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GEFERSA-III EMBANKMENT DAM HEIGHTENING

By

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Addis Ababa, Ethiopia
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School of Civil and Environmental Engineering

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GEFERSA-III EMBANKMENT DAM HEIGHTENING

ABSTRACT

The sediment problem is a major problem that highly reduces the live storage capacity of the reservoir and its life span. The Gefersa-I/II dam is one of the major gravity dams in our country Ethiopia which faces a series of sediment deposition problems for a long period. To trap the incoming sediment and to reserve additional live storage, AAWSA tried to build an embankment dam called Gefersa-III the upstream of Gefersa-I/II dam. Even if this dam was built for sediment trap, the reservoir of this embankment was filled with the coming sediment and then the sediment starts flowing to the former dam (Gefersa-I/II gravity dam). Initially, different alternatives were seen as a possible solution to overcome the sediment problem, but in the end, AAWSA decided to heighten the existed embankment dam with a 7m additional height. Hence this case study mainly focused on determining the suitable and stable heightening type which is compatible with the existing dam condition. Among different heightening types, this case study was considered two heightening alternatives (homogeneous and concrete face rockfill heightening) to come with one best heightening type by compare and contrast the considered alternatives from a different point of view. In both heightening alternatives, this case study was tried to keep the operation of the reservoir in the existing embankment during the time of heightening. Due to this reason in both alternatives, this case study was decided to heighten the embankment starting from the crest of the existed embankment by performing local modification only. When the design for both heightening alternatives done, a berm was created on the crest of the existed embankment which was used as a working space for the heightening of the embankment. In both cases, the fill was decided to be done on the downstream face of the embankment to keep the reservoir operation during heightening. In case of homogeneous heightening, the heightening was designed to be raised with 1:3 side slopes at the upstream side and the downstream side slope was kept as 1:3. Whereas the concrete face rockfill heightening was raised with 1:1.3 and 1:1.75 side slope for upstream and downstream side respectively. Steady-state Seepage analysis was conducted for existing embankment and the considered heightening alternatives using seep/w of Geostudio software and as the result shows, the seep water to the body of the embankment was $1.3713*10^{-6}$, $3.05*10^{-7}$, $8.255*10^{-8} m^3/sec$ for existed, homogeneously heightened and vertical face rockfill heightened embankment respectively. The second section of the modeling part was all about slope stability analysis for different loading conditions of existing and considered heightening alternatives. As the slope stability analysis section shows, the existing and considered heightening alternatives have become safe against slope failure. Deformation analysis was conducted for the existing embankment and both heightening alternatives. In this section upstream and vertical settlement was determined for existing and heightening alternatives. Not only settlement, but stress distribution was also conducted for existing and heightening options. While the modeling section was accomplished, the heightening alternatives were compared and *contrasted concerning different points of view, and finally, the concrete face rockfill heightening type was decided as the best heightening type for the existing condition of the Gefersa-III embankment dam.*

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LIST OF ABBREVIATIONS

AAIT Addis Ababa institute of technology
AAWSA Addis Ababa water and sewerage authority
asl Above sea level
ACERD Asphalt Core Earth-Rock Fills Dam
BC Before Christ
CFRD Concrete Faced Rock Filled Dam
m^3Cubic meter
^{o}cdegree cent grade
D/S Downstream
FoS Factor of Safety
FEM Finite Element Method
FSL Full Supply Level
GSI Geological strength index
GpaGiga Pascal
g gravity
ha hectare
H Height of Dam
ICOLD International commission of large dams
Kmkilo meter
Kpakilo Pascal
MpaMega Pascal
No. Number
PI Plastic Index
USACE UnitedStates Army Crop of Engineers
USBR United states Bureau of Reclamation
USSD United States Society on Dams
U/S Upstream

1 INTRODUCTION

1.1Background

One of the major factors which affect the output of dams is excess sediment deposition through the reservoir. Sediment deposition has great influence on the live storage capacity of the reservoir. Even if sediment deposition has great influence on reservoir storage, the attention given for sediment prediction is less. Due to this reason significant difference may happen between actual sediment which enters in to the reservoir and under predicted sediment. If so; due to huge delta formation sediment deposition in the reservoir may become a major problem through the life span of the structure.

As history indicates, Addis Ababa was instituted as the capital city of Ethiopia in 1886, and now a day it has highly developed to being themost business area over the country ***(Tesfaye, 2017***). While Addis Ababa was established as capital city, its water source was the natural springs located around Entoto Mountains. As the number of populations and urbanization in the capital city increase, a water treatment plant was constructed around Entoto area to treat the water from the springs and Kechene River. Through time Gefersa dam was proposed to be constructed across Gefersa River to satisfy the potable water demand of the population in the capital city and its construction was completed in 1944. The Gefersa reservoir at dam sitecovers around 55.56 square kilo meter catchment area, and is located in Oromiya Regional State ***(Fasil, 2012)***.

After implementation, Gefersa Dam has faced serious problem due to excess sediment deposition in the reservoir. As previous studies and researches show, the original Gefersa dam was modified for a number of times to manage the excess sediment deposition in the reservoir. The dam was modified the first time in 1955 by increasing the dam height. This is followed later by another modification in 1966 and erection of a separate dam (Gefersa III) upstream of the main dam to trap sediment and reduce the sediment reaching the reservoir of the main dam. Despite these interventions, the sedimentation problem appears to persist. Due to this reason, the mitigation measure was continued for the third time by renovating the basic hydraulic works of the dam. Even after this mitigation measure, the problem continues to be a series issue requiring another round of rehabilitation / modification work.

1.2 Statement of the Problem

Dams areusually proposed and constructed to serve a number of generations with limited maintenance through their life time. Once a certain dam construction is ended, it may face series problem due to faulty design and/or construction and the mistake accounted in hydrology part. One of the major mistakes which might be accounted in the hydrology section is under estimation/prediction of the amount of average sediment which may enter in to the reservoir. Such mistake may lead the dam to a series sedimentation problem during its functioning which highly reduces the capacity and design period of the reservoir**.**A long term sediment deposition in the dam sitehighly affects the overall live storage capacity of the reservoir in turns reduction in the benefits acquired from the reserved water ***(Tesfaye, 2017***).

