mountain, 4,000 m above sea level. There is no hydraulic structure to use the water resources in Keleta River. The main road from Dera to Sire contain bridge on the Keleta River at downstream of 8.3 km the propose embankment dam for Dodota Irrigation Project. From the literature and to solve the geological problem of the site the propose embankment dam selected central asphaltic core embankment dam with zoned geometry. Asphalt has long been used in the construction of dams: to grout foundations when running groundwater washes away Portland cement particulate grout; as a coating for conduits penetrating the dam, to control seepage; as a protective coating on exposed foundations that are subject to air or water slaking; and as the water barrier element of an embankment dam. Consideration must be given to maintenance requirements so that economies achieved in the initial cost of construction will not result in excessive maintenance costs. For minimum cost, the dam must be designed for maximum utilization of the most economical materials available, including materials, which must be excavated for its foundation and for appurtenant structures. Based on this ground, and assessing different design standards basically USACE, USACE, Design Standards Embankment Dams No. 13, USBR, ICOLD, and Fell, Geotechnical engineering of dams, the selected type of dam is rock fill dam with central asphalt core that gives rise to the lesser cost, lesser time of construction as well as best use of rock and shell materials.

The geometric dimension of the propose embankment dam are 235 m of bottom width at maximum cross section, 10 m top width, dam crest length of 250 m, free board 5 m and dam height of 60 m. The dam section geometry for model analysis of the dams selected considering the thick alluvium deposit under the dam foundation and at large cross-section. The propose Asphaltic Concrete core bed elevation of the river 1665 m and crest at elevation of 1725 m. The upstream and downstream slopes of the dam are taken as 1.8 H: 1V and 1.6H: 1V. The propose embankment dam zoning have the following material. The clay core alternative is upstream and downstream of slope is with 1V :0.25H is recommendable.

Asphaltic concrete core zone: Many research result shows that the asphalt concrete core in an embankment dam construction, especially in a seismic region can withstand very severe seismic shaking without cracking and loosing water tightness. From the literature and construction of experience for Dodota Irrigation project Asphaltic concrete core width variable between 1m to 0.5 m from the base plinth up to the dam crest elevation, will constitute the primary barrier against water leakage inducing most of the water head loss. This material will be effectively confined by Filter-Transition Zone material, while undergoing deformations of the dam body subjected to the external loads. Filter-Transition Zone material will constitute a second safety barrier against water leakage, in case of any potential local leakage through the asphalt core. The Asphalt Concrete is constituted by bitumen, aggregates and filler materials.

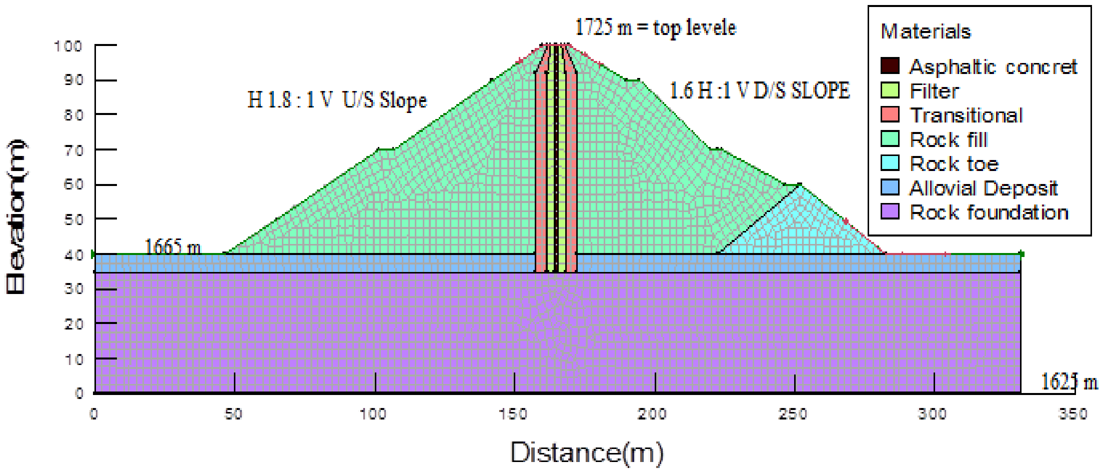

Figure 3.5: The propose embankment dam geometric AC

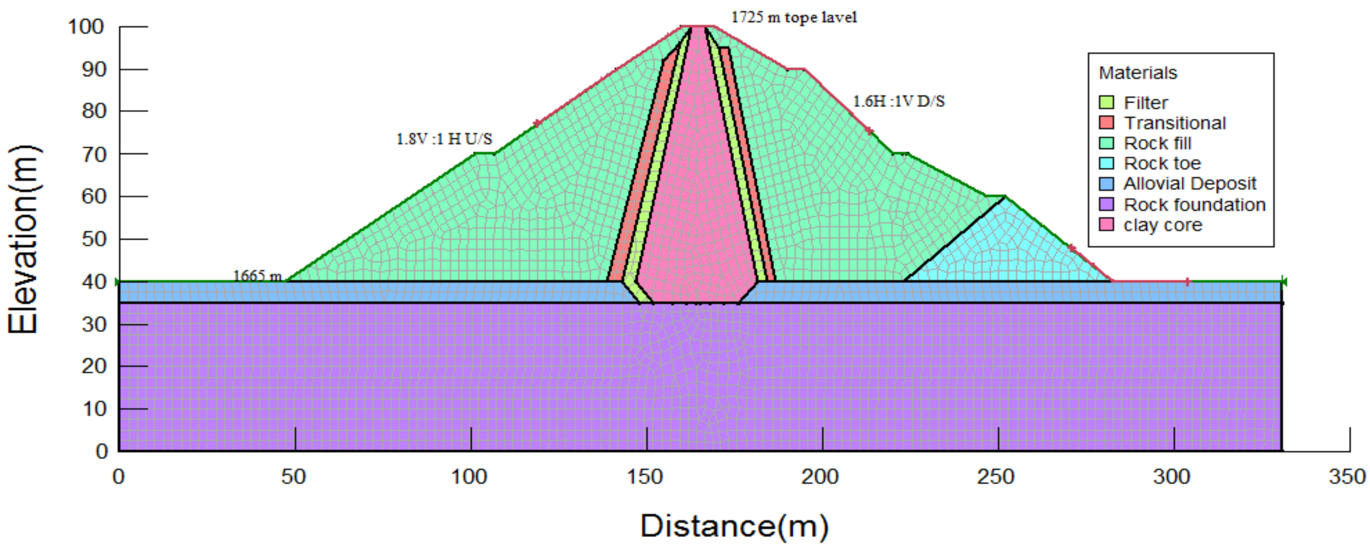

Figure 3.6: The propose embankment dam geometric CC

Filter-Transition Zone: A transition zone of fine filter on each side of the asphalt core is 2.5 m upstream and downstream. This material, at direct contact with the core obtained as natural alluvial gravel or crushed rock, is a well graded sand and gravel to be compacted simultaneously with the asphalt core in 0.20 m thick layers. It exerts the effective confinement of the asphalt core while undergoing deformations of the dam body subjected to the external loads. In case of temporary cracking of the asphalt core it will constitute a second safety barrier against water leakage inducing significant water head loss while being protected against erosion by the downstream transition zone.

Transition Zone:-A second transition zone of crushed rock adjacent to the first transition on both sides of the core is 4m and 5m upstream and downstream respectively. This zone is a well graded mix of sand, gravel, cobbles and boulders up to 200 mm size, to be compacted in 0.40 m thick layers. It operates as a transition between the filter-transition zone and the rock fill material zone and exerts the effective transfer of load between the two zones.

Rock fill Material:-A well compacted rock fill zone of quarried adjacent to the second transition zone on upstream sides of the core and downstream side of the fill is designed on the bases of optimization to utilize most economical fill materials of the shell as random fill surrounded by alluvial gravel and rock fill which are quite good in their free draining nature. The material will be obtained by quarries or alluvial deposits and must be free-draining.

3.3 Material used for Analysis

The software used for analyzed output of the thesis was Geo Studio. Geo Studio composed of eight software products that enable everything from simple-to-complex analyses. When integrated, the products offer a broader analytical environment that offers significantly more power and capabilities. The fully-integrated software suite includes limit equilibrium stability analysis and seven finite element applications for modeling geotechnical and earth science problems. SLOPE/W Slope stability analysis, SEEP/W Groundwater seepage analysis, SIGMA/W Stress-deformation analysis, QUAKE/W Dynamic earthquake analysis, TEMP/W Thermal analysis, CTRAN/W Contaminant transport analysis, VADOSE/W Vadose zone and soil cover analysis, and AIR/W Air flow analysis. For this research SLOPE/W for slope stability, SEEP/W for estimation of seepage through dam body and foundation, SIGMA/W for stress-deformation, QUAKE/W for dynamic earthquake has been used.

3.4 Methodology

The intent of this study is to provide safe and reliable alternative design for propose dam Dodota Irrigation Project and compare the alternative design Asphaltic concrete core rock fill dam with the clay core rock fill dam. 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 the propose dam type.

The general activities accomplished during investigation of the site:-such as primary and secondary data is collected, visited the Dodota Irrigation project to collect primary data such as physical observation of the site condition, gathering of required photographs and examine different head work that proposed as alternative site. Discussing with different senior professional experienced in dam construction or in dam design. After creating a computer model of an asphaltic core rock fill dam with determined material properties and geometrical conditions, the pore water pressures developed within the body of the dam and in the foundation under steady state seepage has been initially estimated with the help of the SEEP/W software.

Then creating a computer model design propose embankment dam with determined material properties and geometrical conditions for calculating the slope stability was carried out using the Slope/W computer program based on the limit equilibrium method and the Morgenstern-Price method was used to obtain the factors of safety. With determined material parameters and geometrical conditions initial static analysis is performed on the model with SIGMA/W. The analysis is carried out with the aim of controlling the model and truth of results together with using the stresses obtained from static analysis in dynamic analysis.

In the next step, dynamic analysis is accomplished on dam model considering the record of a specific earthquake, which may be the record prepared from seismotectonic of the site belonging to the dam under study.

After checking all analysis of the dam design criteria by using Geo-studio 2012 give conclusion and recommendation on the possibility of constructing dam for Dodota Irrigation Project.

3.4.1 Data availability

Before conducting of any research, it is imperative to make a tough search for the data. Therefore, the primary assignment of the study is getting relevant information and data of the study area. For this thesis, geological and hydrological data are taken from feasibility study document of Dodota Irrigation Project and Water Works Design and Supervision Enterprise work of different project experience and some others adopted from typical works of (Duncan et al., 1980).

3.4.1.1 Primary data

Primary data collected from the site. These data may include topographic survey of the head work axis and command area, physical observation of the site condition and gathering of necessary photographs and all important information.

3.4.1.2 Secondary data

All necessary Secondary data collected from geotechnical investigation and laboratory report used for Geo-studio 2012 such as embankment material property (unit weight, angle of internal friction, cohesion of soil, hydraulic conductivity and condition of soil water content), impervious material, and foundation material, aggregate, filter and rip rap. The detail design report and project drawing of Dodota Irrigation project collected from Oromia Irrigation Development Authority (OIDA) and Korea International Corporation Agency (KOCA).Topographic map of the Dodota district from Ethiopia Map Agency.

3.4.2 Material properties used for analysis

The material properties used for different analysis in Geo -Studio model is given in the next table 3.1. The foundation was modeled with a linear elastic model specified using values of Young's Modulus and Poisson's Ratio. Also the dam body was modeled with a nonlinear elastic model. Due to the complex nature of soil behavior, all observed behaviors of an embankment dam might not necessarily lend themselves to a precise analysis. In such instances, reliance on engineering judgment, based on professional experience of responsible engineers, is generally considered prudent and acceptable. The parameters used for analysis was taken from KOCA, 2015 final feasibility study, laboratory report (Saba Engineering, construction design share company and Water Works Design & Supervision Enterprise) and (Duncan et al.,1980). These parameters were used in a finite element numerical model to evaluate the seepage, stability and settlement of Dodota Dam Irrigation project propose embankment dam. These parameters were used in a finite element numerical model to evaluate the seepage, stability and settlement of the propose embankment dam for the Dodota Irrigation Project. The following table 3.1 lists the parameter of material properties of the proposed dam zoning.

Table 3.1: Parameter of material properties

Material type	Material properties’						
	γ_d	C'	∅’	K	E *	G_{max}	V	Remark
Asphaltic core	24	45	26	$1.32*10^{-10}$	$2.5*10^{5}$	$5*10^{4}$	0.35	Hoeg, 1993
Clay core	16	20	23	$4.78*10^{-7}$	$1*10^{5}$	$1.3*10^{3}$	0.3	KOCA, 2015
Filter	20	0	32	0.001	$4*10^{4}$	$1.9*10^{4}$	0.3	KOCA, 2015
Transitional filter	20	0	32	0.0019	$4*10^{4}$	$3.8*10^{4}$	0.3	Duncan et al., 1980
Rock fill	22	0	42	0.01	$5*10^{5}$	$2.2*10^{4}$	0.15	KOCA, 2015
Rock toe	18	0	38	0.005	$5*10^{5}$	$2.2*10^{4}$	0.15	KOCA, 2015
Alluvial deposit	19	10	29	$5.8*10^{-6}$	$1*10^{5}$	$4.2*10^{3}$	0.2	KOCA, 2015
Sound Bed rock	23	15	42	$1*10^{-8}$	$2*10^{6}$	$8.5*10^{5}$	0.1	KOCA, 2015

*Bowles et al, (1997)

Where:-

γ_d=Dry unit weight (KN/m^3), C' =Cohesion (KN/m^2), ∅ ’ =Internal Friction angle (degree), ∅’ =Internal Friction angle (degree), E =Young's Modulus (KN/m^2), K=Permeability (m/s),

G_{max} = Maximum Shear Modulus (KN/m^2), V =Poison's Ratio , k_L = Modulus Number

3.5 Method of analysis

3.5.1 Seepage analysis

Piping, erosion and development of excessive pore pressure are cases that need to be checked for the safe design of embankment dam. Using the SEEP/W component of Geo-Studio software a steady state seepage analysis was done to determine the amount of seepage through embankment dam and foundation. SEEP/W is a finite element based Geo-studio component, used for seepage analysis. It is used for modeling of movement of water and pore- water pressure distribution through a pores media such as 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, 2015). There are two fundamental types of seepage analysis: steady state and transient. A steady-state seepage analysis is an analysis 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 also carried 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 seepage analyses, three finite element components are keys:-geometry, material property and boundary condition these fixed or determined.

1. Geometry:-since dam dimension is fixed, the geometry drawn on the SEEP/W window and discretized in to small elements as finite element numerical models (SEEP/W, 2012).

2. Material Property:-There are four different material models to choose from when using SEEP/W. Those are None (used to removed part of a model in an analysis), Saturated / Unsaturated model (Hydraulic conductivity function and Water content function), Saturated only model (Hydraulic saturated conductivity(Ksat), Saturated water content, Coefficient of volume compressibility (Mv) and Air conductivity set to zero) and Interface model (Hydraulic normal and tangent conductivity and Air conductivity) . 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, 2012)

3. Boundary condition:-.The boundary condition applied directly on geometry items such as region, faces, region line, free line or free point.

SEEP/W is formulated on the basis that the flow of water through both saturated and unsaturated soil follows Darcy's Law which states that:

$$q = ki \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.1)}$$

Where:

q =the specific discharge (m^2/s),

k=the hydraulic conductivity (m/s), and

i=the gradient of total hydraulic head (m).

Darcy's Law was originally derived for saturated soil, but later research has shown that it can also be applied to the flow of water through unsaturated soil (see Richards, 1931 and Childs & Collins-George, 1950). The only difference is that under conditions of unsaturated flow, the hydraulic conductivity is no longer a constant, but varies with changes in water content and indirectly varies with changes in pore water pressure.

The equation of finite element for seepage analysis would be given by (SEEP/W, 2012).

$$[K]\{H\} = \{Q\} \quad \text{--} \quad (3.2)$$

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

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

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

Boundary conditions can be depending on fundamental options either can be specified with H (head) or Q (flux). For simplicity, there are some fundamental types of boundary conditions which can be assigned such as potential seepage face, zero pressure and head pressure.

In SEEP/W, the presence of an upstream reservoir is reflected in the boundary condition assigned to the upstream face of the embankment. The upstream boundary nodes are designated as head boundaries with total head equal to normal water level in the reservoir plus elevation at river bed. For estimation of seepage using SEEP/W, the downstream toe is assigned total head equal to water head at dam toe plus elevation head. The downstream slope is assigned a potential seepage face type of boundary condition. A potential seepage face review boundary is a Q = zero boundary with the Potential Seepage Face. The total head boundary condition in SEEP/W has been assigned which is expressed in the form the following equation.

$$H=\frac{u}{\gamma w}+Y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

Where:

H=the total head (m)

u=the pore-water pressure (kPa)

γ_w =the unit weight of water (kN/m^3)

Y= the elevation (m)

3.5.2 Slope stability analysis

SLOPE/W is a Limit Equilibrium software product, used for stability analysis of earth slopes through Limit equilibrium method. It can effectively analyze problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods and loading conditions. Using limit equilibrium, SLOPE/W can model heterogeneous soil types, complex stratigraphic and slip surface geometry, and variable pore-water pressure conditions using selection of soil models. Slope stability analyses can be performed using deterministic or probabilistic input parameters. Stresses computed by a finite element stress analysis may be used in addition to the limit equilibrium computations. It is one of the most complete slope stability analysis programs available. Beginning an analysis in this program is through definition of the geometry by drawing regions and lines that identify soil layers. Then analysis method, soil properties and pore-water pressures can be chosen and applied.

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.

The pore water pressures developed within the body of the dam and in the foundation under steady state seepage has been initially estimated with the help of the SEEP/W software. In the next step stability analysis can be run and the results are presented through display of the minimum slip surface and factor of safety. The stability of an embankment depends on the characteristics of the foundation and fill materials, on the geometry of the section mainly. But the saturation level and loading conditions are also influencing factors. The procedures for static slope stability are well established, and have been concisely documented by (Duncan et al, 1980). Duncan recommends that evaluation of a slope focus on defining geometry, shear strengths, unit weights, and pore water pressures.

The slope stability analysis of the designed dam will be analyzed using SLOPE/W of Geo-studio 2012 software. The factor of safety calculated at different loading condition:- Upstream and downstream slopes during construction condition (with or without earthquake),Upstream and downstream slopes end of construction condition (with or without earthquake), Upstream and downstream slopes under steady state seepage condition (with or without earthquake) and during sudden draw down.

The general Procedure followed in using SLOPE/W are:-To open the SLOPE/W window set page size, unit and scale, grid spacing and axis of working area, then draw the geometry of dam zoning on the SLOPE/W window assigned respective material property for each region and material model, the factor of safety for reservoir full condition uses a parent-child terminology to describe the relative position of each analysis within a SEEP/W steady-state analysis is the Parent and is used to define the initial pore-water pressure conditions for the transient analyses.

The slope stability investigation was carried out using the SLOPE/W computer program based on the limit equilibrium method and the Morgenstern-Price method was used to obtain the factors of safety. This particular method has been adopted because, unlike Fellenius or Bishop’s or Janbu’s methods, the Morgenstern-Price method satisfies rigorous methods because they satisfy all three conditions of equilibrium: force equilibrium in horizontal and vertical direction and moment equilibrium condition (Cheng, 2008). Rigorous methods can provide more accurate results than non-rigorous methods. Spencer’s method is also applicable to virtually all slopes. Spencer’s method also satisfies both moment and force equilibriums and gives factors of safety values very close to those obtained by the Morgenstern-Price method (Duncan & Wright, 2005).

At the end of this analysis the state of condition of embankment slope stability will be determined through a single number called factor of safety. If the slope satisfies the state of equilibrium, the section will be checked for stress-deformation and dynamic analysis otherwise revision of dam zoning will be carried out. According to USACE (2003), the minimum factor of safety with respect to loading condition has been shown in table 3.2 below

Table 3.2 Minimum required factor of safety versus loading condition (USACE, 2003).