Reservoir in Gefersa-I/II gravity dam was created to satisfy the demand of potable water of populations in Addis Ababa and its surrounding. Despite of satisfying water demand of targeted population, this reservoir faces a series sedimentation problem and due to this reason the proposed live storage capacity of the reservoir has been reported to differ with its actual live storage capacity.To trap the incoming sediment and to reserve additional storage, a second dam Gefersa-III embankment dam was constructed upstream of the main dam (Gefersa-I/II gravity dam). However, Gefersa-III has now stopped operation due to silt problem, anddeposition has continued to the main Gefersa-I/II gravity dam. This problem highly affects the peoples of Addis Ababa and its surrounding who depend on the water reserved in the dam. At preliminary study stage, two possible alternatives were stated to reduce/or eliminate the effect of sediment deposition in Gefersa-III dam. The first one wasdredging the deposited sediment from the reservoir, and the other one was to extend the dam height. When the former alternative is investigated, it was found to be very costly. Accordingly, AAWSA selected the second alternative, i.e., extending the height of Gefersa-III embankment dam, as a better alternative to increase the live storage capacity of the reservoir and manage the problem of sedimentation.

1.3 Objectives of the Study

1.3.1 General Objective

The main objective of this study is to assess dam height extension options for the 19 m high Gefersa-III Embankment dam to extend the dam height by 7m including 2m freeboard to increase its storage capacity to effectively manage sediment deposition in the reservoir.

1.3.2 Specific Objectives

To attain the above general objective, the specific objective of the study is to:

- Asses various dam heightening types that are potentially suitable for Gefersa-III embankment dam and select the most favorable and feasible ones;
- Determine dam geometry and material for the dam extension that is compatible with the existing dam and its foundation economically and technically;
- Evaluate and check the safety and stability of the dam heightened under static condition.

2 LITERATURE REVIEW

2.1 General

Dams are one of the hydraulic structures known by causing flood attenuation and sediment trapping. Most probably, reservoir sedimentation occurs where ever the structure is with an approximate rate of about 0.8 % per year. According to Palmieri (2003) the live storage due to this sedimentation is predicted as 45km^3/year and around 13 billion US dollar economy may be demanded to replace this lose.Now a day the average life of the reservoir is around 30year, perhaps they are designed for a 50 year dead storage(sedimentation).Due to this reason the sedimentation problem comes around at the middle age of the reservoir and causes the reduction of the live storage capacity of the reservoir. As *Draft cold, 2009* tried to predict, the reservoir all over the world completely filled by silt after 200 to 300years from now. When a river channel transport a long and constant flood, the flow may change its original stream and the structure may become outflanked ***(Draft cold, 2009).***

Almost all rivers contain sediments. When a reservoir is created behind a barrier/dam due to impoundment, the sediment that is coming to the reservoir is deposited at the bed of the river. When sediment deposition is continued for a long period of time, the reservoir loss considerable volume of storage. Despite of the researches conducted through six successive decades, sedimentation is still become the most serious technical problem faced by the dam industry ***(Patrick, 1996).***

Poor estimation of sediment deposition in reservoirs is one of the major factor which leads to inaccuracies in investment and operation cost estimation for planned dams. Although it is often difficult and expensive to collect necessary sediment data from a river, it is, nevertheless, very important to establish a relatively accurate estimate of the sediment deposition rate in order to come up with a reasonable cost estimate for planned dams. It should be noted that sediment deposition show seasonal variation and this need to be considered in estimating sediment deposition rate. According to Mahmood (1986), dam planners should have historical sediment data for at least a half of the life of the project.However having such a huge recorded sediment data may be very difficult ***(Patrick, 1996)***.

In most case once project is proposed, it have not been stopped due to lack of recorded sediment data which define the historical trend of the river. However, the dam planner tries to predict the most accurate value of the sediment deposition through the life span of the reservoir, but the structure may become out of service before its life span due to different factors. A good example which can be mentioned here is the Chixoy hydro dam which becomes outflanked before its life span due to the occurrence of unexpected erosion with high rate ***(Patrick, 1996)***.

2.2 Dam Heightening

2.2.1 Dam HeighteningPerspectives

Bockhartsee dam is a dam located in Austria's mountainous region with a height of 50m. Through time, the demand of energy in the market was increase due to the increase in number of population and expansion of industrialization in the country. To generate the required additional energy, heightening of existing Bockhartsee dam was seen as best alternative solution rather than constructing new dam. Bockhartsee dam is a rock fill dam with central concrete membrane.

According to a proposal by Prof. Dr. W. Schober, the heightening of Bockhartsee Dam was carried out by providing a retaining wall on the crest of the existing dam (***Kainrath, 2018***). Initially, it was proposed to extend the dam height by raising the central core up to the new level Later on this idea was discarded by a better alternative solution which consists of providing a retaining wall on the crest of the existing dam and the overall fill being done on the downstream face of the dam (see Figure 1). This create safe environment for the upstream slope protection by providing berm on the upstream side of the dam. In addition to this the reservoir in the dam can be operated throughout the season without the interruption for construction of the heightening part.

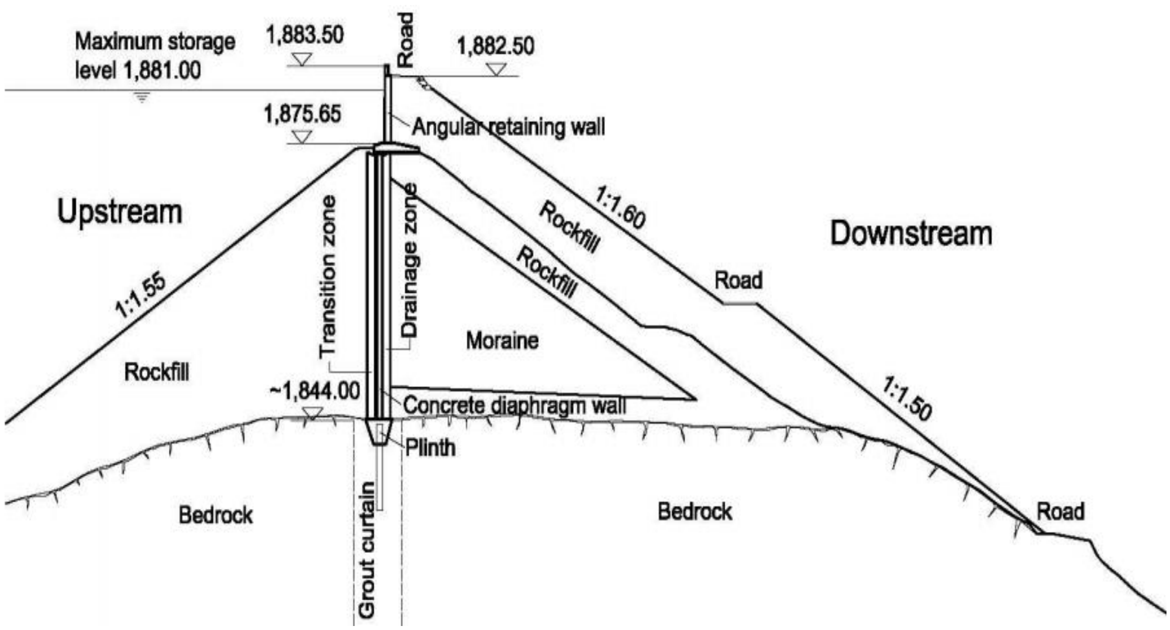

Figure 1: Bockhartsee Dam Heightening ***(Kainrath, 2011)***

Another example of embankment dam heightening is the case of Wayoh dam heightening. Wayoh dam is an earth fill dam with central clay core having a height of 19m above the river bed. The dam is found in Pennine, England. In 1959, it was proposed to increase the water supply capacity of the dam by modifying its existing geometric condition. The dam was proposed to be raised up to 26.6m above the river bed level by shifting the original position of outlet works. To create good compatibility between the new fill material and the existing one, the outer most layer of the existing dam was removed from the body of the dam. While the dam was heightened, the upstream and downstream slopes were made steep to minimize the overall volume of the heightened part of the dam. The heightening was commenced by using homogeneous material (earth) as the existed dam by lowering the reservoir level. On the time of heightening the level of reservoir was drawn down to prevent the interruption of reservoir full condition during construction.

Ladybower dam heightening also can be another good example of homogeneous dam heightening. This dam is a rock fill embankment dam found in Bamford England having a height of 43m above river bed level. After the dam was constructed, it faces a series of settlement problem for several years. To overcome this problem around four different actions were taken and the fifth remedial action also performed due to the increase in design flood from flood routing. To compromise the additional flood from flood routing, the heightening of existed dam was become best solution and then the dam was raised with 2.95m additional height. The heightening was performed as homogeneous as the existed dam by raising the core of existing dam to the crest of the new heightened dam.

The first stage of Breitenbach Dam (Germany) construction was completed in 1956 as earth-rock fill dam with central clay core. After a decade, the dam was proposed for homogeneous heightening with 12.5m additional height to increase the reservoir capacity of the embankment. As shown in figure 2 below, initially; the dam is proposed to be raised as homogeneous as the existing embankment dam. However the analysis conducted in 1975 was clearly demonstrate that if homogenous heightening type is adopted, the work of construction requires drawdown of the level of reservoir in the existed embankment, in which the only source of the water supply of the entire area may be reduced***(Ljubomir, 2005).***

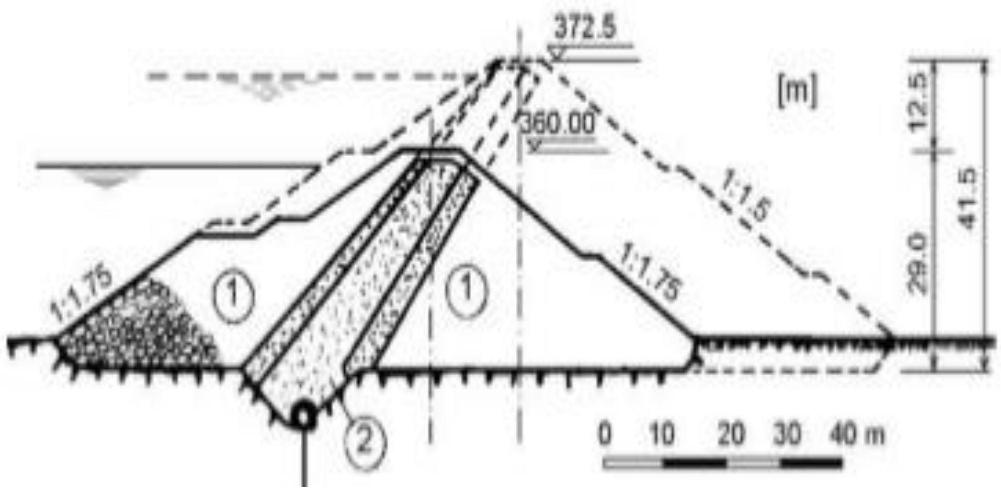

Figure 2: Breitenbach Dam (Germany), with the first design for heightening (after Weinhold &Haug, 1988). (1) Rock fill (2) clay core***(Ljubomir, 2005).***

Later on the proposed heightening way (homogeneous heightening) was changed to asphalt core rock fill heightening to overcome the problem which may come due to the drawdown of the level of existing reservoir. Finally the heightening was successfully done by providing asphalt concrete material which serve as water proof. Horizontal asphalt concrete was used as a joint in between the old and new impervious material. Since thebond between the horizontal asphalt concrete and the existing clay material must be water proof, it has to be designed carefully. To create a good bondage between the existing and new materials, asphalt concrete with high temperature over the existing material was placed(***Ljubomir, 2005)***.