Critical Slope	Load condition	Reservoir characteristic	Minimum FoS
Upstream and downstream	During construction and end of construction	Reservoir empty	1.3
Upstream and downstream	Steady state seepage	Reservoir at normal maximum operating level	1.5
Upstream	Draw down	Rapid draw down to critical level	1.3
Down stream	Steady state seepage earth quake	Reservoir at normal maximum operating level	1.1
Upstream			1.1

Providing all the necessary parameters limit equilibrium will compute a factor of safety with the equation Duncan and Wright, (2005) below:

The factor of safety (SF) is calculated as the ratio of the total available shear strength (or resistance) available (s) along a failure surface to the total stress (or driving force) mobilized (τ) along the failure surface.

$$F.S=\frac{\text{Total stress}(S)}{\text{Shear Stength}(\tau)} \quad \ldots\ldots\ldots \quad (3.4)$$

For stability analyses of embankment dams, the recommended factors of safety will vary with loading conditions. Long-term loading conditions (steady seepage) require higher factors of safety while short-term loading conditions (Steady state seepage with earthquake) will require lower factors of safety (USSD, 2007).

3.5.3 Stress-Deformation Analysis

SIGMA/W, component of GEO-STUDIO applied for stress deformation analysis. This package is based on finite element principle. SIGMA/W used to compute stress-deformation with or without the changes in pore-water pressures that arise from stress state. SIGMA/W is formulated for several elastic and elastic-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).

In Geo Studio all boundary conditions must be applied directly on geometry items such as region faces, region lines, free lines or free points. There is no way to apply a boundary condition directly on an element edge or node. The advantage of connecting the boundary condition with the geometry is that it becomes independent of the mesh and the mesh can be changed if necessary without losing the boundary condition specification (SIGMA/W, 2012)..

Young’s Modulus, Poisson’s Ratio, Cohesion, Friction angle of the soil and rock materials used as input parameters (SIGMA/W, 2012). Fundamentally, there are two types of boundary conditions that can be applied in a stress-deformation model: force or displacement. In general, it is common to fix the left and right sides of a problem along with the bottom edge (SIGMA/W, 2012).

3.5.4 Dynamic analysis

QUAKE/W is a part of Geo-Studio and fully integrated with other components. QUAKE/W a 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,2012). Similar procedure to that of SEEP/W, SLOPE/W & SIGMA/W will be used to fix the geometry. There are four different material models to choose from when using QUAKE/W. A summary of these models and the required soil properties are given below:-

I) None (used to removed part of a model in the analysis)

II) Linear elastic model:-The material properties required for this model are Unit weight, Poisson’s ratio, Damping ratio pore-water pressure function, cyclic number function and G constant or function.

III) Linear model:-The material properties required for this model are Unit weight, Poisson’s ratio, cohesion ,angle of friction, damping ratio constant or function, Ksa and Ks functions, pore-water pressure function, reduction function and G max constant or function

IV) Non-linear model:-The material properties required for this model are unit weight, Poisson’s ratio, cohesion, angle of friction, damping ratio and Max damping ratio, pore-water pressure function, recoverable modulus function(G). The simplified input data for the linear elastic model are listed in the table 3.1 with respect to material and the maximum shear modulus (Gmax) is determined by equation 3.5 Gmax is considered to be constant.

$$G_{max} = \frac{E}{2*(1-V)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.5)$$

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

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 type of boundary condition (QUAKE/W, 2012).

3.5.5 Design of earthquake

The earthquake hazard evaluation for the proposed embankment dam for Dodota Irrigation project site (7.3°E and 39.4°N) is based on the method of Maosong (2010) as implemented by Jibson (2011) and has been accepted as a worldwide standard by the Global Seismic Hazard Assessment Program (Morata , 2008).

ICOLD, 1989, whose revision is under way, defines the following seismic design levels for the design of new dams and also for the safety evaluation of existing dams:

1. **Operating basis earthquake (OBE)**:- The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of excedence during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years.) The associated performance requirement is that the project functions with little or no damage, and without interruption of function.
2. **Safety evaluation earthquake (SEE)**:- is that level of shaking for which damage can be accepted but for which there should no uncontrolled release of water from the reservoir. The SEE is normally characterized by a level of motion equal to that expected at the dam site from the occurrence of a deterministically evaluated maximum credible earth quake or of the probabilistically evaluated earthquake ground motion with a very long return period for example 10,000 years.

   This is the proposed terminology in the revised Bulletin 72. Other terms in use are: Maximum Credible Earthquake (MCE) and Maximum Design Earthquake (MDE).

In a conventional pseudo-static, limiting equilibrium, earthquake stability analysis, a horizontal earthquake force is applied to the sliding body in addition to the static forces. The additional horizontal force is proportional to the total mass of the sliding body, and the factor of proportionality is denoted "earthquake coefficient". This type of analysis is applicable only for dams constructed of materials that do not experience a significant reduction in strength during cyclic loading. The dense rock fills, which makes up the bulk of propose embankment dam and the asphaltic concrete core are of this type. The permeability of the rock fill and the transition zones is so great that the excess pore pressures generated during cyclic loading dissipate quickly, and no significant accumulation of pore pressures takes place during an earthquake (ICOLD 2011).

Slopes composed of materials that build up significant dynamic pore pressures during earthquake shaking or that lose more than about 15% of their peak shear strength during shaking are not good candidates for pseudo static analysis (Kramer, 1996). The limitation of pseudo static analysis is that, because it is a limit-equilibrium analysis, it tells the user nothing about what happens after equilibrium is exceeded. According to Feasibility Report, it appears that an earthquake may be expected in the dam zone with peak ground accelerations (PGA). For this thesis consider OBE system for the Earthquake analysis.

Table 3.3: PGA values as per available feasibility report and from the seismic map (Appendix B.4)

Design Earthquake	Horizontal acceleration	Vertical acceleration
Operating Basis Earthquake(OBE)	0.15g	0.075g
Safety Evaluation Earthquake (SEE)	-	-

4. RESULTS AND DISCUSSION

4.1 Seepage Analysis

The Finite Element Models used in the analyses and the computed discharges are shown on respective section. As can be seen from SEEP/W results, phreatic level vertically drops down indicating that the use of asphalt core as a water barrier material is quite effective. It is one of the tasks of design and construction to make the structure functional in the sense that the water is properly drained away and the quantity of drained water is tolerable and small.

The pressure distribution was also resulted as expected with minimum at the top and maximum at the bottom along with the depth of water increment. In the graphs of velocity and hydraulic gradient, the distributions of their values displayed centrally as the total seepage flow through the embankment and foundation determined with a flux section shown below (Figure 4.1). An analysis of the expected quantity of seepage through the embankment and dam foundation for asphaltic concrete core using SEEP/W software model which is the total flow per unit distance into the section is computed as $2.8175*10^{-9} m^3$/sec/m and $2.3321*10^{-7} m^3$/sec/m respectively and total flux section through the dam and foundation is $2.360275*10^{-7} m^3$/sec/m. Considering the crest length of proposed embankment dam for Dodota Irrigation Project approximately 250 m, the total volume of water seeps through the dam body and the foundation is computed to be 0.000059 m^3/sec. An analysis of the expected quantity of seepage through the embankment and dam foundation for clay core using SEEP/W software model which is the total flow per unit distance into the section is computed as $4.06975*10^{-6} m^3$/sec/m and $1.2647*10^{-6} m^3$/sec/m respectively and total flux section through the dam and foundation is $5.3345*10^{-6} m^3$/sec/m. Considering the crest length of proposed embankment dam for Dodota Irrigation Project approximately 250 m, the total volume of water seeps through the dam body and the foundation is computed to be 0.001334 m^3/sec. A leakage of 0.03 m^3/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 for Dodota Irrigation project with central asphaltic concrete core is more acceptable to control leakage trough the dam body than clay core. The end result obtained in this studies as shown below for asphalt concrete core dam, under seepage analysis is approves to the result found by Wang and Hoeg in 2009, and finally they concluded that “Among all the dams built, there is no case of reported leakage through an asphalt concrete core Dam” (Wang and Hoeg, 2009).

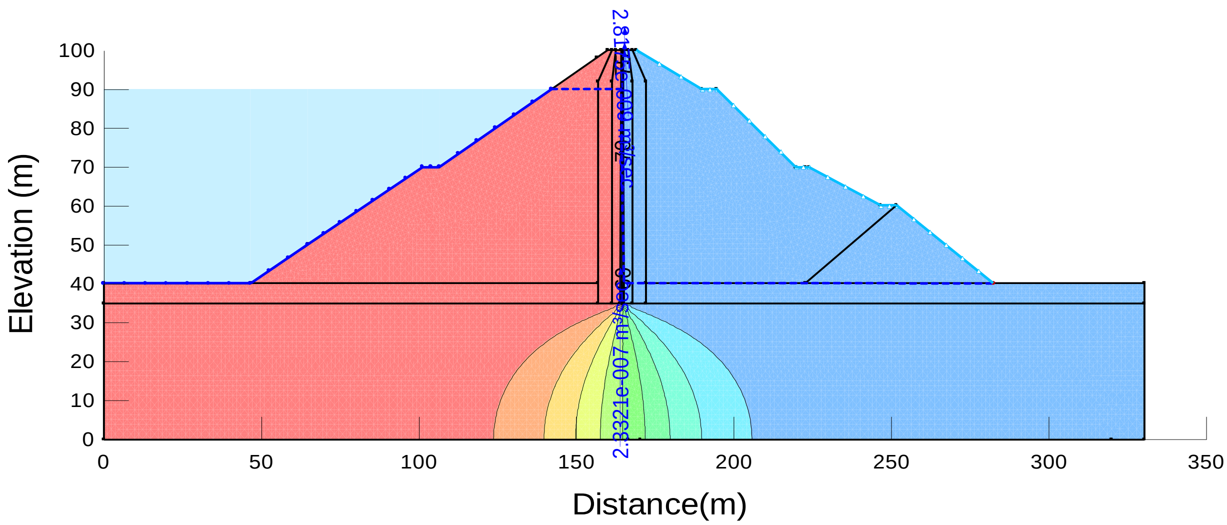

Figure 4.1: Seepage through the dam body and foundation asphaltic core dam

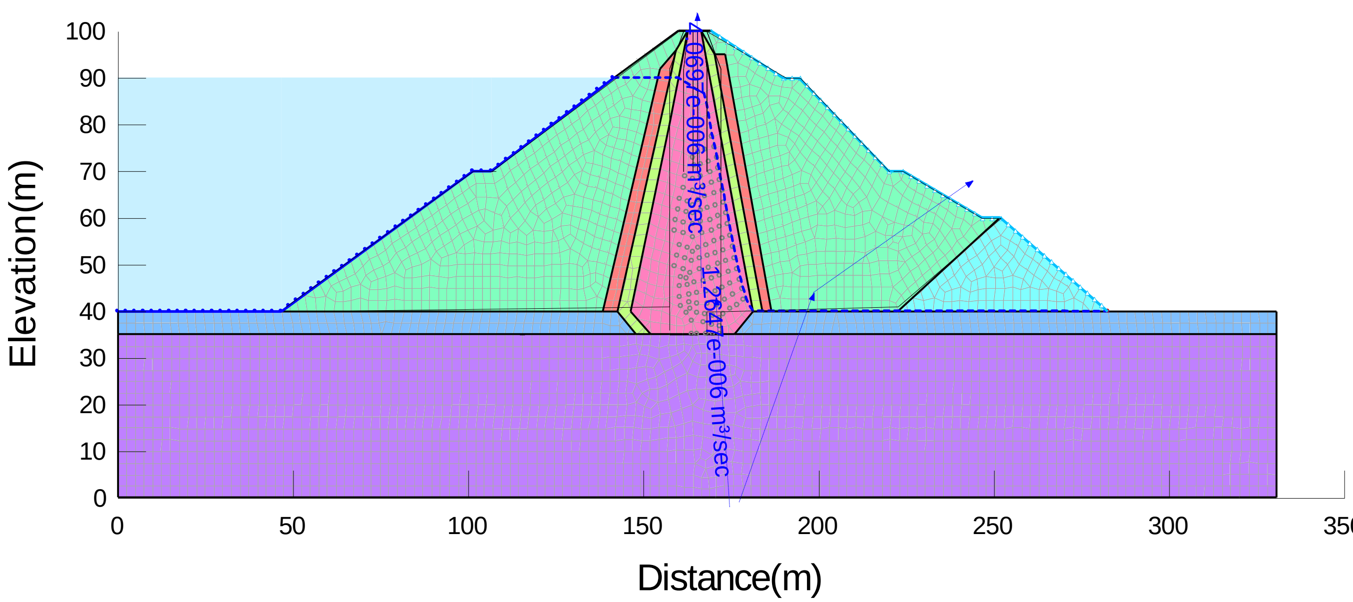

Figure 4.2:Seepage through the dam body and foundation through CC dam

As can be seen from SEEP/W results in Figure 4.3, the blue thick line indicates that the line of zero pressure which rapidly drops from asphaltic concrete top section to bottom section then to toe section. The distribution of pore water pressure through the dam body and the foundation at steady state condition is also shown in Figure 4.3. The negative sign indicates that the portion of the dam above phreatic line is a negative/suction pressure zone which means dry area.

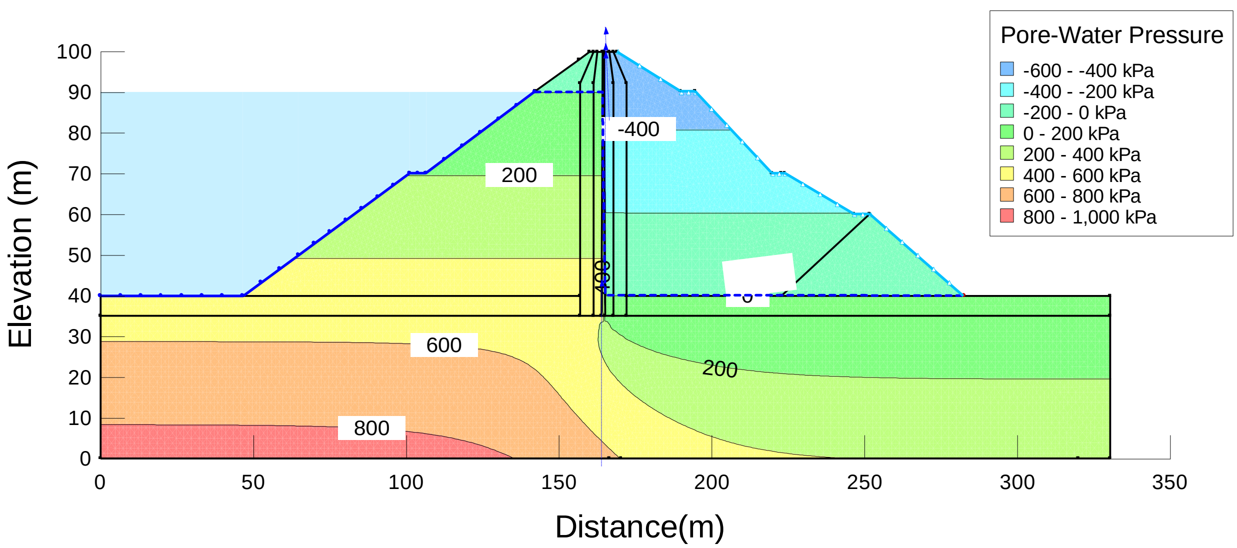

Figure 4.3: Contour of pore water pressure

4.2 Slope Stability Analysis

Evaluating the stability of the upstream and downstream slopes of dam structures is one of the most important issues in dam engineering. A stable slope under static conditions is thought to have resistance to sliding which is greater than the driving forces that exist due to the slope geometry. However, seismic loading of a slope can induce greater driving forces that will potentially make a once stable slope unstable. For the slope stability analysis for seismic load for the propose embankment dam the horizontal acceleration (α_h) value was selected as 0.15 for the analysis. The vertical acceleration (α_v) value was applied as 50% of the horizontal acceleration (α_h) value = 0.075.

4.2.1 Slope Stability during construction

The stability of both upstream and downstream slopes during construction has been analyzed for the propose embankment dam without Earthquake and with Earthquake. A minimum factor of safety for

Rock fill asphaltic concrete core dam downstream and upstream without Earthquake 1.791 (Figure. 4.4) and 1.644 (Figure A.1) respectively and the factor of safety to be done if the Earthquake happen during construction the result with seismic loading condition downstream and upstream 1.334 (Figure. 4.6) and 1.211(Figure A.3) is indicated respectively. A minimum factor of safety for Rock fill clay core dam downstream and upstream without Earthquake 1.791 (Figure. 4.5) and 2.274 (Figure A.2) respectively and the factor of safety to be done if the Earthquake happen during construction the result with seismic loading condition downstream and upstream 1.399 (Figure. 4.7) and 1.661 (Figure A.4) is indicated respectively. From the analysis result show that the minimum factor of safety is greater than the recommended minimum value for the two conditions which is 1.3 without Earthquake and 1.1 with Earthquake suggested by (USACE, 2003). The asphaltic concrete rock fill dam and clay core rock fill dam is safe during construction condition according to (USACE, 2003).

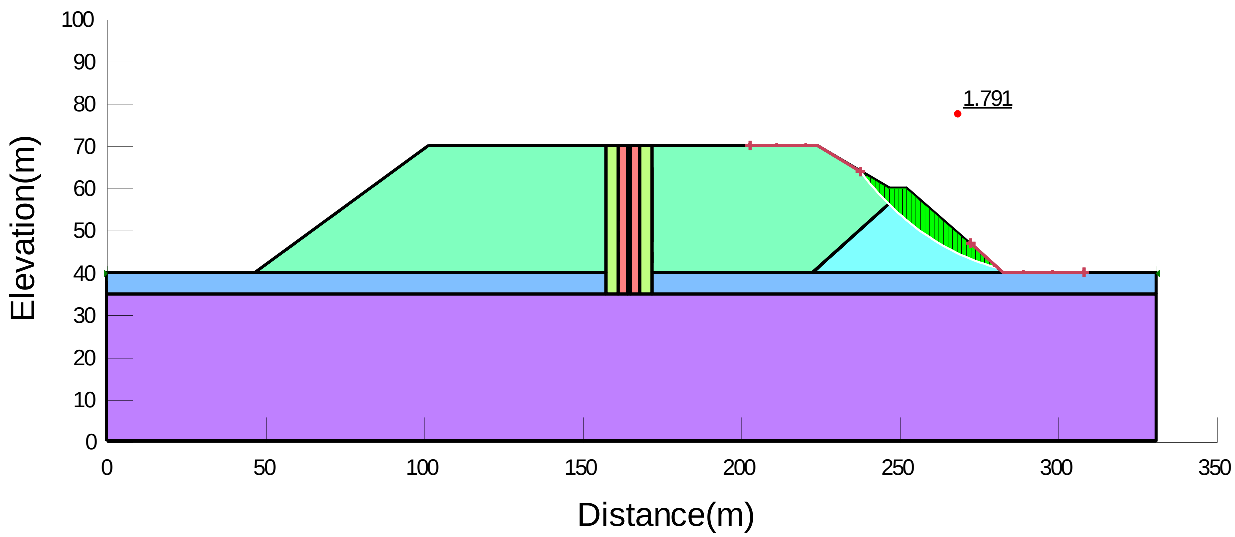

Figure 4. 4: Slope stability during construction without Earthquake AC (D/S)

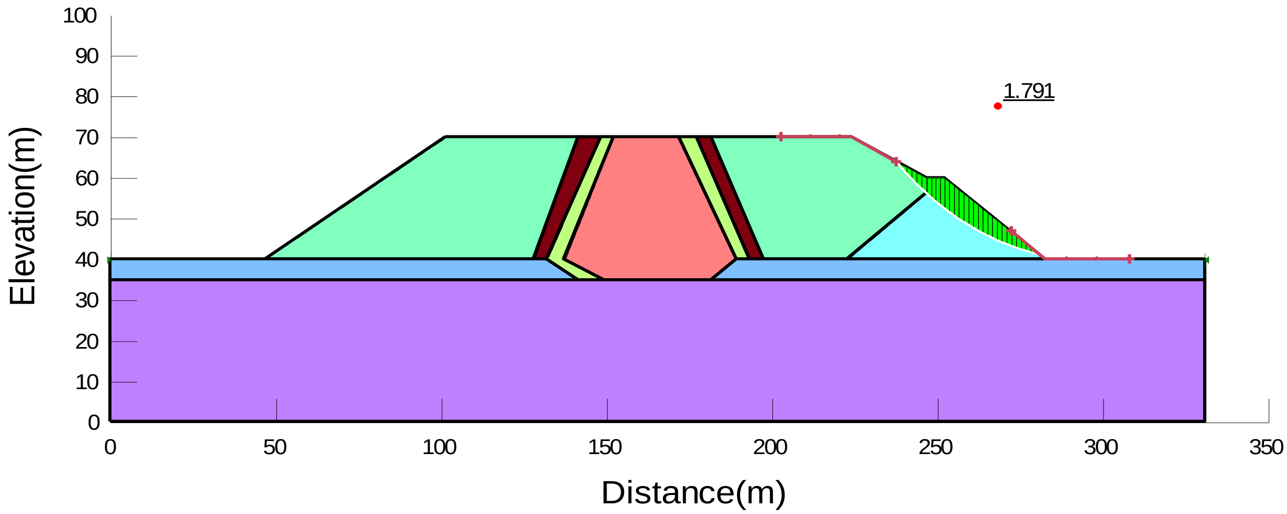

Figure 4.5 : Slope stability during construction without Earthquake CC (D/S)

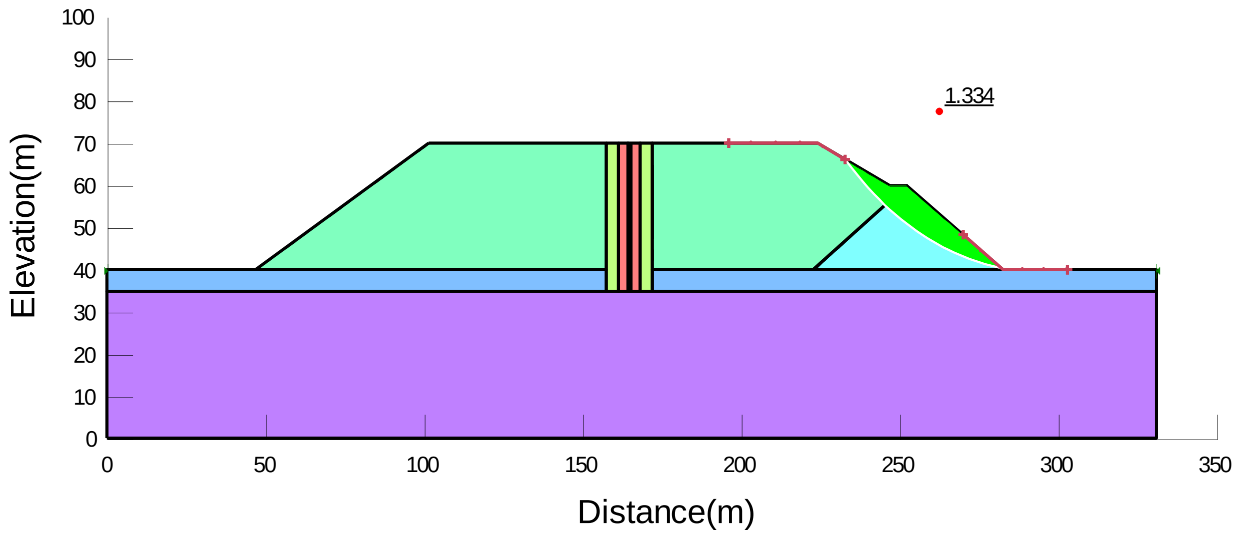

Figure 4. 6: Slope stability during construction with Earthquake AC (D/S)

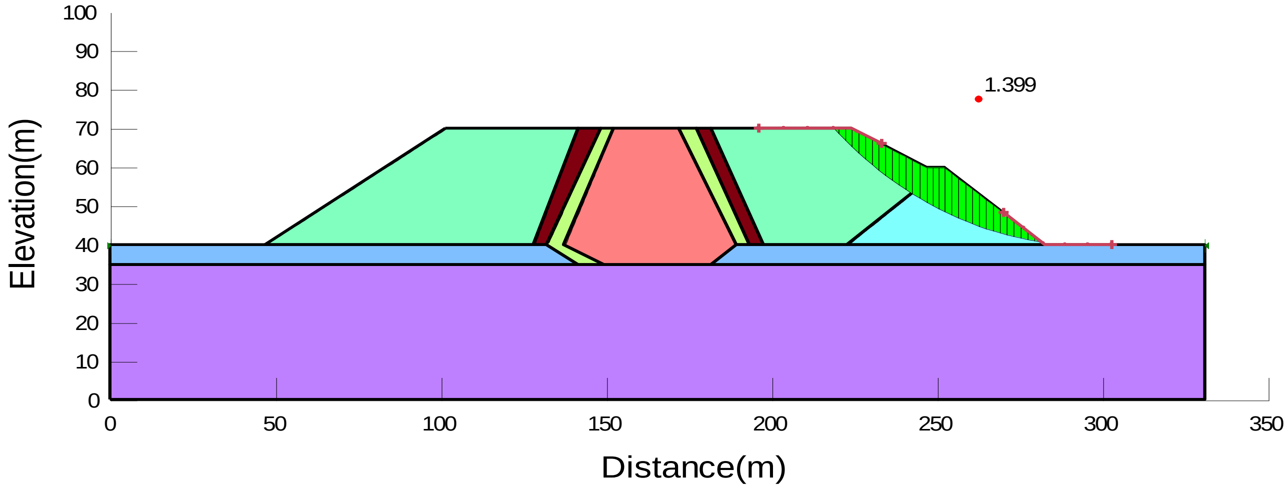

Figure 4.7: Slope Stability during construction with Earthquake CC (D/S)

4.2.2 Slope Stability at end of construction

The stability of both upstream and downstream slopes during end of construction has been analyzed for the propose embankment dam without Earthquake and with Earthquake. A minimum factor of safety for the propose asphaltic concrete rock fill dam downstream and upstream without Earthquake 1.756 (Figure. 4.8) and 1.747 (Figure 4.10) respectively and the factor of safety to be done if the Earthquake happen at the end of construction the result with seismic loading condition downstream and upstream 1.246 (Figure A.5) and 1.286 (Figure A.7) is indicated respectively. A minimum factor of safety for the propose asphaltic concrete rock fill dam downstream and upstream without Earthquake 1.767 (Figure. 4.9) and 1.761 (Figure 4.11) respectively and the factor of safety to be done if the Earthquake happen at the end of construction the result with seismic loading condition downstream and upstream 1.297 (Figure A.6) and 1.298 (Figure. A.8) is indicated respectively. From the analysis result show that the minimum factor of safety is greater than the recommended minimum value which is 1.3 without Earthquake and 1.1 with Earthquake. The critical factor of safety satisfied at end of construction for the propose embankment as the standard of (USACE, 2003). The propose embankment dam rock fill with asphaltic concrete core rock fill clay core also stable in stability analysis at end of construction with earthquake and without earthquake the standard sate by (USACE, 2003).

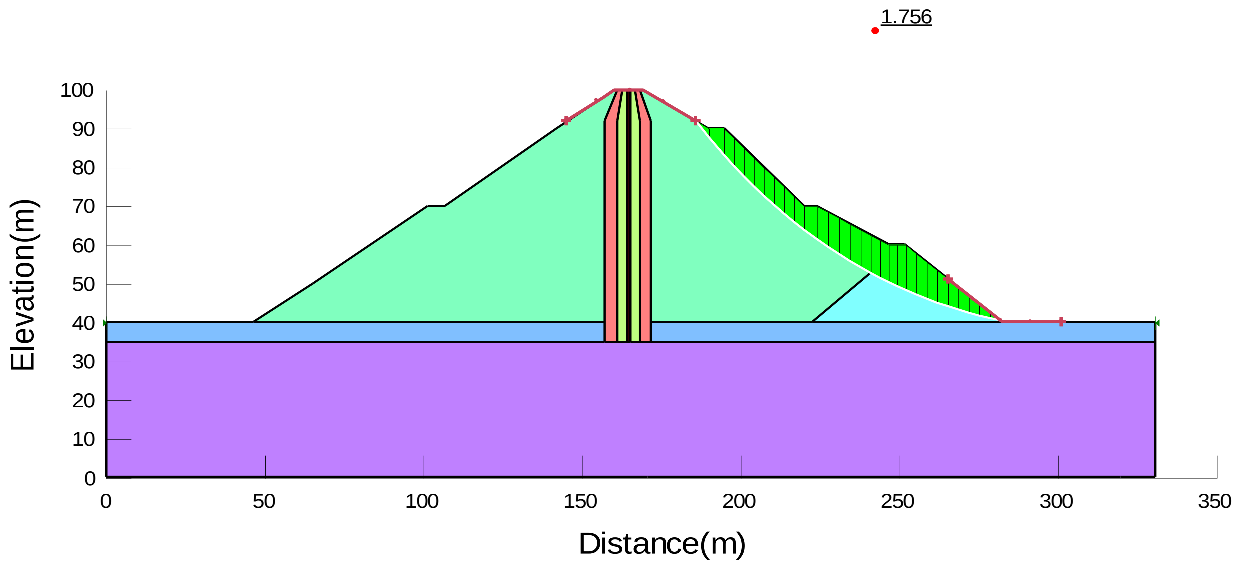

Figure 4. 8: Slope Stability at end of construction without Earthquake AC (D/S)

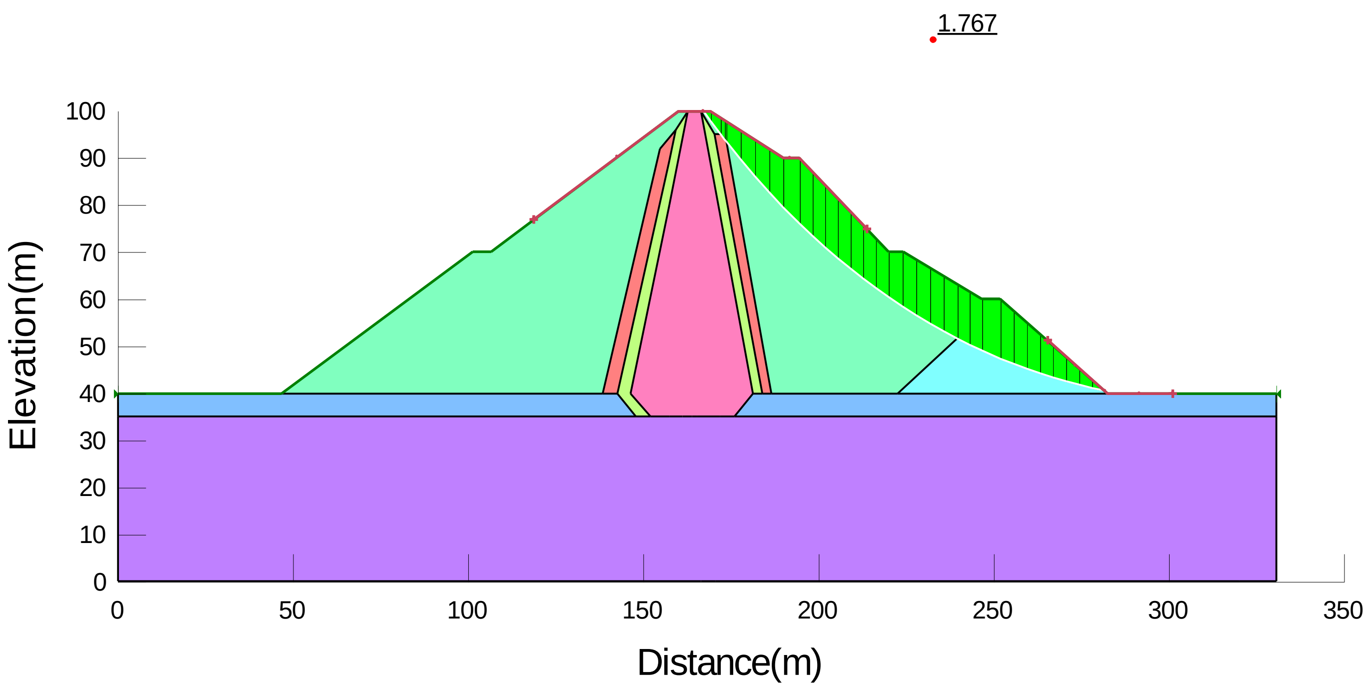

Figure 4.9: Slope Stability at end of construction without Earthquake CC (D/S)

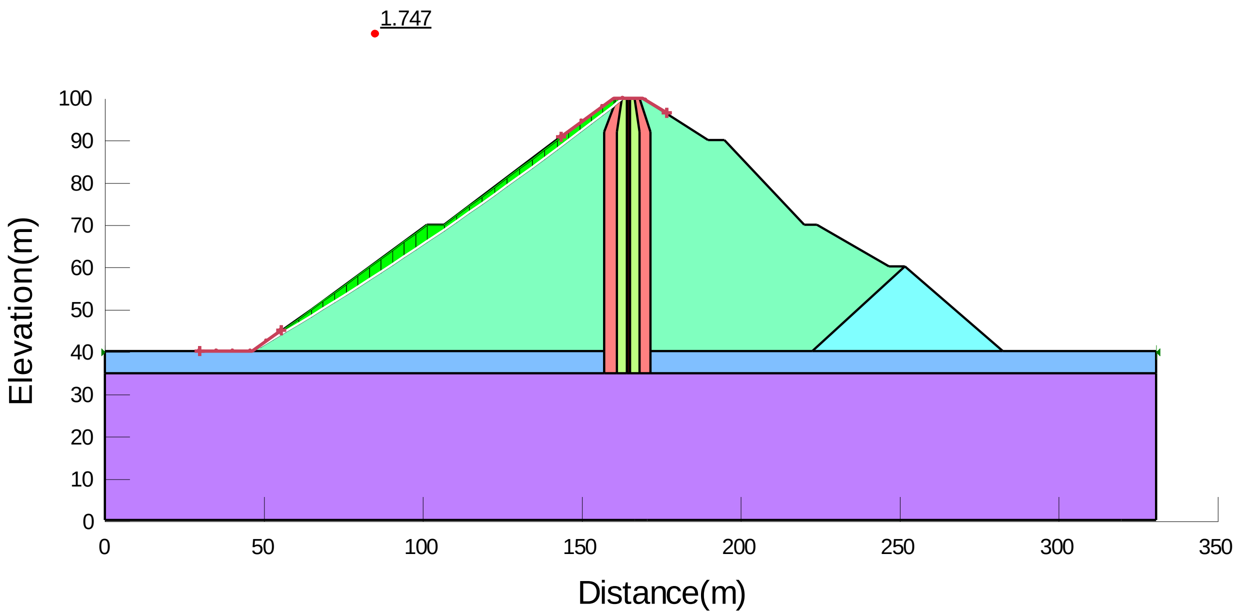

Figure 4. 10: Slope Stability at end of construction without Earthquake AC (U/S)

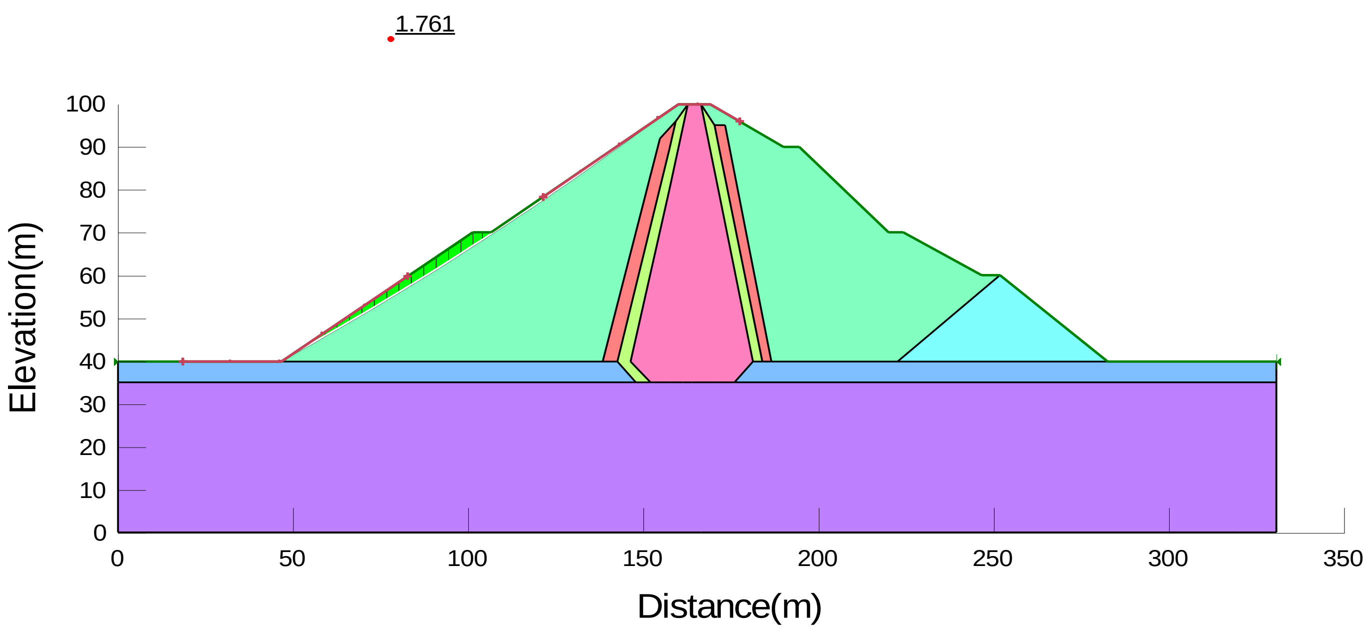

Figure 4. 11: Slope Stability at end of construction without Earthquake CC (D/S)

4.2.3 Slope stability under steady state flow condition

The stability of both upstream and downstream slopes during for steady state condition when the reservoir is at normal pool level factor of safety has been analysis for the propose embankment dam without Earthquake and with Earthquake. A minimum factor of safety for steady state downstream and upstream without Earthquake 1.61 (Figure 4.12) and 1.75 (Figure 4.14) respectively and the factor of safety to be done if the Earthquake happen at the end of construction the result with seismic loading condition downstream and upstream 1.184 (Figure A.9) and 1.081 (Figure A.11) is indicated respectively. A minimum factor of safety for steady state downstream and upstream without Earthquake 1.736 (Figure 4.13) and 1.698 (Figure 4.15) respectively and the factor of safety to be done if the Earthquake happen at the end of construction the result with seismic loading condition downstream and upstream 1.261 (Figure A.10) and 1.061 (Figure A.12) is indicated respectively. From the analysis result show that the minimum factor of safety is greater than the recommended minimum value which is 1.5 without Earthquake and 1.1 with Earthquake. From the result the propose embankment dam of rock fill with asphaltic concrete dam and clay core is safe under steady state flow condition under usual and unusual loading condition according to the stability under steady state flow condition of (USACE, 2003). The result obtained clearly shows that, slope stability during steady state loading condition due to upstream water pressure and its distribution in embankment material during earthquake give minimum result than slope stability at end of construction this indicate the reservoir water pressure have effect on stability due to asphalt concrete core attractive result is attain the minimum safety factor requirement recommended by (USACE, 2003).