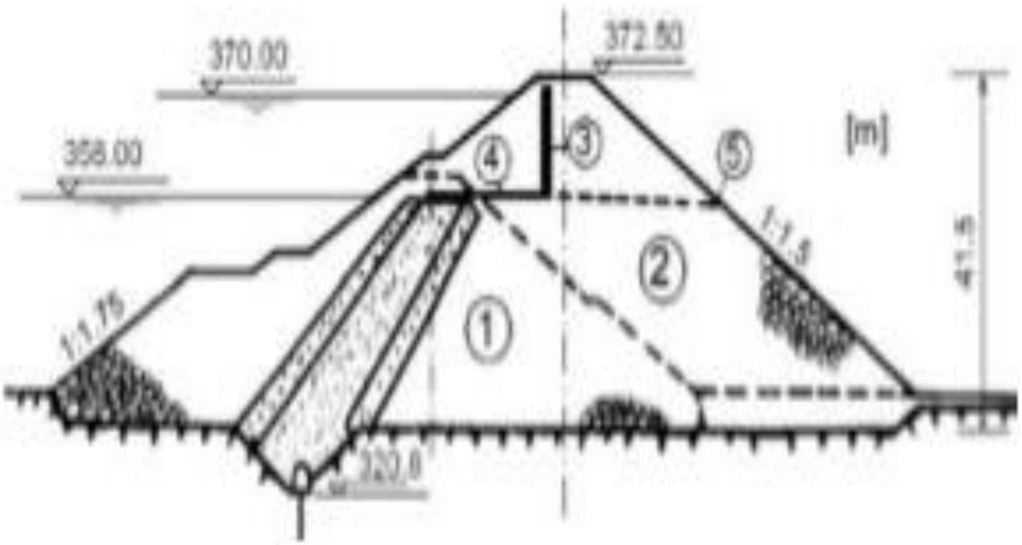

Figure 3: Breitenbach Dam with Heightening (after Weinhold & Haug. 1988). (1) First Stage, 1956; (2) Second Stage, 1977-1978; (3) Asphalt Concrete Diaphragm; (4) Horizontal Asphalt Concrete Layer: (5) Control Pipes**(*Ljubomir, 2005*).**

Pactola Dam (USA) is also another embankment dam in which heightening was performed due to the occurrence of heavy flood in 1972. The reconstruction of this dam was achieved by extending the height of the dam and widening of the existing spillway.Following this reconstruction, the spillway has been widened by 56m, whereas the dam has been heightened by 4.6m. The reconstruction of the dam had faced different obstruction which made the work more complex. However providing geomembrane as a waterproof material namely geomembrane for heightening sectionhas been considered as a best solution to raise the height of existing dam to the required level.On this dam heightening; adopting geomembrane heightening was create advantage of avoiding borrow pit for clay, reduce demand for the filter material which might be transported from considerable distance, in turns this play great role to accomplish the work with in limited period, and at the end considerable economy was saved. To select suitable geomembrane type, four laboratory tests were carried out and a geomembrane with 1mm thick was selected. The middle of the existing crest was the starting point of the geomembrane where it is embedded in a vertical trench of 90cm deep and 15cm wide filled with lean concrete. This vertical trench was provided in order to ensure a good joint is created between the existing embankment and the proposed geomembrane. At the banks the trench was provided with 60cm deep and 23cm wide in which the membrane was anchored by means of bolts. The membrane was laid up on well compacted fine graded rock fill with an inclination of 1:1(***Ljubomir, 2005)***.

Despite off the merits which may acquire from the geomembrane heightening, it may be very difficult to adopt such type of heightening if keeping the normal reservoir operation in the existing embankment is mandatory. This is because the upstream of the embankment may require significant modification and the reservoir level in the existing embankment may dropped to a number of drawdown stages. As the heightening ofPactola Dam (USA) in figure 3 below shows, the construction of the heightening section requires modification of the existing embankment within its 10.6m (1414.6-1404) vertical height.Therefore; this type of heightening may not suggested for the heightening works in which reservoir operation is required.

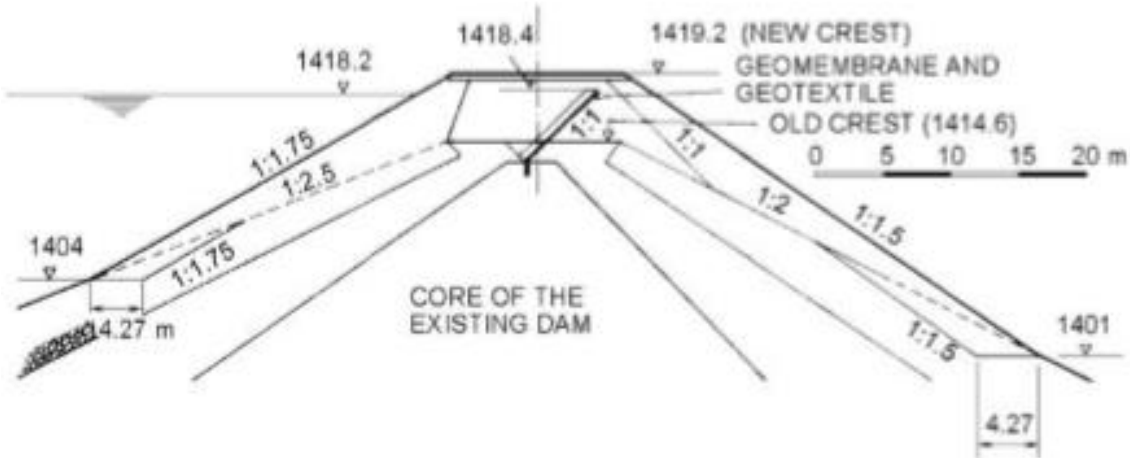

Figure 4: Detail of the Crest of the Pactola Dam (after Lippert & Hammer. 1989)***(Ljubomir, 2005).***

Concrete face rock fill heightening type is comparatively considered as simple heightening type.Especially; it is very suitable for stage construction. In this section the Pindari dam can be mentioned as a best example. This dam was seen for heightening to retain additional volume of water in the dam to satisfy the required irrigation water. To achieve this, the existing dam was raised from 44.5m to 85m.The heightening of the dam was achieved with an extension of the facing with the same inclination as in the original dam and expansion of the embankment towards the downstream sidewhereas another section of heightening was constructed by stages in a number of sequences, and the heightening over the old spillway was performed in the last stage.Since the level of rising is too high, this leads to alter the location of almost all appurtenant structures of existing dam. During the period of heightening, the facing section was designed separately to facilitate the rest part of heightening. The existing dam has 488m long with 1:1.3 side inclinations. The thickness of the concrete facing is determined according to the formula d =488+0.066H, where H is the vertical distance from the crest, in which the minimum and maximum facing thickness are 488mm and 760mm respectively. A grout curtain was also constructed throughout the upstream foundation of the facing. The spillway is located on unlined rock with 88.4m wide and 300m long just next to the dam. The new cross section of the heightened dam is presented in Figure 5a, while the section of the part over the old spillway (which is completely new) is presented in Figure 5b.