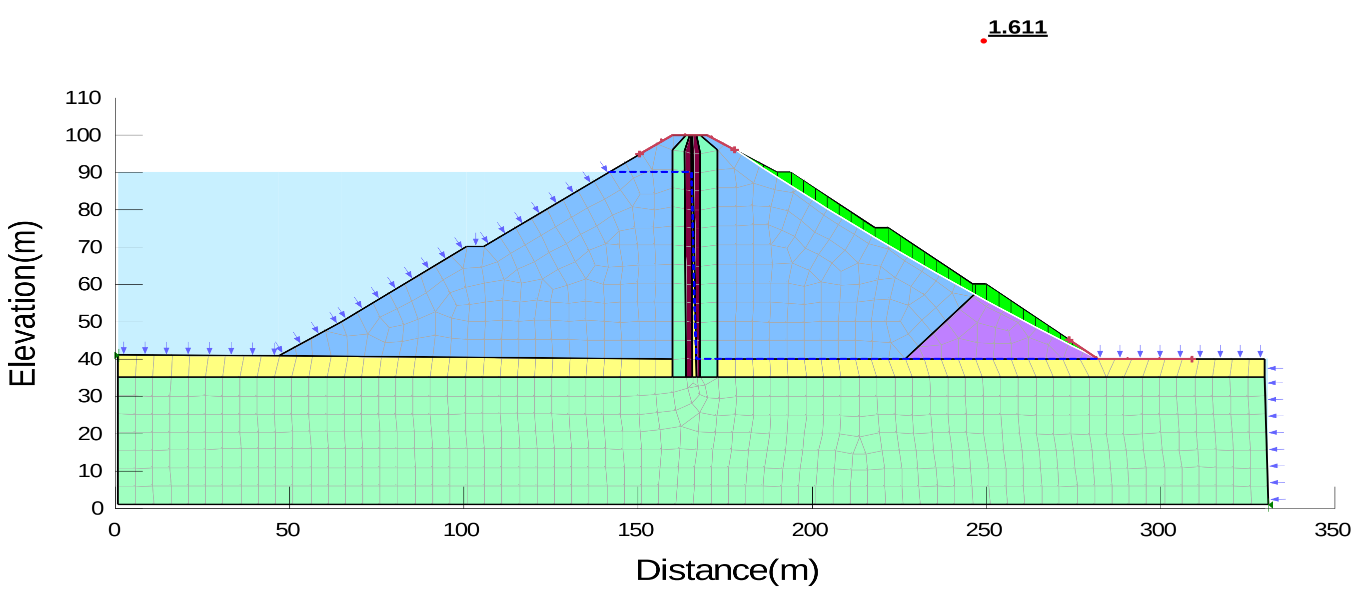

Figure 4.12: Factor of safety at Steady state without earthquake AC (D/S)

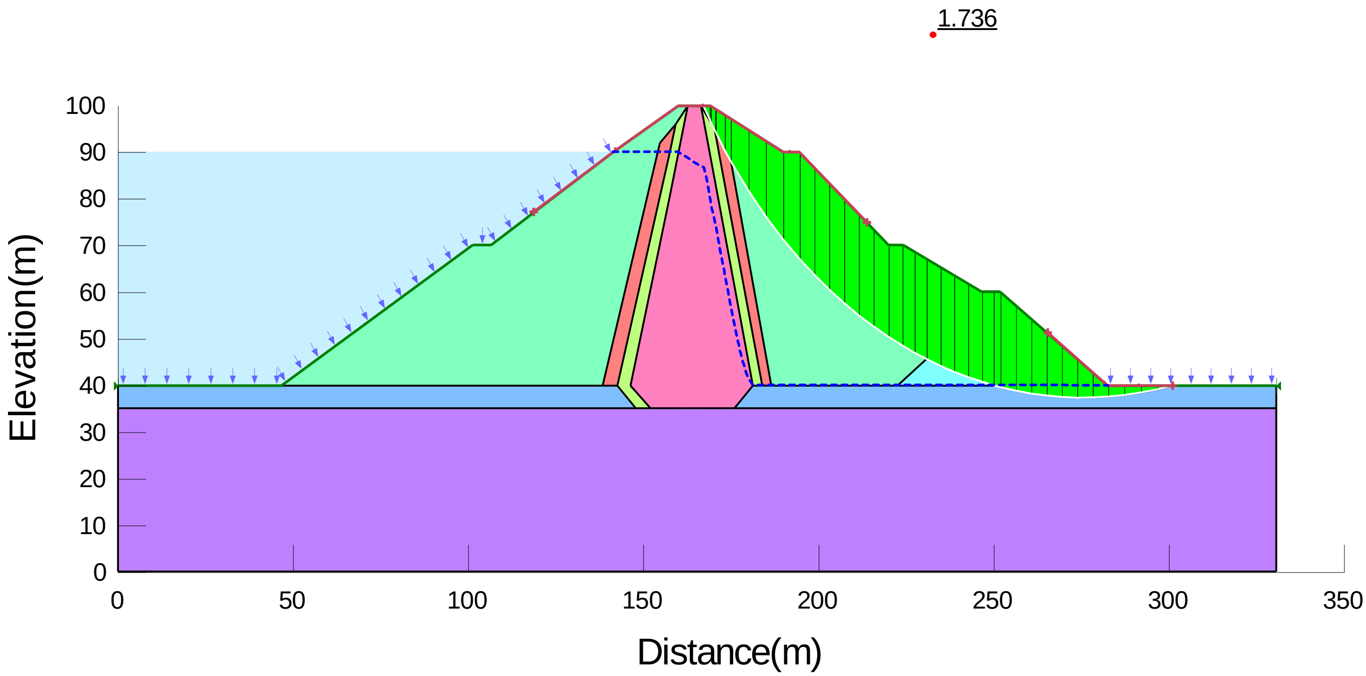

Figure 4. 13: Factor of safety at Steady state without earthquake CC (D/S)

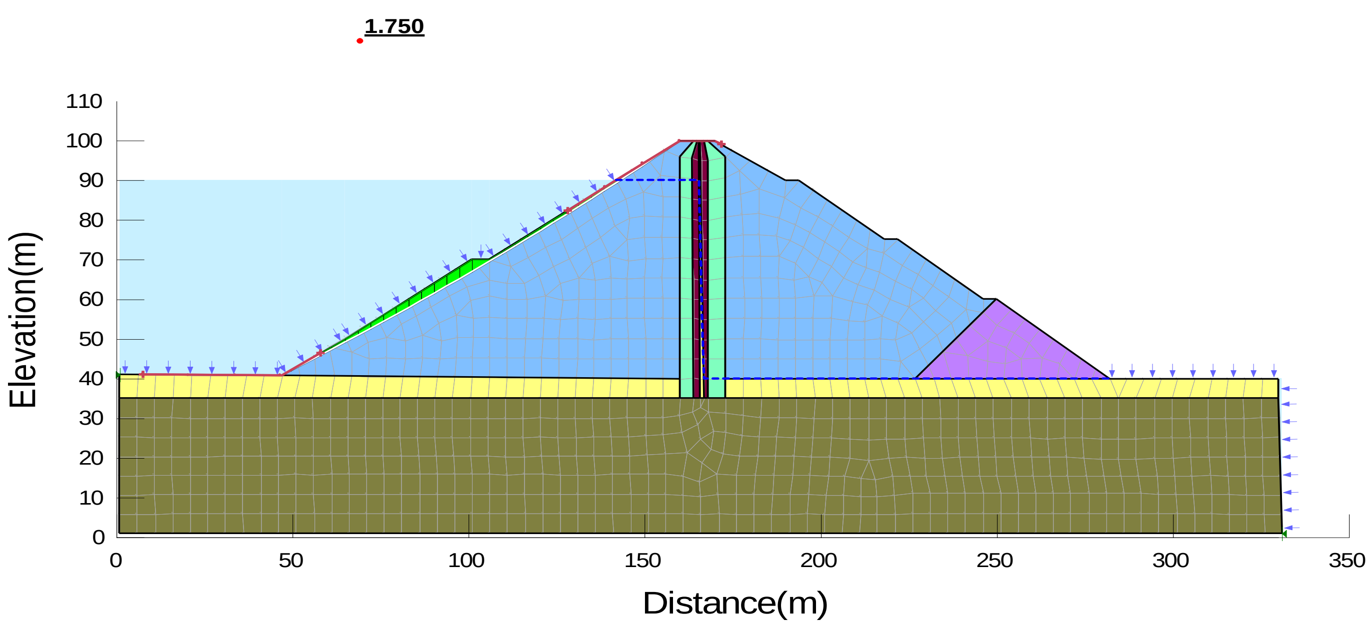

Figure 4. 14: Factor of safety at Steady state without earthquake AC (U/S)

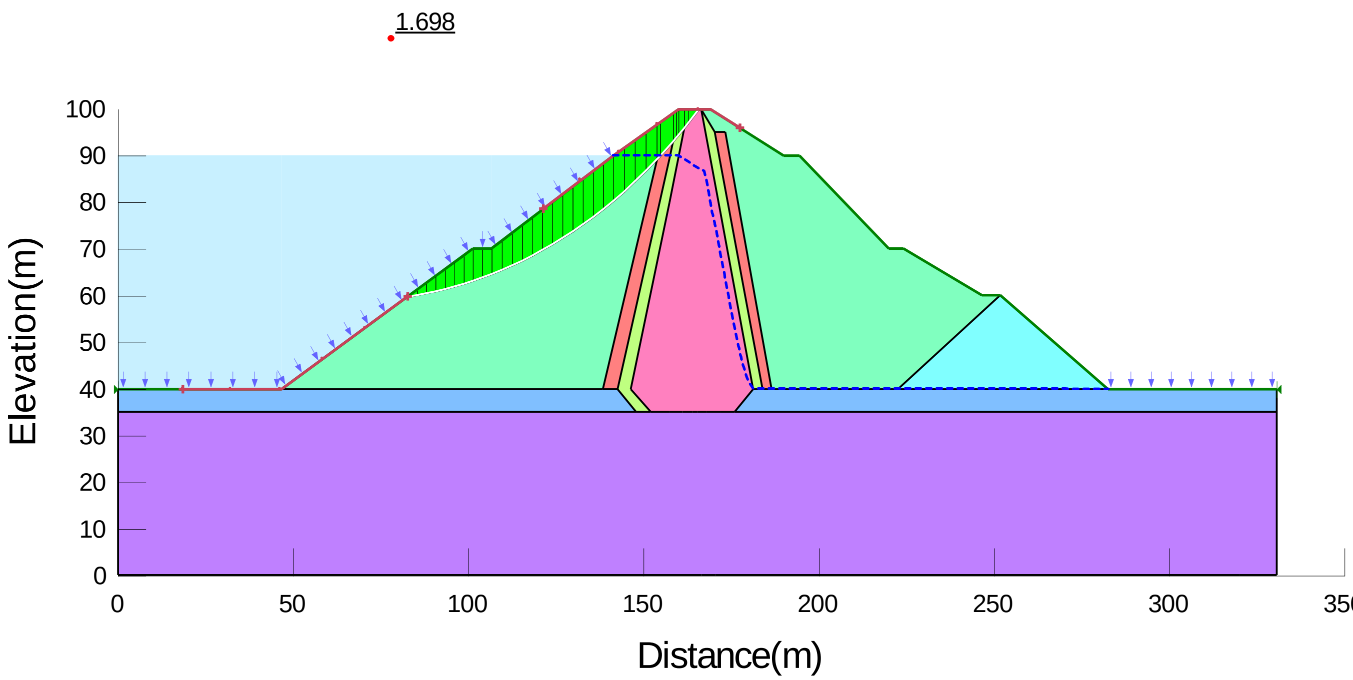

Figure 4. 15: Factor of safety at Steady state without earthquake CC (U/S)

4.2.4 Slope Stability under sudden drawdown/transient flow condition

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. The required initial pore water pressure just before sudden draw down has been taken from the steady state condition. The propose dam is no more affected by draw dawn condition since the command area is supplied water through the canal. The draw dawn condition from 90 m (normal pool level) head to minimum draw dawn 70 m head. The required boundary condition has been attached and the phreatic line of pore water dissipation has been analyzed and the phreatic level drops with increasing time of analysis as shown in Figure 4.16, as pore- water pressure get time to dissipate.

The stability has been analyzed for 30 days since draw down with an interval 0.25 day (6 hours) and the factor of safety critical computed at the possible immediate time after the draw down has been found to be 1.247 (Figure 4.16) which is near to the adequate as compared to the minimum requirement 1.3 according to USACE, (2003) for this loading condition. The factor of safety of clay core condition better alternative during draw dawn case with minimum factor of safety 1.625 (Figure 4.17) satisfied the minimum requirement of according to (USACE, 2003).

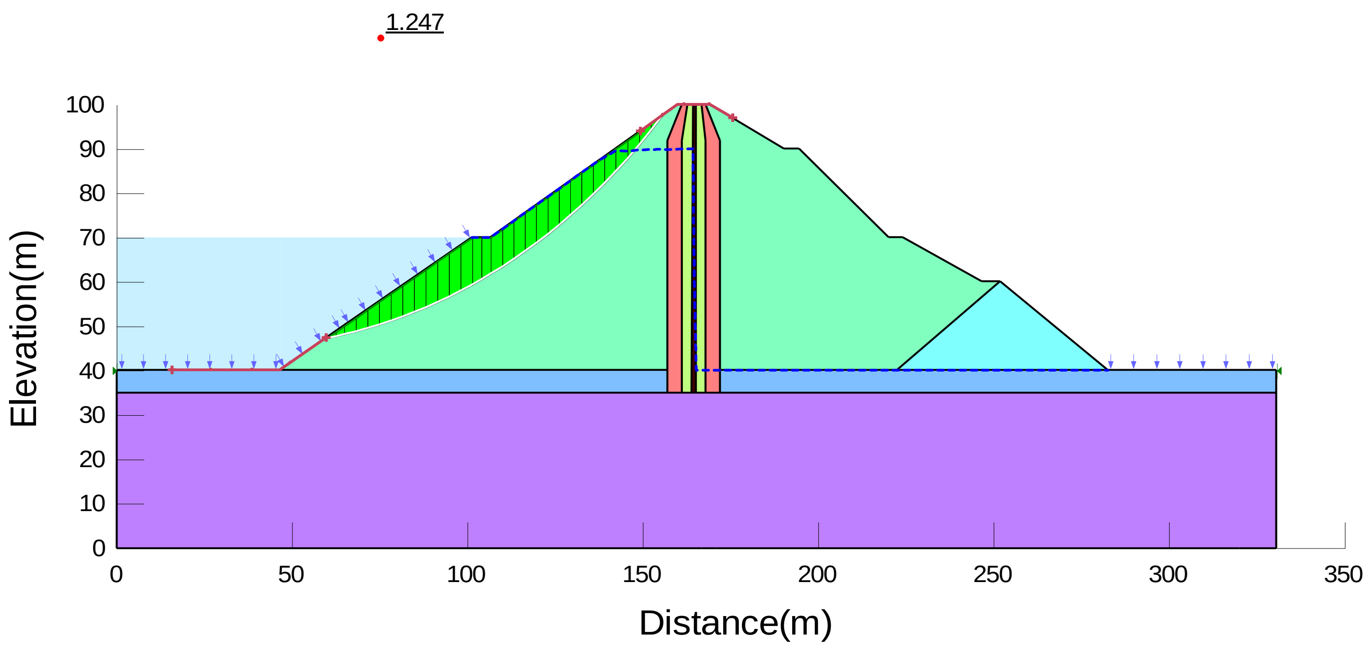

Figure 4.16: Factor of safety for critical time after sudden drawdown AC

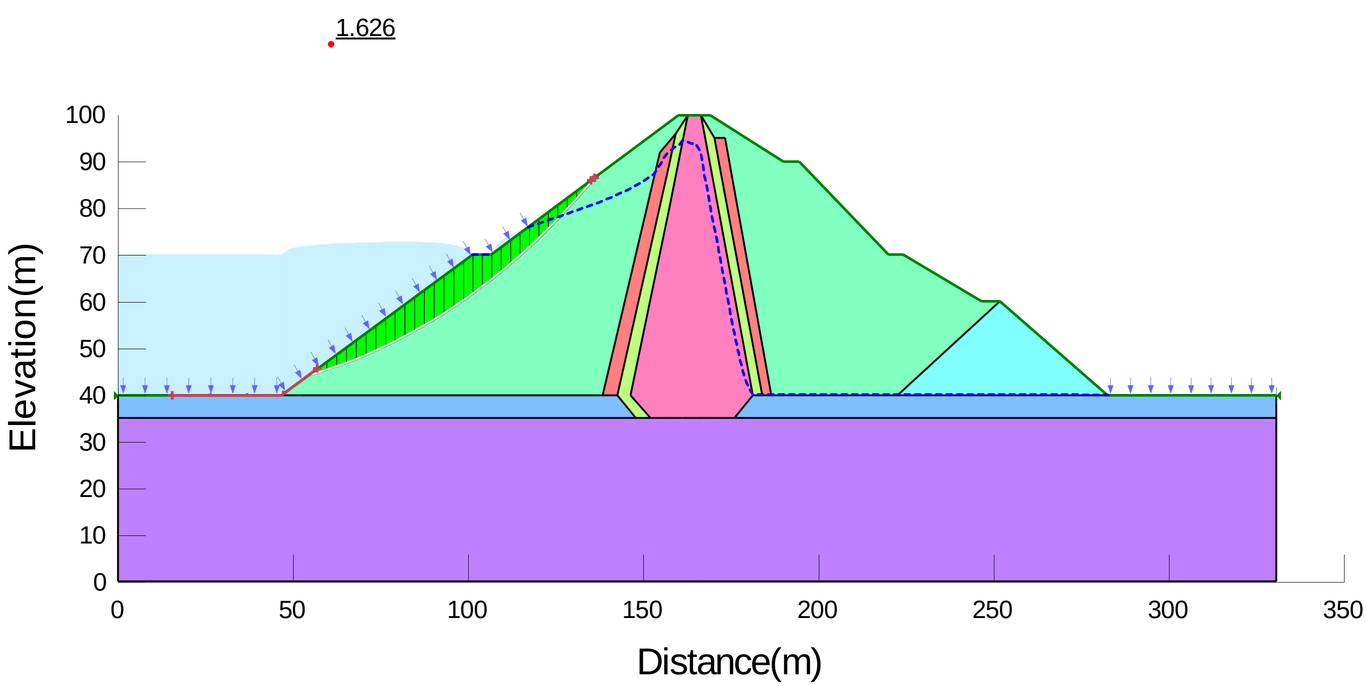

Figure 4. 17: Factor of safety for critical time after sudden drawdown CC

Table 4.1: Recommended and computed factor of safety for different loading condition AC condition

Loading condition	Recommended FoS for U/S and D/S(USACE,USBR, NRCS and FERC)	Factor of safety (FoS) computed		
		Upstream	Downstream	Remark
During construction without Earthquake	1.3	1.644	1.791	Ok
During construction with Earthquake	1.1	1.211	1.334	Ok
End of construction without Earthquake	1.3	1.747	1.756	Ok
End of construction with Earthquake	1.1	1.286	1.246	Ok
Steady state without Earthquake	1.5	1.75	1.611	Ok
Steady state with Earthquake	1.1	1.081	1.184	Ok
Transient state	1.1 to 1.3	1.247	-	Ok

Table 4.2: Recommended and computed factor of safety for different loading condition CC condition

Loading condition	Recommended FoS for U/S and D/S(USACE,USBR, NRCS and FERC)	Factor of safety (FoS) computed		
		Upstream	Downstream	Remark
During construction without Earthquake	1.3	1.761	1.767	Ok
During construction with Earthquake	1.1	1.298	1.297	Ok
End of construction without Earthquake	1.3	2.274	1.791	Ok
End of construction with Earthquake	1.1	1.661	1.339	Ok
Steady state without Earthquake	1.5	1.698	1.736	Ok
Steady state with Earthquake	1.1	1.261	1.061	Ok
Transient state	1.1 to 1.3	1.626	-	Ok

According to Kutzner, 1997 with a homogeneous dam on an equivalent foundation the slope itself is the critical slip plane, provided the shear strength is given by friction only. In contrast, the critical plane cuts through the dam if the shear strength is given by friction and cohesion. With a zoned dam with earth core on a foundation of equal or higher shear strength, the critical slip plane touches the foundation line. In any cases if the foundation is stiffer than the embankment the slip plane may not touch the foundation line depending of the rigidity of the foundation.

As upstream dam body is subjected to varying stress and strain conditions due to reservoir operation, the use of different materials there is limited. Earthquake demands free draining materials all over the upstream shell at a permeability of k ≥ 10-2 m/s (Kutzner, 1997). The variations in pore water pressure at different condition strength the use of free draining material to alleviate the problems caused by this pore water pressure development.

Embankment stability during earthquake loading has been assessed by performing by a horizontal force (0.15) and vertical force (0.075) (seismic coefficient) is applied to the embankment to simulate earthquake loading to determine the critical (yield) acceleration required to reduce the factor of safety to not less than unity or one. For all the above condition on the propose embankment dam rock fill with asphaltic concrete core safe in slope stability analysis with earthquake according to different literature of (USACE, 2003), (USBR, 1990), (ICOLD, 2011).

4.3 Settlement Analysis

4.3.1 Static analyses

Static analysis of an embankment dam is carried out by considering mainly self-weight and hydrostatic force in the analysis. Contours of compressive stress for the gravity loading are shown in figure below and observed that variation of compressive stress in to the dam body, these stresses are varying from top to bottom. The effective vertical stress and shear stresses computed in the dam body and foundation are 1489kpa and 613kPa respectively (Figure 4.18 and 4.20) with asphaltic concrete rock fill. The effective vertical stress and shear stresses computed in the dam body and foundation are 2980kpa and 1185kPa respectively (Figure 4.19 and 4.21) with clay core rock fill dam. The maximum vertical stress and effective vertical stress produced by the embankment are located at the vesicular sound bed rock foundation, this stress level is much lower than the expected bearing capacity of the foundation rock and maximum shear stress is located at the bottom of asphaltic concrete core at Sound bed rock foundation and minimum of stress is occurred in highest part of the dams. For rock layers allowable bearing capacities are computed based on (Ethiopian Building Code of Standards, EBCS, 1995).The minimum bearing capacity of the foundation rock is 7Mpa to 49Mpa for the propose embankment dam from laboratory result of different test pit (KOICA, 2015). From result of the structure on foundation no problem of settlement and sliding of the foundation since asphaltic concrete core rock fill dam and clay core rock fill is recommended for the Keleta River. From the above result rock fill with asphaltic concrete less bearing capacity on foundation than clay core rock fill dam .

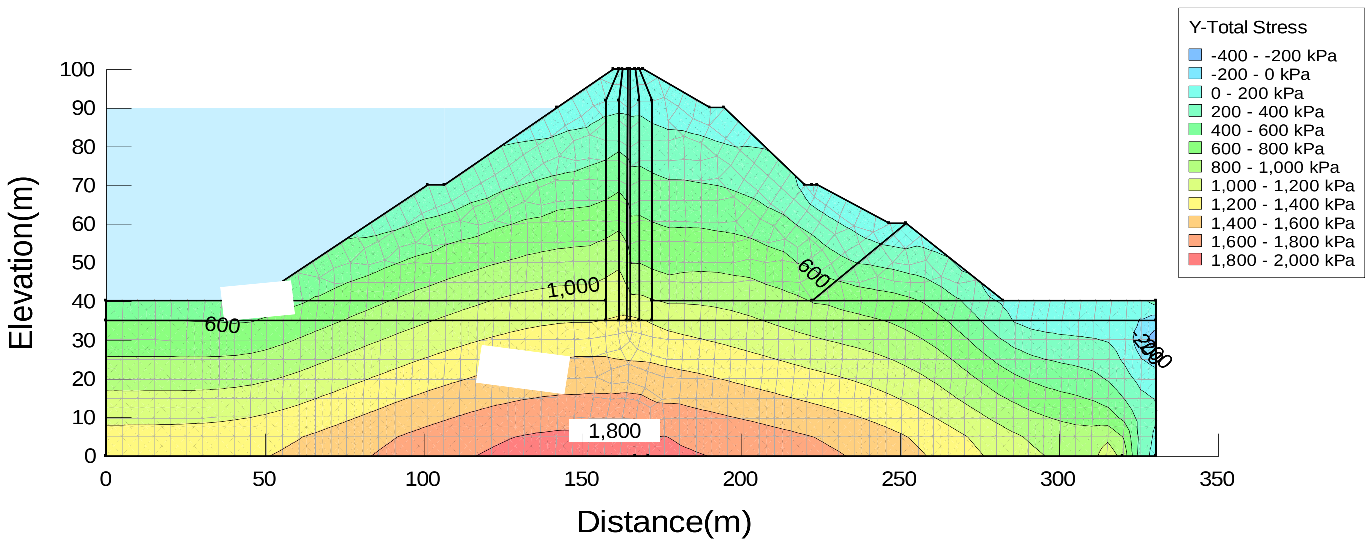

Figure 4. 18: Computed vertical stresses in dam body and foundation (static analysis)

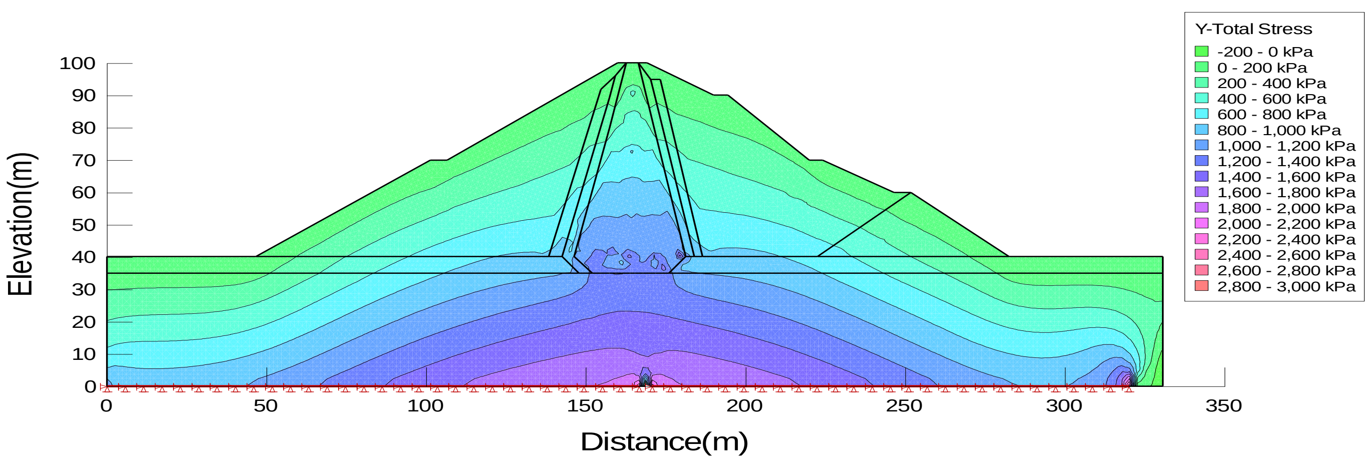

Figure 4. 19: Computed Maximum vertical stresses in dam body and foundation CC (static analysis)

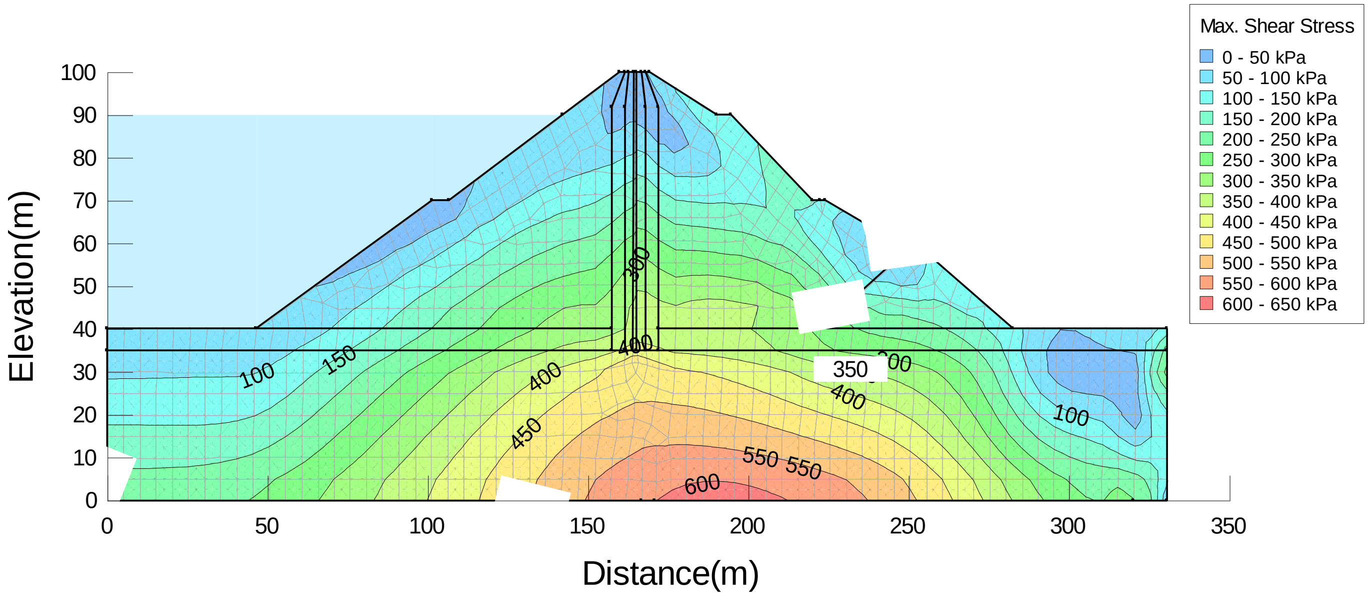

Figure 4. 20: Computed Maximum shear stress in dam body and foundation AC (static analysis)

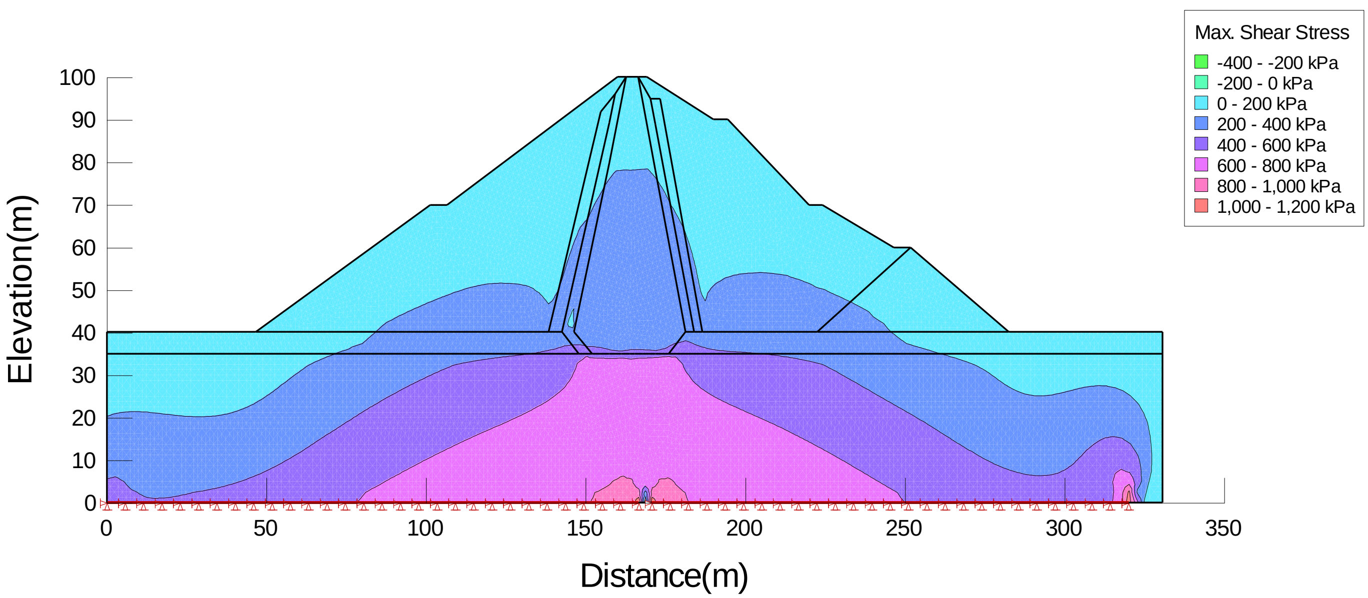

Figure 4. 21: Computed Maximum shear stress in dam body and foundation CC (static analysis)

Because of its flexible and ductile nature of asphalt core, the maximum vertical displacement of asphalt core dam of magnitude 0.5939 m will not cause significant problem, as it do in clay core dam. Besides to Hoeg,(2013) and Fang and Liu, (2011), Static analysis of an embankment dam is carried out by considering mainly self-weight and hydrostatic force in the analysis. As the impervious layer ranges from 0.5 m at the top to 1.2 m at the bottom, the horizontal displacement must be at most less than 0.5 m in order not to have cracking through the core. The maximum vertical displacement-0.2647 shows on Figure 4.22 and vertical displacements were 0.1397m indicates on Figure 4.24 for static condition respectively with AC. The maximum vertical displacement-1.55 shows on Figure 4.23 and vertical displacements were 0.635 m indicates on Figure 4.25 for static condition respectively with AC The maximum vertical displacement occurs at top of crest level.