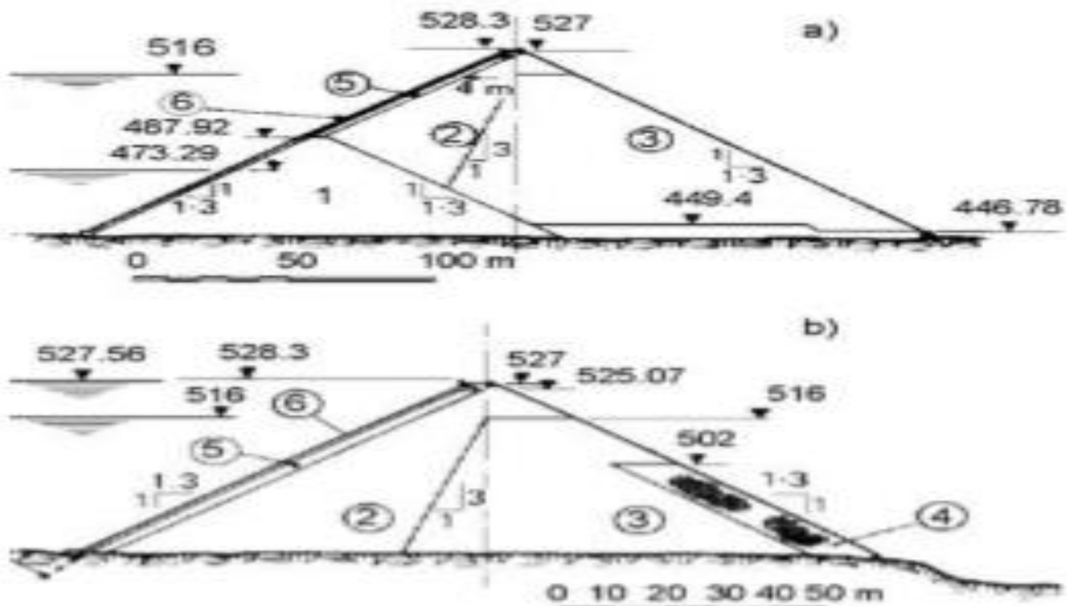

Figure 5: Cross-section of the heightened dam (a) and of the part above the old spillway (b) (after Lenehan, 1992)> (1) The old dam; (2) well compacted rock fill; (3) compacted rock fill; (4) selected compacted rock fill, strengthened with reinforcement; (5) compacted gritting material; (6) new facing***(Ljubomir, 2005).***

Ajaure dam is one of the embankment dams in Sweden with 46m height above the river bed.The construction works of this dam was accomplished in 1966 and start its commission in 1967. Through time heightening of the existing dam was proposed due to the occurrence of excessive horizontal displacement and the requirement of the dam to upgrade its flood retention capacity. After investigation and studies were carried out, it is decided to raise the water level in existing reservoir by 5m additional height. Various heightening alternatives were seen, but at the end a geomembrane was chosen to raise the moraine core of the existing embankment due to the shortage of suitable moraine material on the area and the appearance of left dam stability problem. Bentonite Enriched sand (BES) was placed as a joint between the moraine and the geomembrane core material. Originally the heightening was proposed to be done with side slope of 1V:1.8H and 1V: 1.5H near to crest. However the embankment was heightened by steeper side slope (1V:1.35Hto 1V: 1.40H) to raise the level of the crest by 1m additional height which becomes safety factor or freeboard for the unexpected settlement after construction. As the slope stability analysis section shows, the proposed heightening have low stability and berm was provided at the lower half of the downstream side of the embankment to stabilize the side. Even if this berm was placed, no decrement of deformation was seen. Due to this reason another upper berm was provided nearly total volume of 100,000m^3. Surprisingly, when the second berm was provided the horizontal downstream movement of the central part of the left dam increased dramatically and longitudinal fissures on the crest of the dam were observed. To come up with stable heightening type, a number of iterative height increments were performed, and finally the elevation was reached +447 in which the ongoing horizontal displacement will occur outside the moraine core in turn moraine core become free of transversal crack***(Thomas, 2004).***

The existing Zhushou Dam is a rock fill dam with clay core havinga height of 63.4m above river bed. This dam was seen for heightening due to inflation of the economy and the rapid development of energy of water. The heightening of this dam was carried out as concrete face rock fill embankment to provide water proof material on the upstream of the heightening section. After the completion of the heightening the embankment has a total of 98.1m above the river bed. Arranging deformation in old and new rock fill material under the involvement of gravity load, water load, and earthquake load was the major technical difficulty to be solved. To solve this technical problem, experiences of previous engineering technologies were considered and the necessary theoretical research was conducted to estimate the stress and deformation of the dam accurately.

2.2.2 Advantage of Dam Heightening

The first major advantage of dam heightening is that, the volume of storage due to heightening is significant with respect to the level of heightening. For the same volume of storage, Dam heightening is very cost effective than constructing new dam. Since all the components of dam like spillway and outlet work is in place on the existing dam, the only cost is become cost for heightening. This clearly shows how much dam heightening is very cost effective than constructing new dam to store the same volume of water.