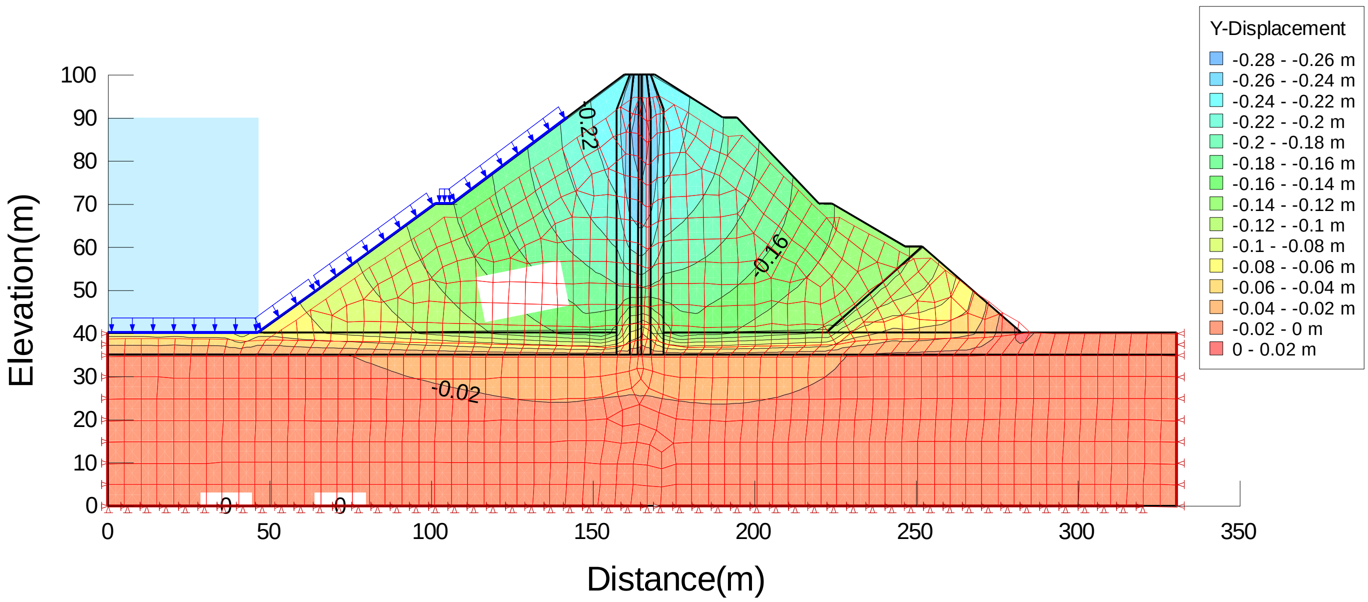

Figure 4. 22 : Contours of vertical displacement for static condition AC

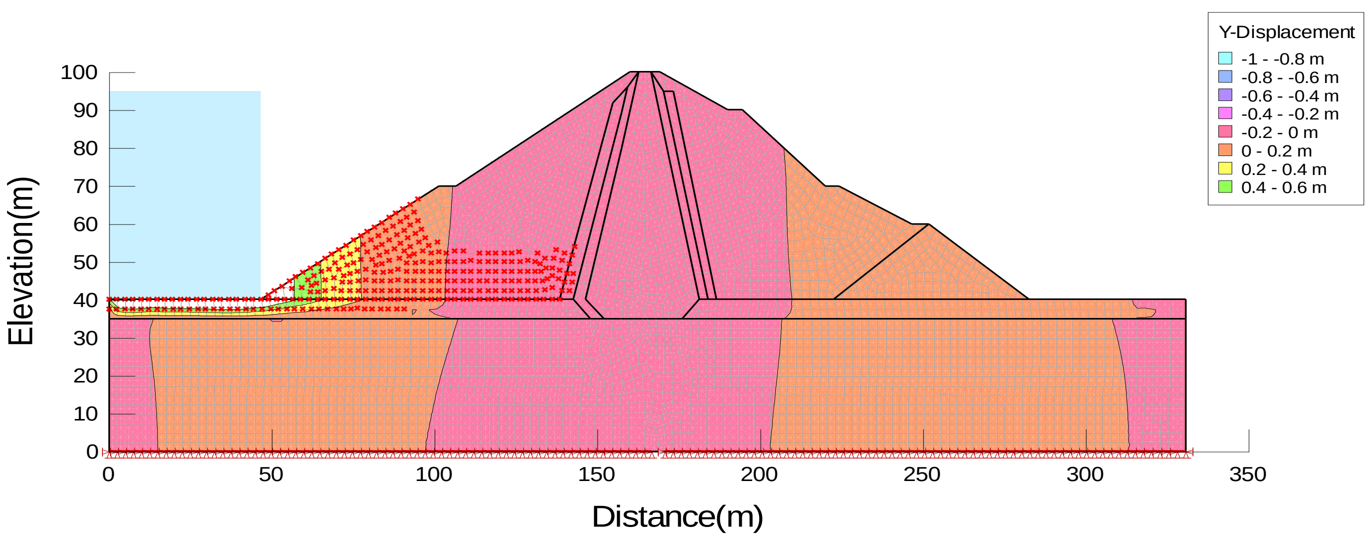

Figure 4. 23: Contours of vertical displacement for static condition CC

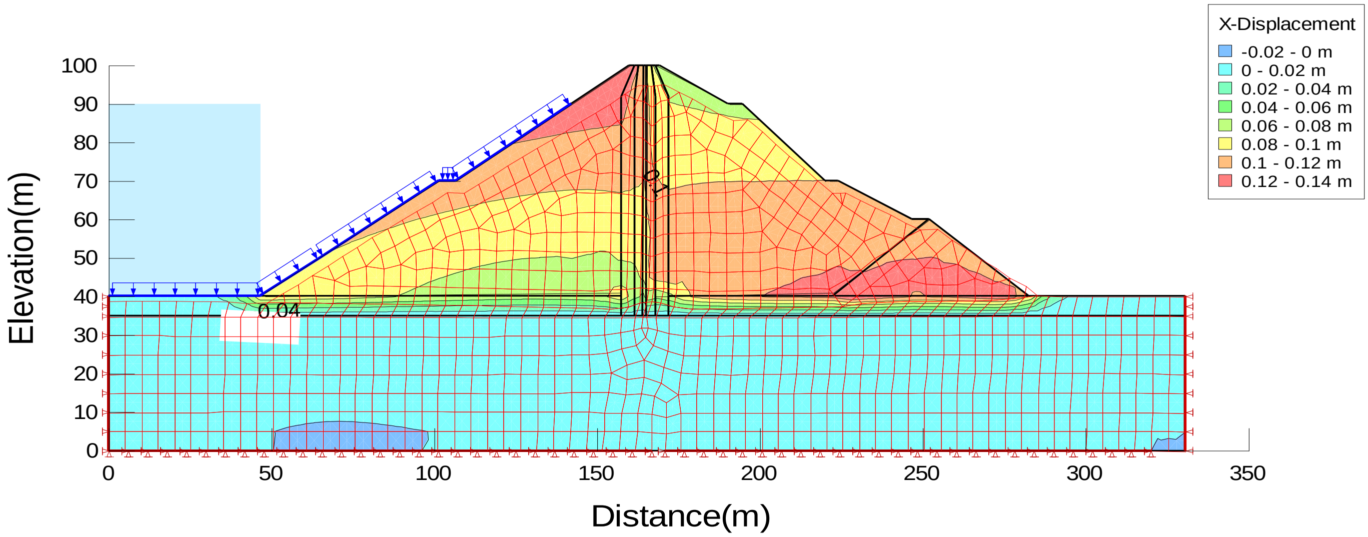

Figure 4. 24: Contours of horizontal displacement for static condition AC

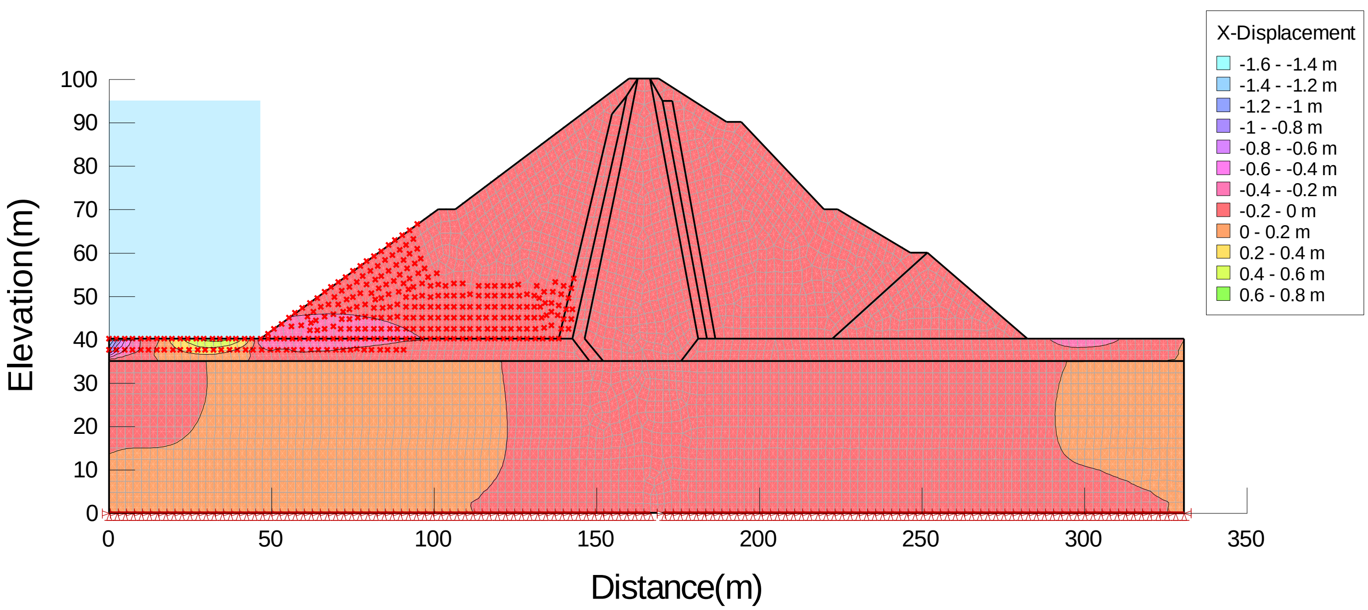

Figure 4. 25 : Contours of horizontal displacement for static condition CC

4.3.2 Dynamic analyses

QUAKE/W can be used for dynamic analysis of embankment dam subjected to earthquake shaking or for dynamic analysis of point dynamic forces from a sudden impact load. QUAKE/W determines the motion and excess pore-water pressures that arise due to shaking. The Earthquake shaking computed by QUAKE/W for vertical and horizontal displacement is calculated.

As the impervious layer ranges from 0.5 m at the top to 1.2 m at the bottom, the horizontal displacement must be at most less than 0.5 m in order not to have cracking through the core. The maximum horizontal and vertical displacements were 0.052 m (Figure 4.26) and 0.0045m (Figure 4.27) for dynamic condition respectively at time of 10 second shaking for AC condition and The maximum horizontal and vertical displacements were 0.0512 m (Figure 4.25) and 0.0006m (Figure 4.28) for dynamic condition respectively at time of 10 second shaking for CC condition. The maximum horizontal and vertical displacement is at the time of ten second shaking. In general, according to the ICOLD literature, under the earthquake shaking the crest settlements of embankment dams should not exceed 1 or 2% of the dam height. From the static and seismic analysis in Quake/W, it can be concluded that seismic loading has a notable effect on the deformation of the dam. As seen from the dynamic analysis, the maximum shear deformations at the dam body and foundation are under 0.1%. These results show that a seismic loading is unlikely to present a serious problem in terms of the overall stability of the dam. Often it is assumed that for strong rock fills the settlements are small, less than about 1% of the fill height (Jafari, 2014). It is further commonly assumed that the settlements develop essentially during construction. Although these assumptions were correct for many dams, a few dams did show much more settlements (Jafari, F., and Salmasi, 2014).

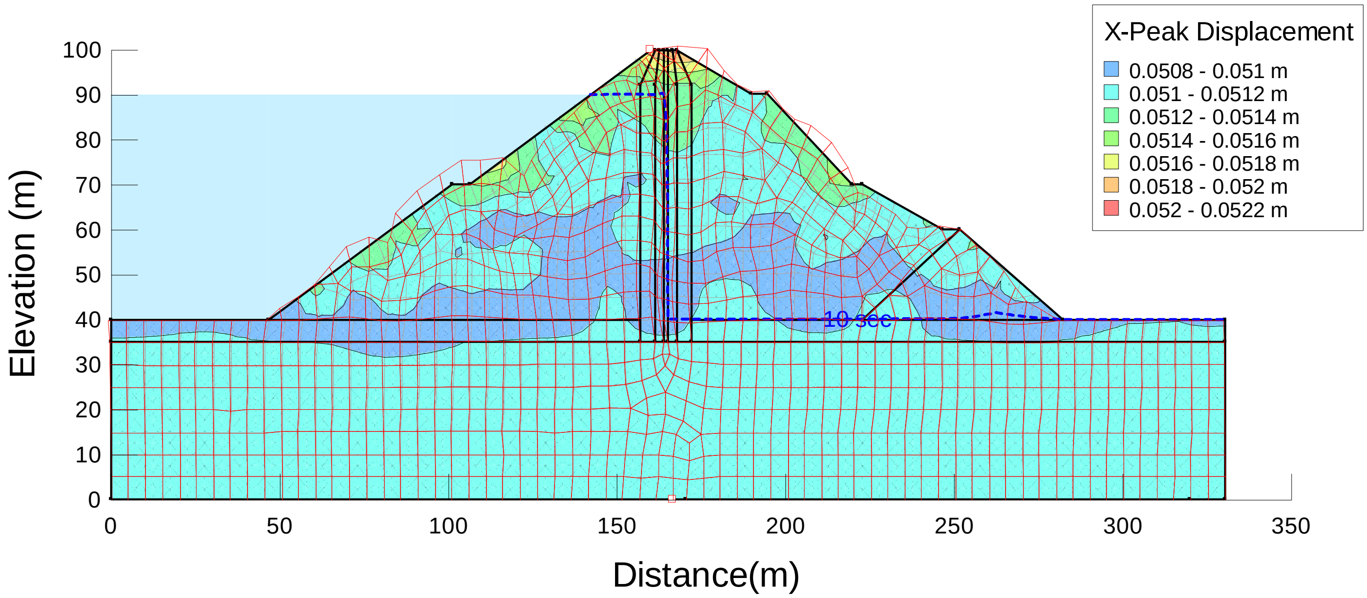

Figure 4. 26: Maximum Horizontal Displacement in the dam body and foundation dynamic state AC

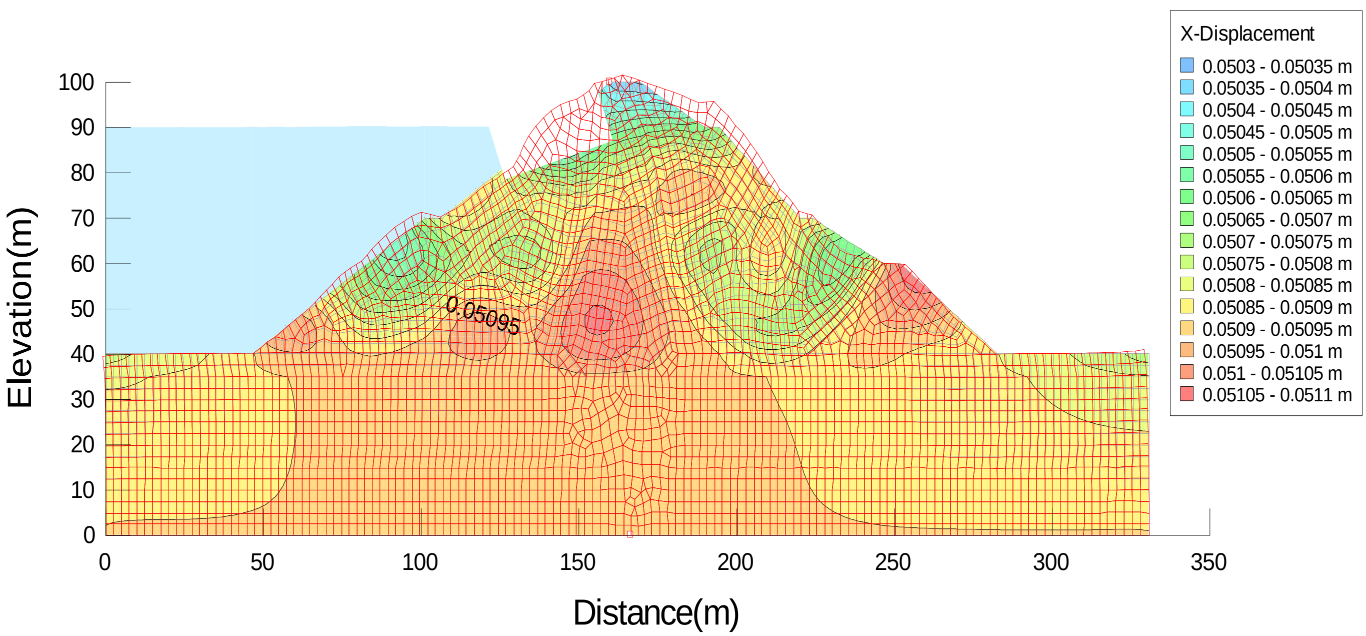

Figure 4. 27: Maximum Horizontal Displacement in the dam body and foundation dynamic state CC

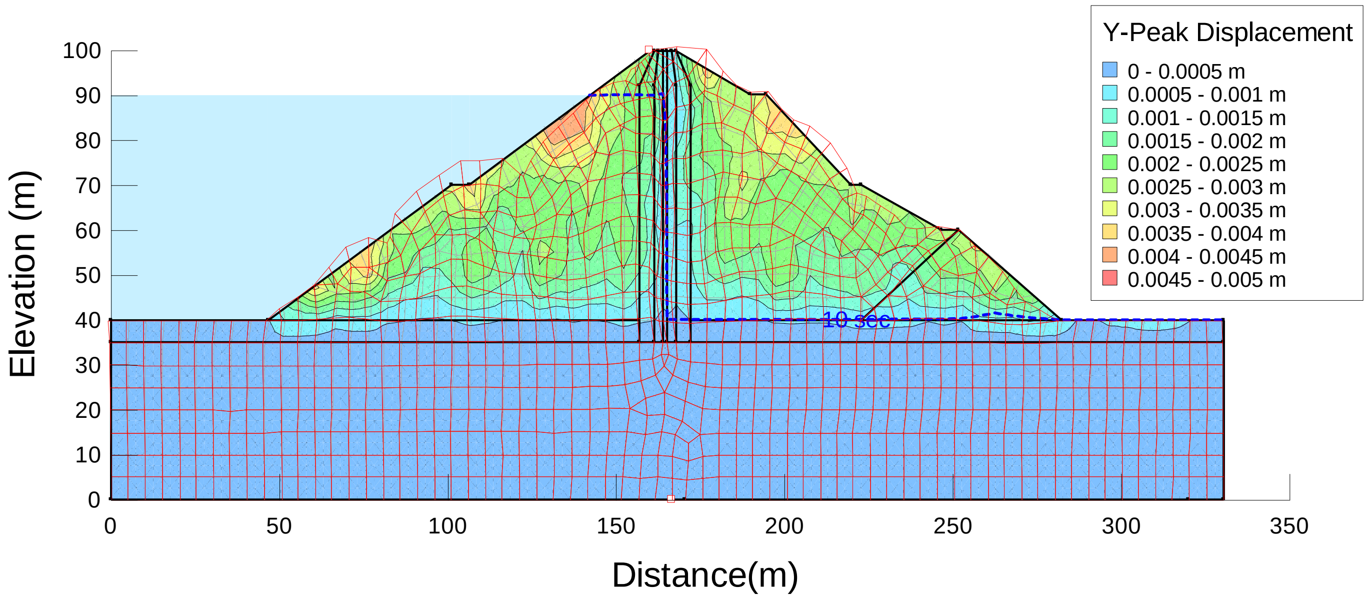

Figure 4. 28: Maximum Vertical Displacement in the dam body and foundation dynamic state AC

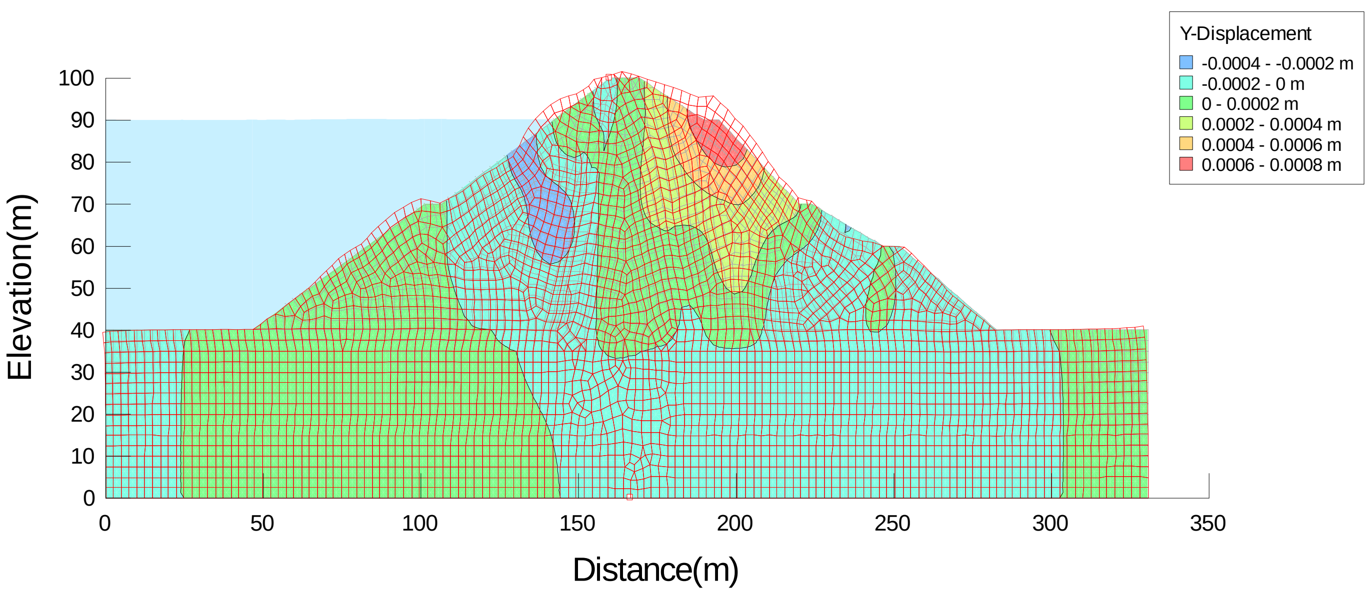

Figure 4. 29: Maximum Vertical Displacement in the dam body and foundation dynamic state

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This paper presented the evaluation of propose embankment dam for Dodota Irrigation Project to introduce the alternative embankment rock fill with asphaltic concrete core and rock fill with clay core. Asphaltic concrete core is important on an area of fault and seismic region due to the flexible nature, high induce deformation and resistant to leakage. The clay core misses a great amount of leakage than asphaltic concrete rock fill core. The proposed embankment dam for project is embankment dam with central asphaltic concrete core and clay core for the analysis. The analyses conclude the following results.

- From over all analysis of my thesis evaluation what understand point is all result gives us the possibility of the propose dam to be implement for the Dodota Irrigation Project .
- The quantity of seepage through dam body and dam foundation is found to be acceptable as compare to standards and the quantity of flux computed for asphaltic concrete core is almost insignificant, from the seepage analysis result the site is possible to constructing dam as alternative.
- From the slope stability analysis of upstream and downstream under the different loading condition the propose embankment dam rock fill asphaltic concrete core and rock fill clay core satisfied the minimum requirement of factor of safety recommended by USACE.
- The shear stress and vertical stress produced by the proposed embankment are located at the vesicular sound bed rock foundation; this stress level is much lower than the expected bearing capacity of the foundation rock and rock fill asphaltic concrete smaller effect on foundation than rock fill clay core.
- The static deformation analysis computed for the propose embankment dam shows the horizontal and vertical deformation that the dam may subject is with tolerable limit for each cases.
- The dynamic deformation also analysis computed for the propose embankment dam during the time of shaking the maximum vertical and horizontal deformations show below within allowable limit of the standard less than1% of the dam height.

5.2 Recommendations

From the analysis and study conducted for evaluation of proposed embankment dam for Dodota Irrigation Project the following recommendation point has been given for future work.

- From over all the thesis results of Dodota Irrigation Project earth fill with asphaltic concert core is used as alternative option for the possibility of constructing dam for the site instead of constructing weir for the project which is not store water during rainy time.
- For the dam to be constructed in the seismic region, use of asphalt concrete core is better than any other dam types to with stand foundation settlement and embankment deformation due to induced earth quake.
- What recommend for the client of the project (OIDA) possible to upgrade the project from weir irrigation to dam irrigation in order to increase the community benefit from the project.
- The client of the project (OIDA) and the Awash Basin Authority give attention for using the huge water resources of the basin during rainy season to use this during dry season and control the flood water damage on irrigation scheme under downstream of the basin; constricting rock fill with asphaltic concrete core is recommend for the basin including Keleta river.
- During the feasibility study and design stage of any dam project its recommended to consider different option of dam type and to give the specific solution of the site geological condition.
- Acknowledging all the designs which have been made so far in the country, the author of this paper recommends consideration of embankment dam with an asphaltic concrete core as a best type under such Earthquake area, shortage of impervious material from the site and for an area high rain fall through the year.

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7. APPENDIXES

7.1 Appendix A

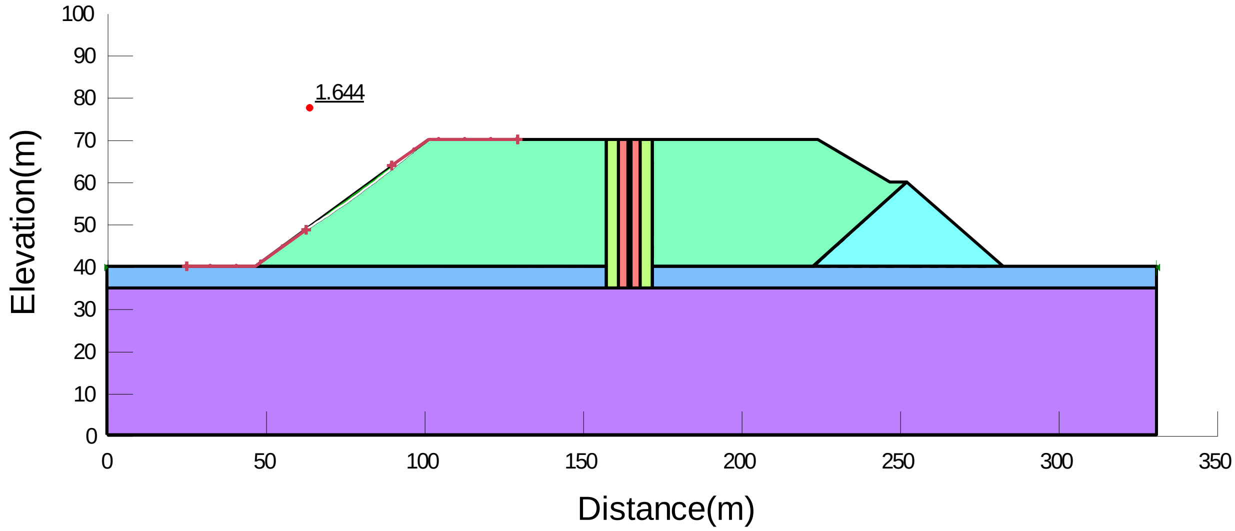

Figure A. 1: Slope Stability during construction without Earthquake AC (U/S)

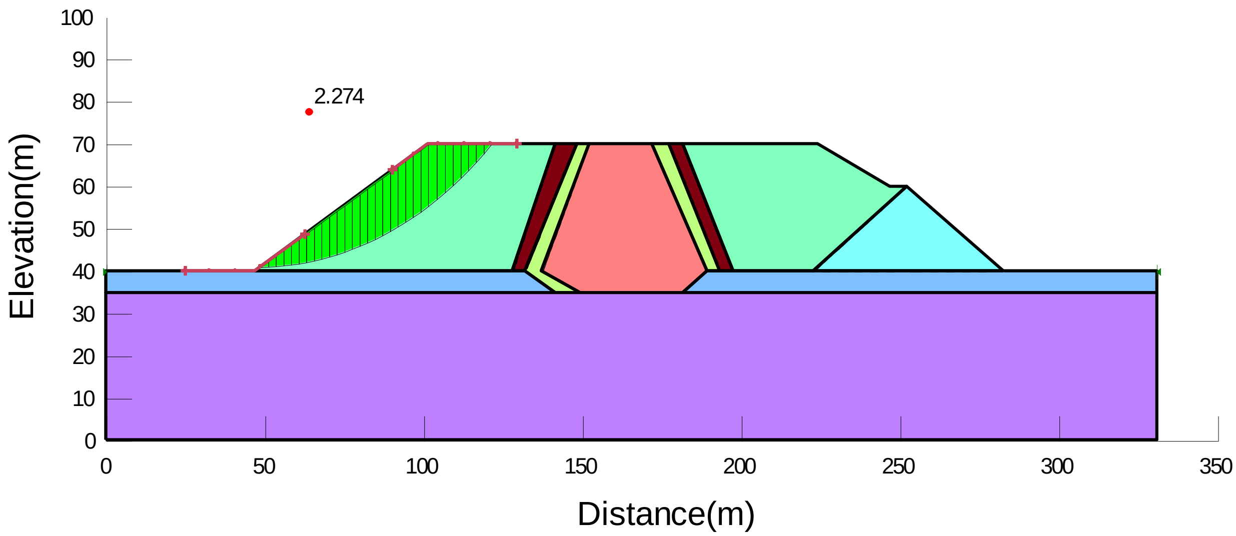

Figure A. 2: Slope Stability during construction without Earthquake CC (U/S)

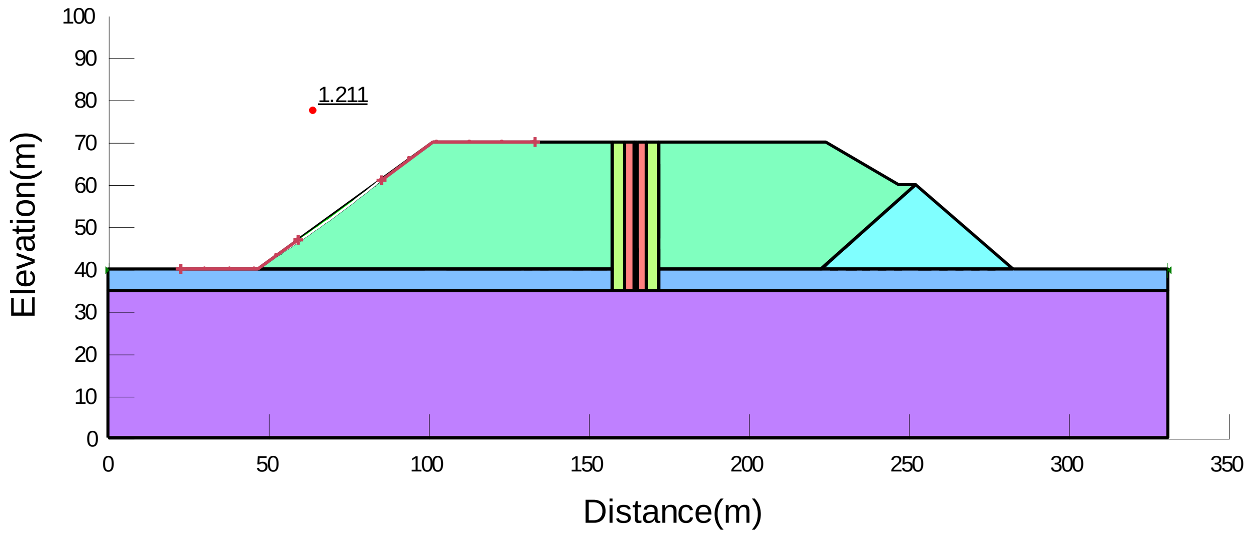

Figure A.3: Slope stability during construction with earth quake AC (U/S)

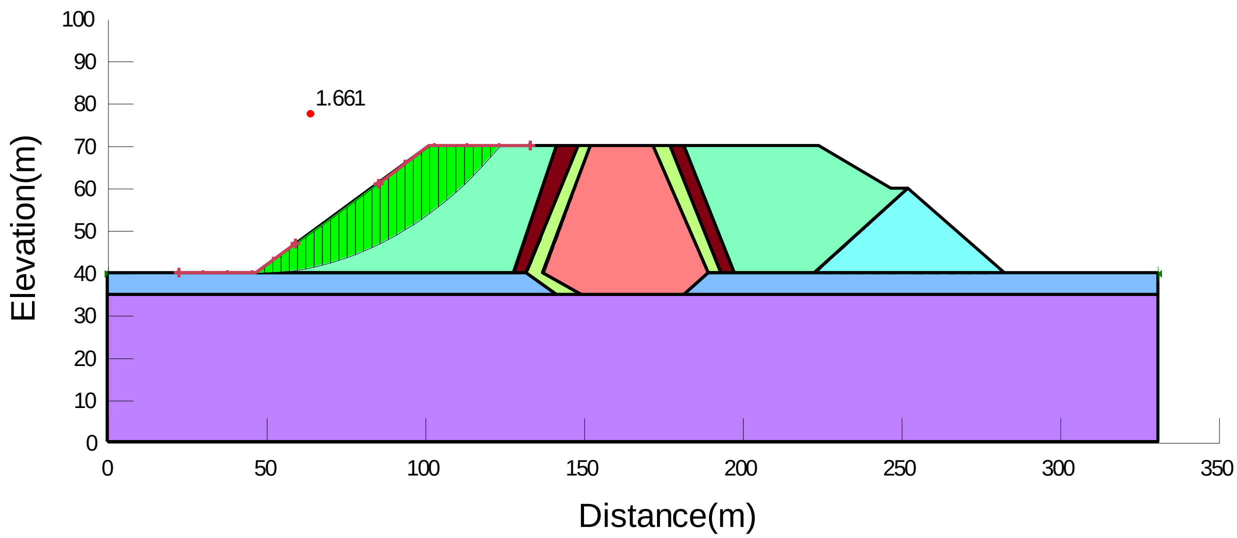

Figure A. 4: Slope stability during construction with earth quake AC (U/S)

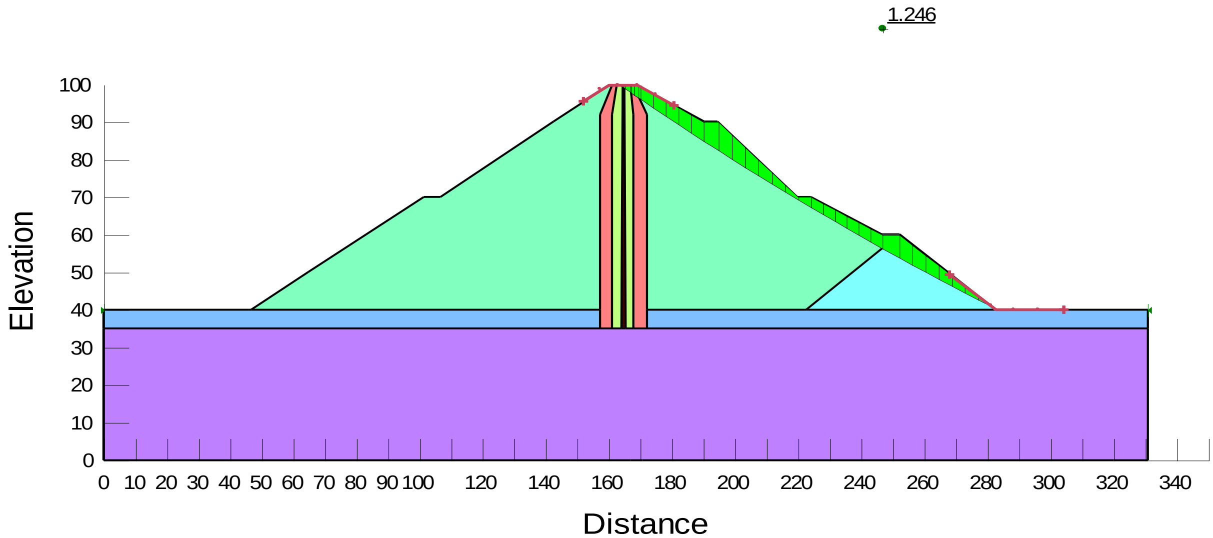

Figure A. 5: Slope Stability at end construction with earth quake AC (D/S)

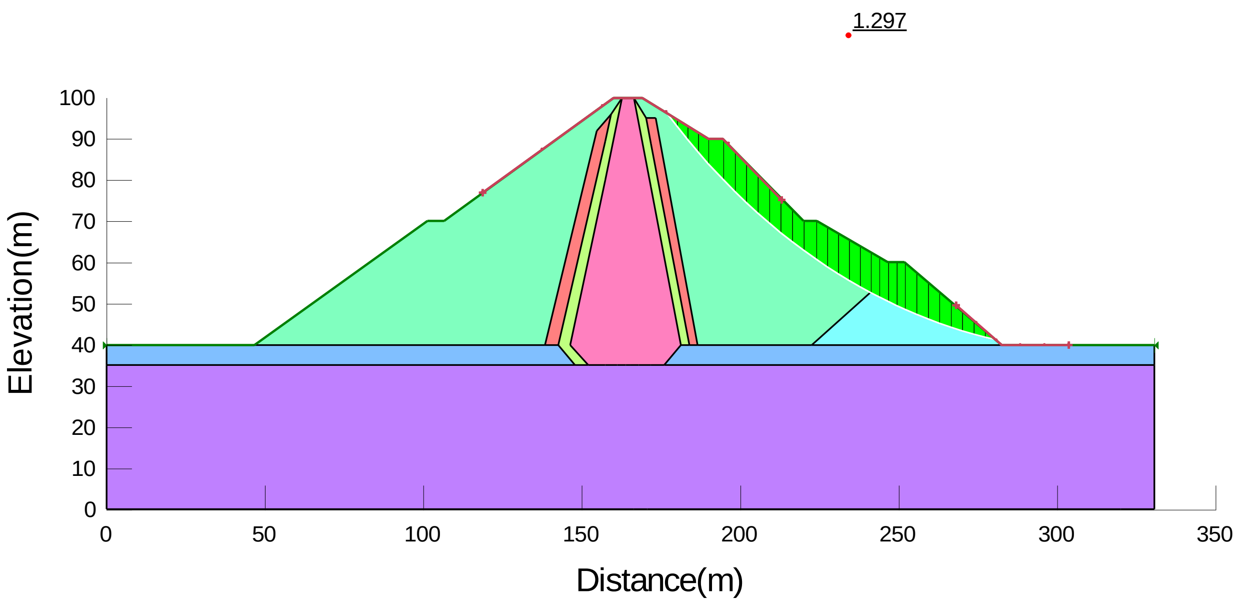

Figure A. 6: Slope Stability at end construction with earth quake CC (D/S)

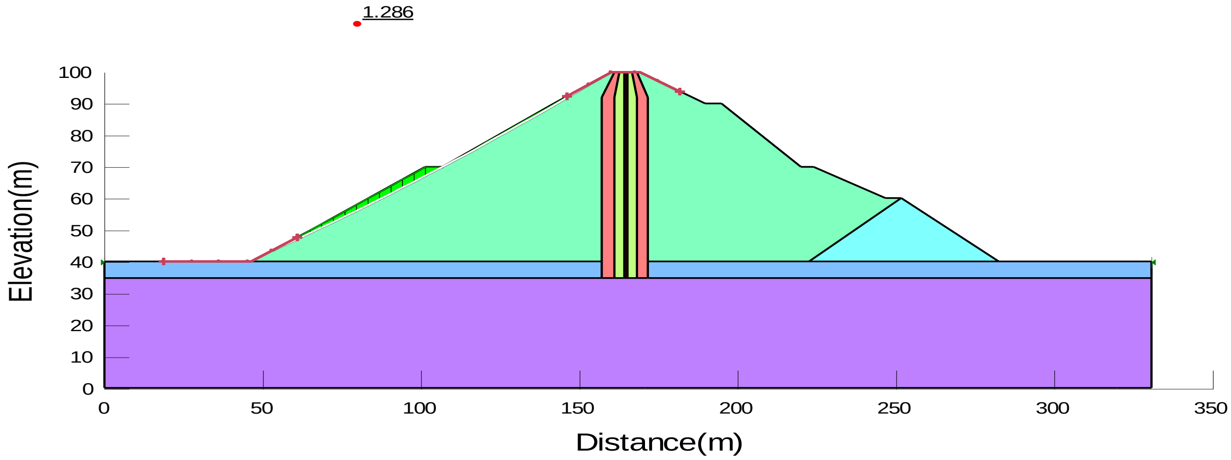

Figure A. 7: Slope Stability end construction with earth quake AC (U/S)

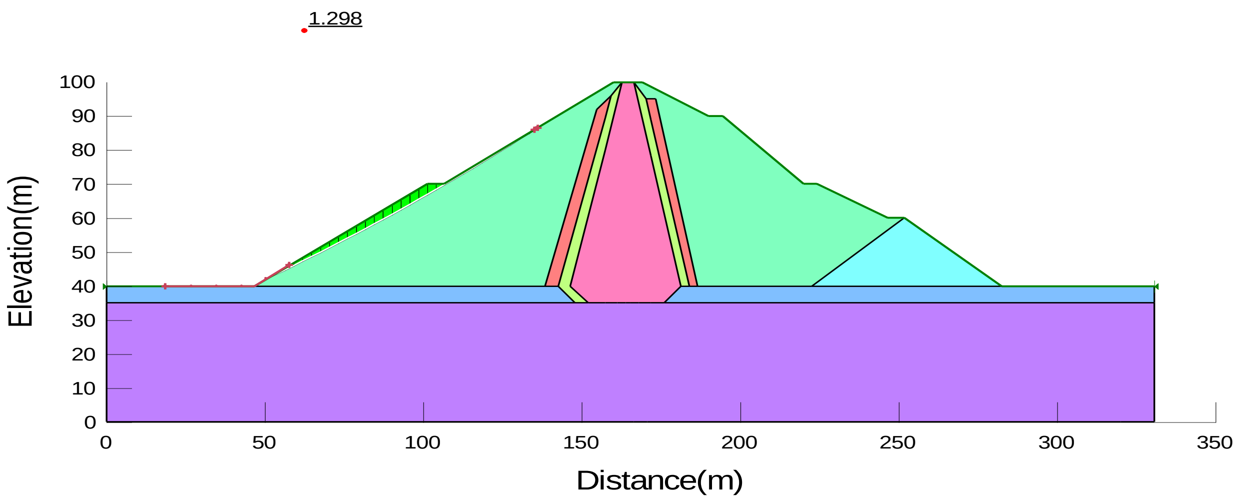

Figure A.8: Slope Stability end construction with earth quake CC (U/S)

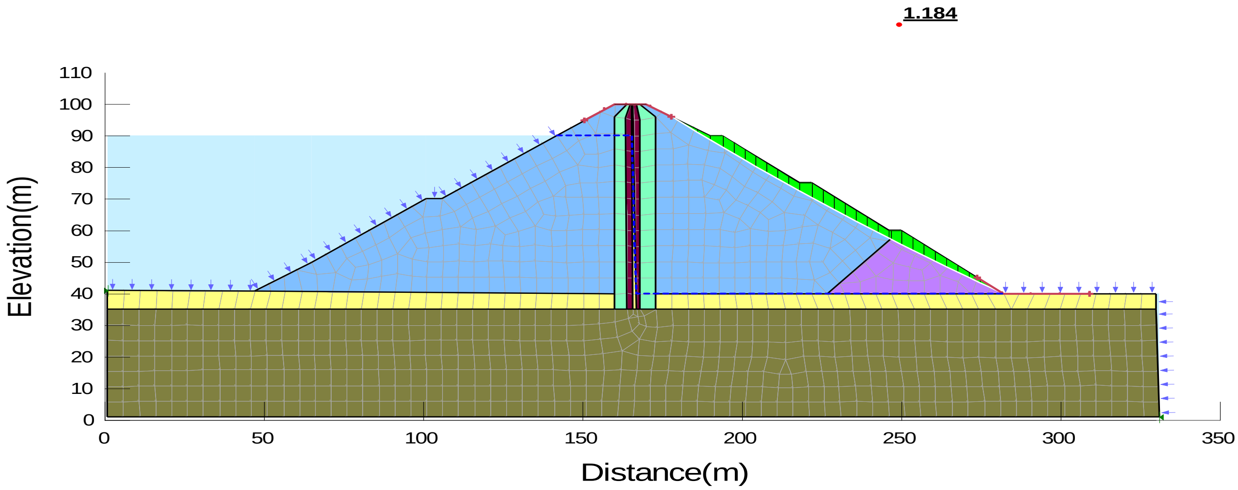

Figure A. 9: Factor of safety at Steady state with earthquake AC (D/S)

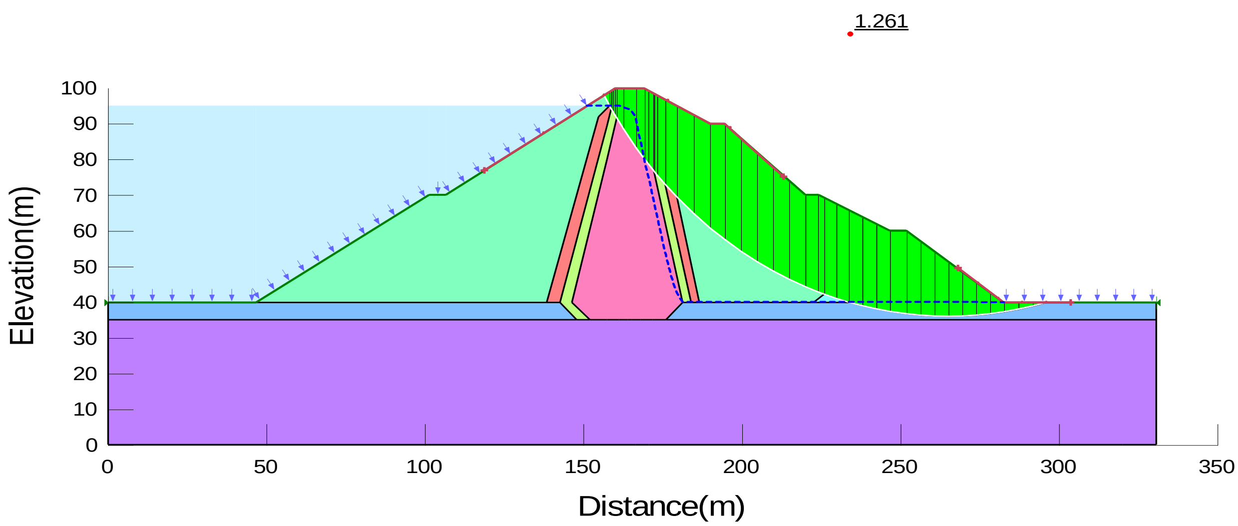

Figure A. 10: Factor of safety at Steady state with earthquake AC (D/S)