2.2.3 Earth Dam Heightening Approaches/Techniques

Once construction of a dam is completed, there may arise a need for storage capacity increment due to different reasons like increased demand for hydro power, water supply, irrigation water etc. To increase the storage capacity of the reservoir behind the dam, it is a must to heighten the height of existing dam. Performing extension on embankment dam is not as such simple task because it may interrupt the functionality of dam appurtenant structures. Even if it has its own risk, to some extent dam heightening may become simple, if the heightening is proposed during construction of the existing dam. This is due to the reality that if the heightening is proposed during construction of the existing dam, it may create safe environment to heighten the dam without altering the position of dam's appurtenant structures. In general almost all heightening affect the position of appurtenant structures and also in case of heightening of embankment dams compacting the joint of the existing impervious material with the joint of new one become another major issue. During heightening; if keeping reservoir full condition is mandatory, the work execution may become more complex and difficult due to the occurrence of high moisture. The way of heightening of dam highly depends up on the height which is proposed to add on the existing dam. This means if the height which has to be increased is significant; it may require modification of the whole cross section of the dam, whereas if the heightening is as such small, the heightening can be done by modifying the local cross section only (***Ljubomir, 2005).***

While the probability of heightening is compared, it is possible to say that heightening of embankment may be proposed more rarelythan heightening of concrete barriers due to the reason that heightening has greater influence on appurtenant structure in case of embankment dam than concrete dams and also providing suitable joint in between the existed and new heightened impervious part become another major issue in case of embankments(***Ljubomir, 2005).***

Ljubomir, (2005) provide different approaches which has to be applied to solve the problematic situations which create unsafe working environment during embankment dam heightening. This sub-section would try to list and describe the possible embankment heightening approaches with their basic advantage and disadvantage.

2.2.3.1 Homogeneous heightening type

In homogeneous heightening, the heightening of the dam is performed using the same material as that of the existing dam. This heightening type for embankment dams is generally considered to be simple and the construction is comparatively straight forward.

Advantage

Since the extension/heightening is conducted using the same material as that of the existing one, it requires no transition material between the existing and the new material. Rather it may require removal of the outer most layers to remove the decomposed material from the existing embankment. This condition may consider as an opportunity to avoid the complexity of performing joint compatibility between the existing and new material. And also due to the material homogeneity, the probability of incidental failure may be comparatively less.

Disadvantage

Considering that while the embankment heightening is performed as homogeneous as the existing one, the embankment may require modification of the whole section and this may lead to drawdown of the reservoir level to the bed or to the required level. This situation may expose the users to danger; if their day to day activities rely on the operation of the reservoir. Purely homogeneous section of earth fill embankment may lead to seepage problem; hence the section of the embankment has to be broad enough to prevent the occurrence of such event. Due to this reason the volume of fill material become too huge and then the cost of fill may become expensive**.**

In case of earth fill dam the level of control of pore water pressure either in the dam body or its foundation is less. Due to this reason the factor of safety against slope stability may be maintained at low level.

In case of earthen dam, it is very difficult to predict and simulate the value of horizontal to vertical ratio of permeability of soil material in the laboratory. As soil material is very sensitive for permeability, the chance of occurrence of piping at the downstream face of the embankment may have a greater percentage of probability. Obviously, if piping occurs, erosion of dam body or foundation becomes another consequence. In general if homogeneous heightening is decided, the material has to be tough enough to withstand the coming seepage and the modeling part has to check with greater care to ensure that the seep water is insignificant and suitable drainage has to be provided for the safe passage of the phreatic line through the filter material***(Robin, 2015)***.

Another major demerit of homogeneous heightening is that, if the core material in the embankment being cracked due to settlement or earthquake effect, the seep water may flow freely through the cracked feature. Hence good design of embankment with suitable drainage may become better solution to prevent the occurrence of such problems.

2.2.3.2 Concrete face rock fill heightening

Concrete face rock fill dam is a type of embankment dam in which dam body is filled by rocks or gravel material with layer. In addition, there is concrete face slab on the upstream side of the embankment which serves as anti-seepage.

This type of heightening is mainly adopted in case of stage construction/execution. While heightening has carried out as concrete face rock fill, the facing is designed conservatively, which is very important to facilitate subsequent construction. The facing always require foundation slab with a width of 7m maximum at the maximum section and 3m wide at the banks. The thickness of the slab facing can be determined by using the formula $d = 488 + 0.066H$, where H is the vertical distance from the crest, in which d is limited as 488mm minimum and 760mm maximum. Most of the time the extension of the facing is achieved by adopting the same side slope as the existing face (***Ljubomir, 2005).***

Advantage

The major advantage of concrete face rock fill heightening is that, it creates safe condition to heighten the embankment by doing only local modification of the existing embankment rather than modifying the whole cross section.It is possible to say that this type of heightening can be done by modifying only the downstream side of the embankment including the crest. Such safe condition play great role, if temporary stoppage of reservoir operation become critical issue.As this type of embankment adopt steep side slope, the total volume of fill material is smaller than earth fill due to the reduction in the width of the embankment and this leads to minimizing the cost of material fill.Since this type of construction require very short term schedule, it can be conducted even in rainy/wet season. Since concrete face rock fill embankment require less width than earth and rock fill embankments, the cost of foundation treatment become comparatively less ***(Robin, 2015)***.

Disadvantage

This type of embankment performs well, but a continuous contact between the impound water in the reservoir and the concrete faces may cause series leakage problem due to the deformation of the concrete face ofthe embankment ***(Robin, 2015)***.

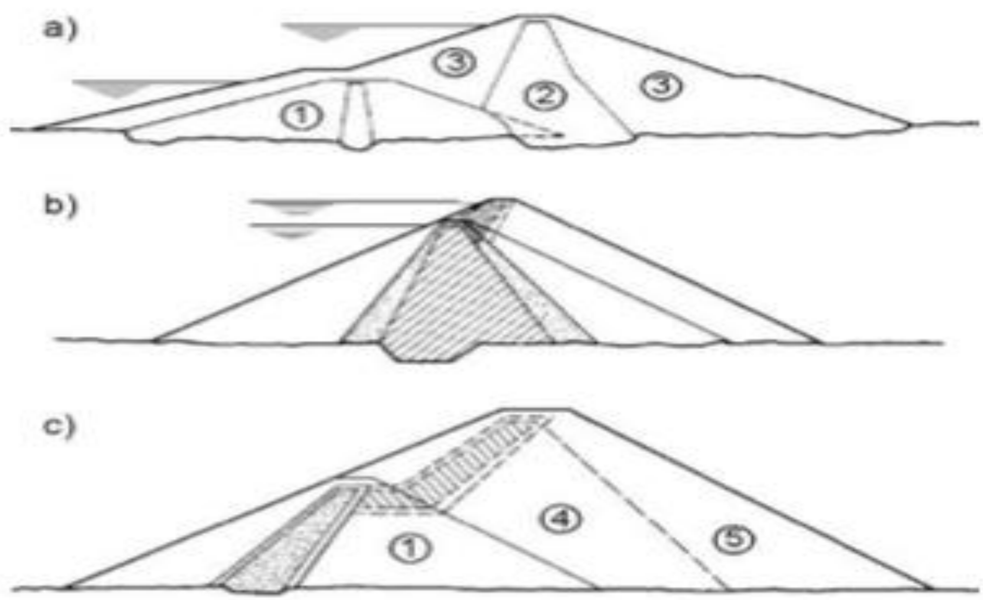

Figure6: Alternative Embankment Dam Heightening Approaches ***(Ljubomir, 2005)***

2.2.3.3 Rock fills with Asphalt concrete diaphragm

This type of construction is carried out by providing waterproof asphalt core to control seepage and impound water. In case of heightening of earth-rock fill embankment, it is possible to replace the impervious earthen material by artificial material. Due to this reason, in practical case, asphalt concrete can be used as replacement of earth fill core material for dam heightening (***Ljubomir, 2005).***

A good example of rock fill embankment dam heightening with asphalt core diaphragm is the Breitenbach Dam in Germany (Ljubomir, 2005).The Dam is a rock fill dam with inclined clay core. It is 29 m high and was constructed in 1956.At the time, it was planned for the dam to be heightened later by 12.5m to increase the dam's storage capacity. Initially the embankment was proposed to be heightened as homogeneous as the existing embankment*(see Figure 3)*.The approach was later changed to rock fill with asphalt core diaphragm including providing horizontal asphalt concrete blanket*(see Figure 4)* to maintain the functionality of the reservoir in the existing embankment which is the only source of supply water for the population in the surrounding. This kind of heightening requires great attention in design and construction of the joints between the existing and new fill material(***Ljubomir, 2005).***

Advantage

This type of heightening is suitablewhen the reservoir operation is maintained during construction. This is because heightening is started on the crest of the existing embankment. Since almost all the fill is done on the downstream side of the embankment it avoids the interruption of reservoir operation during construction. Moreover, since the core is a diaphragm type, it require few of concrete, hence, it saves economy. Since the diaphragm part is waterproof core, it may be tough enough to prevent seepage of water to the body of the dam.

Disadvantage

One of the major disadvantages of this heightening type is that the bondagebetween the existing earth fill core and the new horizontal asphalt concrete diaphragm is relatively complex and challenging to construct.

2.3 Modeling of Dam Heightening

2.3.1 Definition of Modeling

A model can be defined as the simplified form of the naturally existed events which consider only the most important features of the events whereas it ignore the features which seems of insignificant. This modeling is very important because it yields numerical value which can replace direct measuring value of natural entity.Since modeling is the simplified form of the natural system, it becomes essential in water resource management***(Fasil,2012).***

2.4 Embankment Dam Analysis

2.4.1 General

In embankment dam design and analysis, the main objective is to design and construct safe and stable embankment with naturally available material with minimum cost. In the early years before the introduction of soil mechanics, applying mathematical analysis or formulas to determine the adequate section might be best way in case of concrete dam but not for earthen dam. As a naturally occurring material, soil may occur in different form. Its particle size gradations, mineral composition and particle shape leads to complexity of using mathematical analysis in case of determining embankment (earthen dam) section. However, through time the field of soil mechanics become more developed, and significant progresses has been made in the development of investigation and analysis methods, the way of testing construction material.This will allow better design decision through optimization of the available design alternatives in such a way that the design with relatively low cost becomes the best design section of embankment.

2.4.2 Mode of Failure of Embankment Dam

2.4.2.1 Seepage Failure (Hydraulic Fracture)

Obviously soil material in embankments has the characteristic of allowing the passage of water from one section to another section. However, it is possible to minimize the amount of seep water by providing impervious material as core. Due to this, limited amount of seepage may occur through the body of an embankment which usually has little or no significant effect on the stability of the dam. However, excessive seepage for a long period of time from upstream to downstream may cause for the formation of piping and sloughing at the downstream of the dam which at the end may lead to permanent failure. A number of earthen embankments have failed due to the formation of piping. To explain the care that has to be given for this problem ***Garge, 2005*** said that “Ensuring the embankment safety against eternal erosion, piping and excessive pore pressure is the basic task while designing embankment dam”.