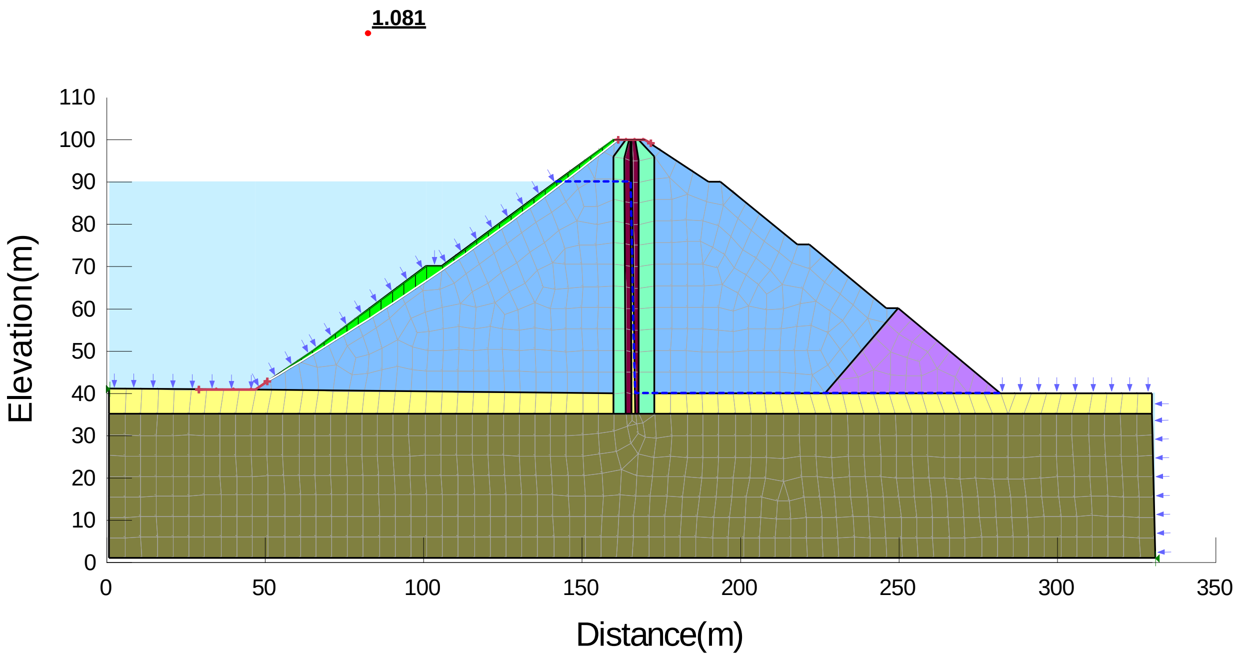

Figure A. 11: Factor of safety at Steady state with earthquake AC (U/S)

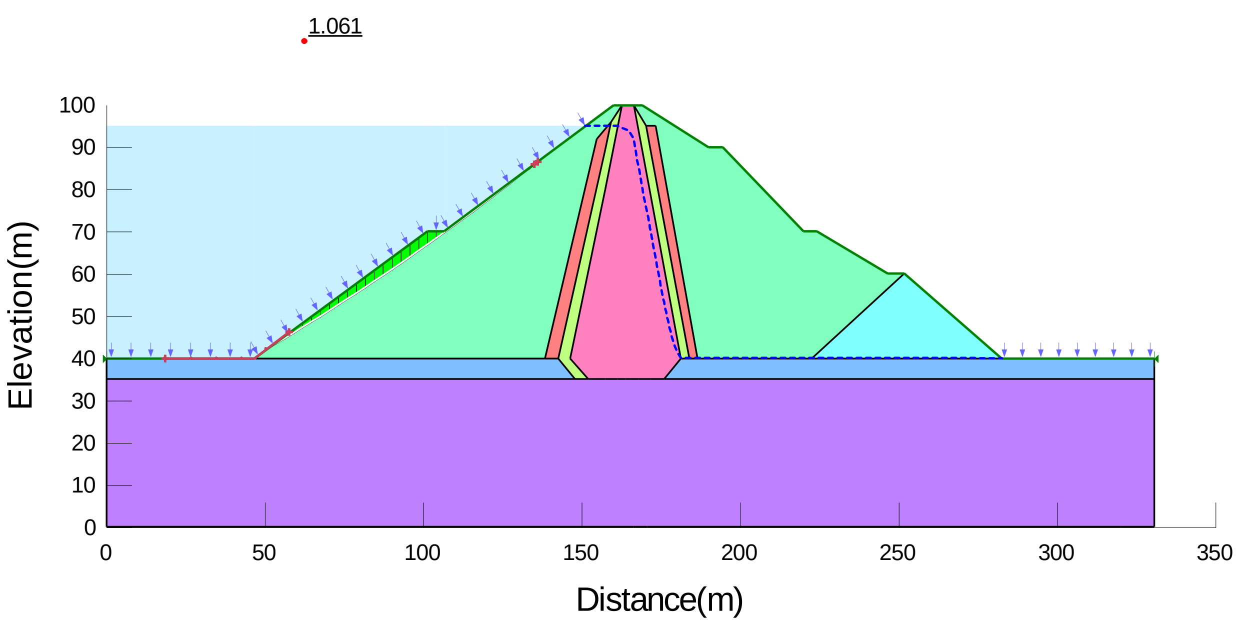

Figure A. 12: Factor of safety at Steady state with earthquake CC (U/S)

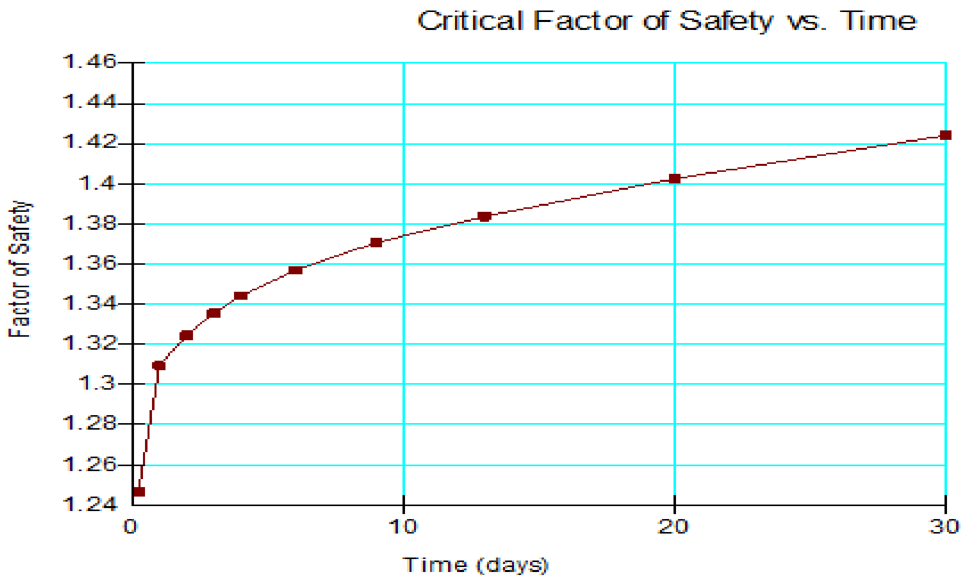

Figure A. 13 : Factor of safety verse time of draw down

7.2 Appendix B

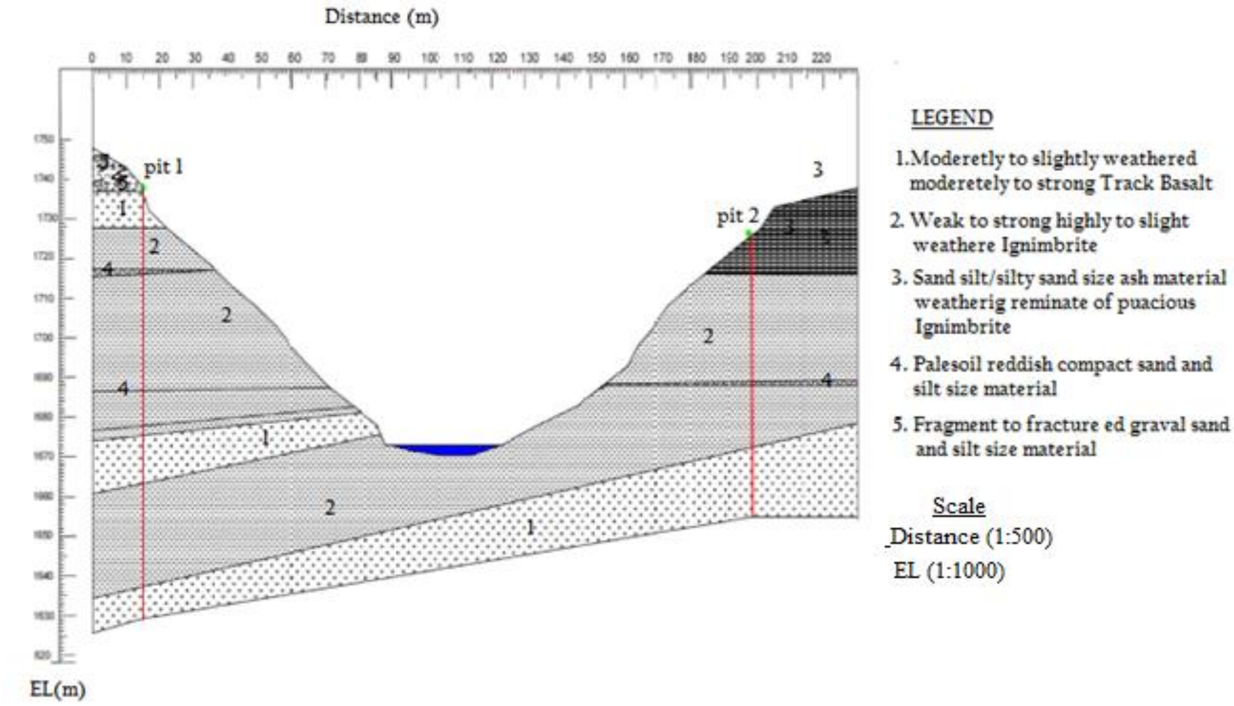

Figure B. 1: Geological section of the propose embankment dam for Dodota Irrigation

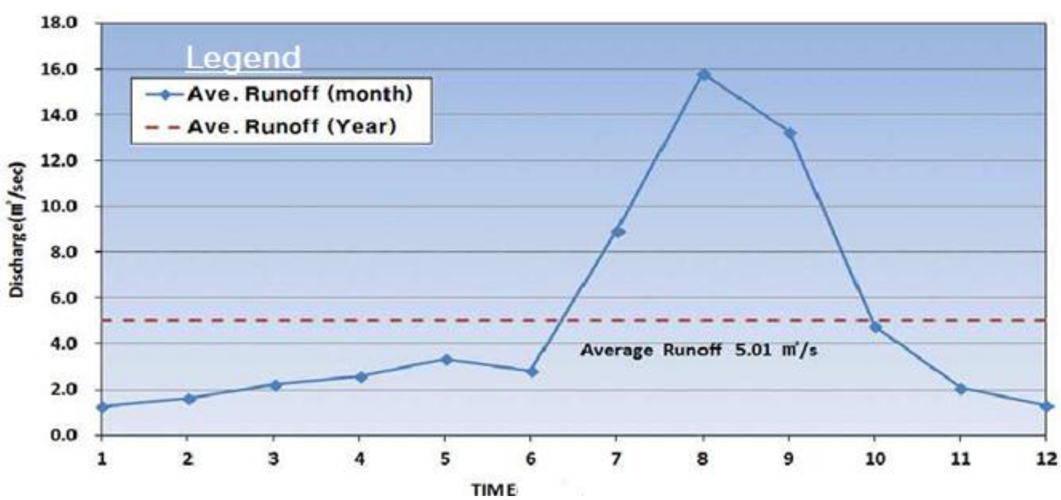

Figure B. 2: Mean runoff of observed data for Keleta River

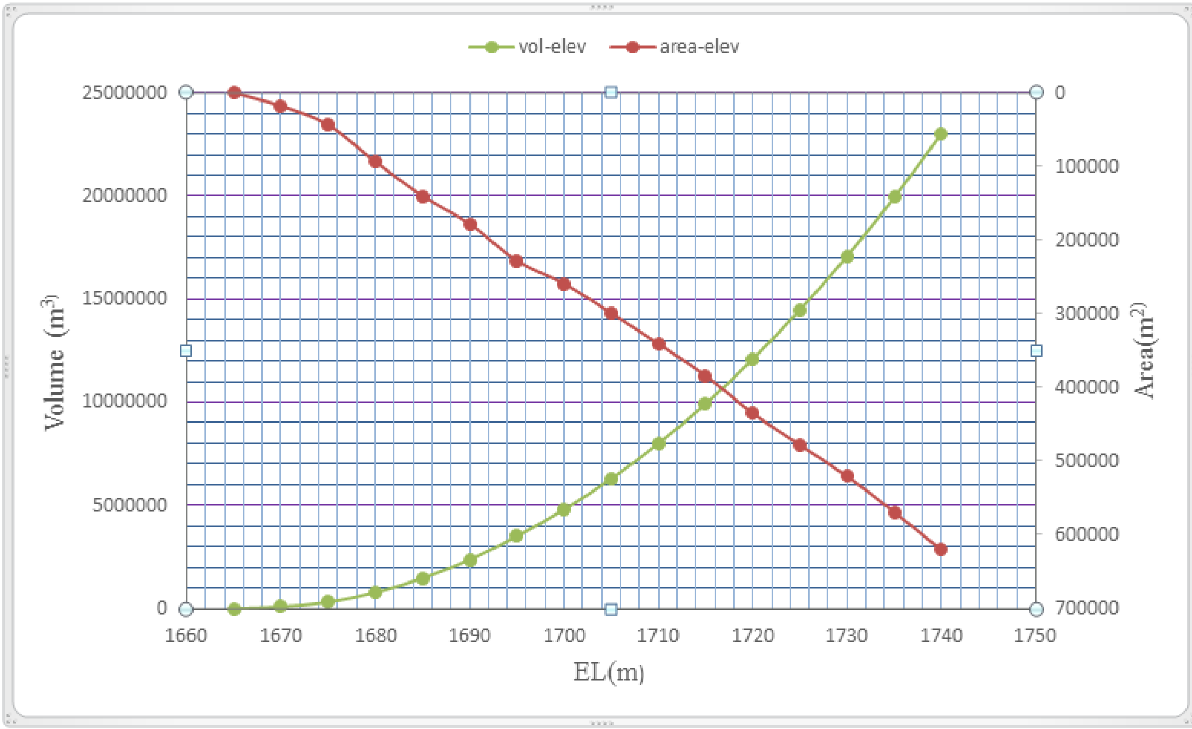

Figure B. 3: Elevation, Area and Volume curve

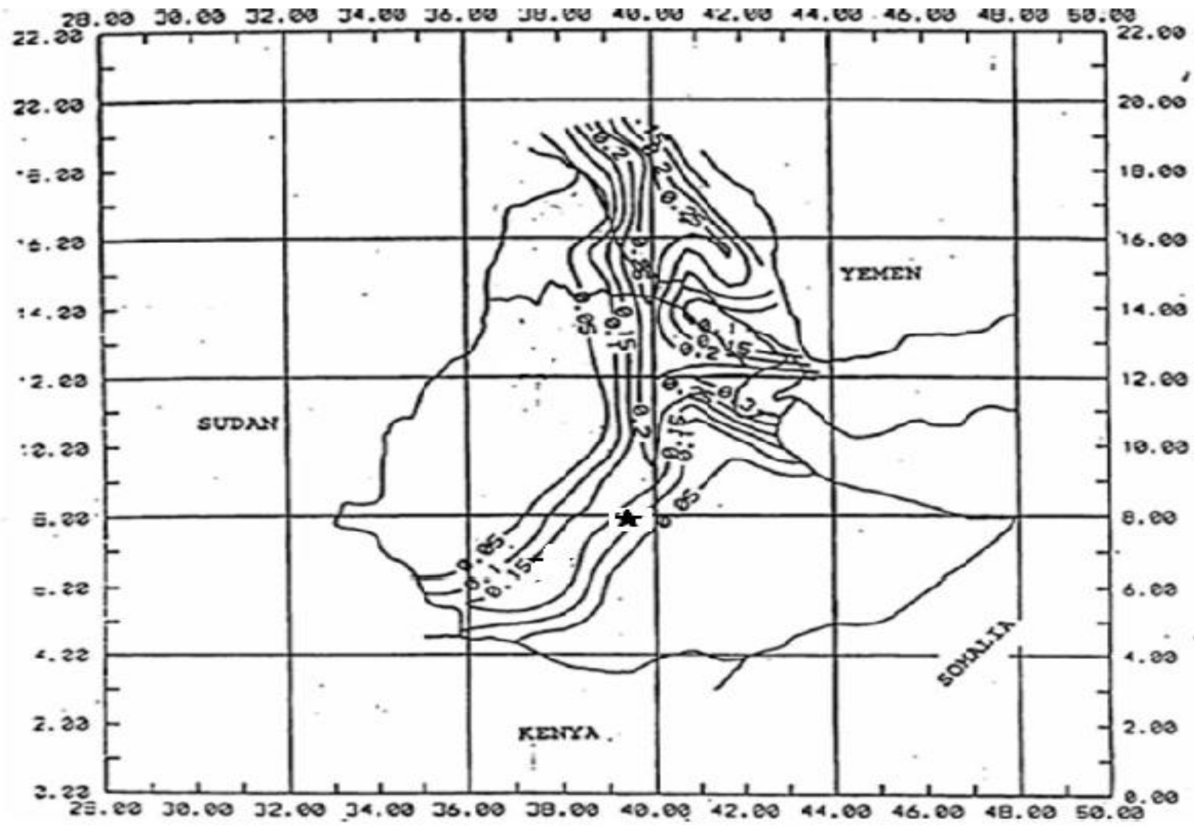

Figure B. 4: Seismic Hazard Map of Ethiopia and its Northern & Eastern Neighboring Countries. The black Star indicates the approximate location of propose Embankment dam site (Source WWDSE, 2016).

7.3 Appendix C

Table C. 1: Elevation, Area and volume of the reservoir with sediment accumulation

EL	Area(m2)	Av.area (m2)	Volume (m3)	Total Volume(m3)	Area (km2)	Volume (MCM)	Crest height	50year sediments
1665	780.596	0	0	0	0.0000	0.0000	1725	1680
1670	35124.45	17952.523	89762.615	89762.615	0.0180	0.0898	1725	1680
1675	52325.34	43724.895	218624.475	308387.09	0.0437	0.3084	1725	1680
1680	134987.456	93656.398	468281.99	776669.08	0.0937	0.7767	1725	1680
1685	145145.479	140066.468	700332.338	1477001.418	0.1401	1.4770	1725	1680
1690	210452.451	177798.965	888994.825	2365996.243	0.1778	2.3660	1725	1680
1695	245350.478	227901.465	1139507.32	3505503.565	0.2279	3.5055	1725	1680
1700	272458.478	258904.478	1294522.39	4800025.955	0.2589	4.8000	1725	1680
1705	325478.45	298968.464	1494842.32	6294868.275	0.2990	6.2949	1725	1680
1710	355489.547	340483.999	1702419.99	7997288.268	0.3405	7.9973	1725	1680
1715	412457.456	383973.502	1919867.51	9917155.775	0.3840	9.9172	1725	1680
1720	457458.456	434957.956	2174789.78	12091945.56	0.4350	12.0919	1725	1680
1725	498456.457	477957.457	2389787.28	14481732.84	0.4780	14.4817	1725	1680
1730	542483.472	520469.965	2602349.82	17084082.66	0.5205	17.0841	1725	1680
1735	595487.489	568985.481	2844927.4	19929010.06	0.5690	19.9290	1725	1680
1740	645489.458	620488.474	3102442.37	23031452.43	0.6205	23.0315	1725	1680