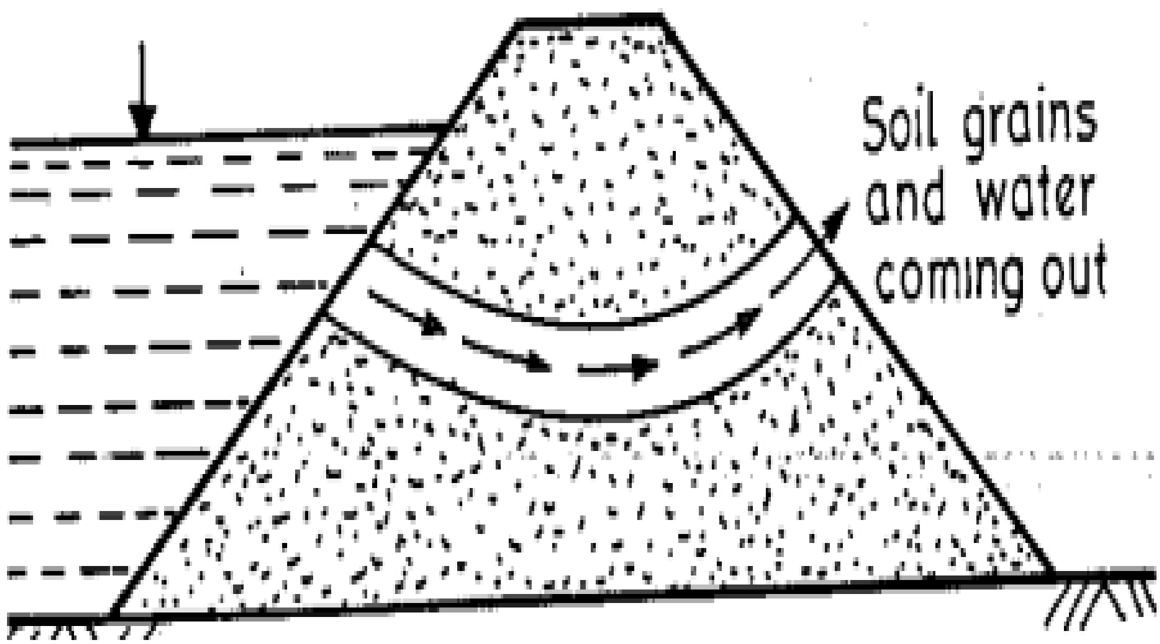

Figure 7: Piping Through the Dam Body ***(Garge, 2005)***

2.4.2.2 Structural failure

About one- fourth of dam failure is caused due to shear or sliding which is generally called structural failure ***(Garge, 2005)***. The sliding in hydraulic structure may be due to a problem in the foundation or embankment slopes. Embankment's stability always depends on the characteristics of the material used to fill the dam section and the level of treatment done for the foundation.

a. Sliding through foundation: This problem may occur when the bearing capacity of the foundation is less than the overall weight of the structure. If the foundation of the dam has weak bearing capacity the entire body of the dam may slide over the foundation ***(Garge, 2005)***.

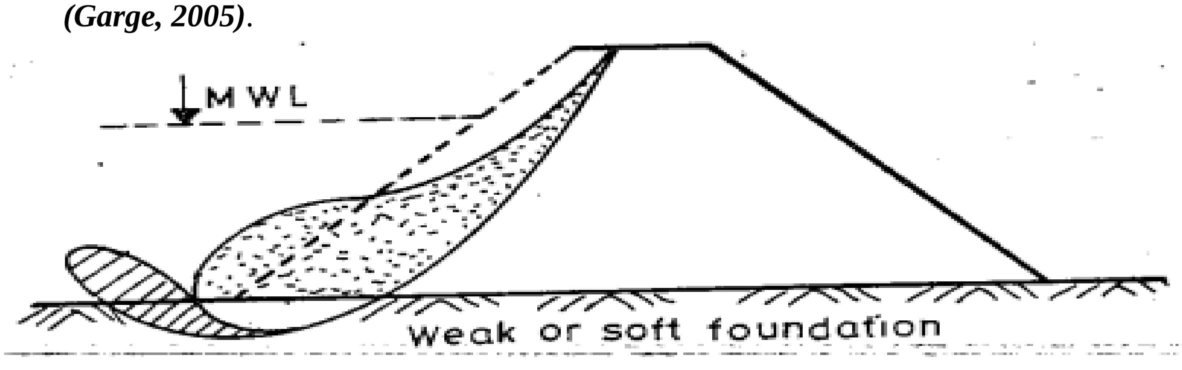

Figure 8: Sliding Due to Weak or Soft Foundation ***(Garge, 2005)***

b. Embankment slide: When the steepness of the side slopes of an embankment dam exceed the allowable limit, the shear stress that is induced in the embankment will exceed the shear strength mobilized by the fill material and the embankment slopes slide.

Slope Stability Analysis

Stabilityin the broadest sense is that, thecapability of the side slope to withstand the disturbing forces tending to move fills materials down slope ***(Western Dam Engineering volume-I, 2013)***.

An earth embankment dam usually fails, because of the sliding of large soil mass along a curved surface ***(Garge, 2005)***.The overall stability of an embankment depends on the bearing capacity of the foundation to withstand the pressure of fill materials, the geometry of the embankment section, and additional factors such as presence of water, loading conditions etc.***(Bahru, 2014).***

The level of the stress around the crest of the embankment, sudden removal of the reservoir at the upstream side and washing out of the fill material due to erosion can be mentioned as the major factor which cause slope instability***(Western Dam Engineering volume-I, 2013)***.

The main objective of slope stability analysis is to ensure the stability of the embankment slopes by determining the factor of safety for different critical loading conditions of the embankment. The stability of the upstream and downstream slopes of the dam embankment are analyzed for the most critical or severe loading conditions that may occur during the life of the dam. These loading conditions typically include:

- Loading condition at the End of Construction
- Loading condition at Steady-State Seepage
- Loading condition at Rapid (or Sudden) Drawdown

Steady State Seepage Loading Condition

When construction is ended, the water starts to collect behind the dam and then water seep through the body of the dam. This loading condition is called steady state seepage loading condition. Since most of the time the upstream face of the dam is in contact with reserved water, this loading condition is considered as critical primarily loading condition to check the stability of the downstream slope (USSD. 2007).

Loading condition at Rapid (or Sudden) Drawdown

Rapid draw down is very similar with that of reservoir operation for pumping in such a way that the water in the reservoir drops down within a short time. Due to this reason the stability of upstream slope of embankment has to be checked for this loading condition ***(USSD, 2007)***.

Loading Conditions and their general safety factor

According to USACE, 2003the factor of safety for different loading condition is listed in the Table 1 below.

Table 1: Minimum Factor of Safety for Different Loading Conditions

Case	Loading Condition	Critical Slope	FoS
1	End of construction	Upstream	1.3
		Downstream	1.3
2	Sudden drawdown	Upstream	1.3
3	Steady state seepage	Upstream	1.5
		Downstream	1.5
4	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

Limit equilibrium method

Limit equilibrium method (LEM) has been introduced in 1930s. In this method, once sliding surface is assumed and divided into a number of differential slices, the disturbing and stabilizing forces and moments are determined for each slice.Finally the factors of safety against sliding become the ratio of the resultant resisting force or moments to the resultant disturbing force or moments. One of the earliest LEM was the Swedish circle method / the Fellenius method. The method relies on circular slip surface, and it has been observed to underestimate the value of the factor of safety. This major problem prompts others such as Bishop to develop improved methods with greater accuracy of factor of safety ***(Carol and Zeena, 2014).***

2.4.2.3 Deformation Analysis

The material used to fill the embankment develops their own weight which leads the structure to significant deformation and displacements. Recommendabledisplacement/or settlement may be allowed, but high internal stress leads to excessive deformation. The stress developed through the dam body or foundation is mainly due to the overall weight of the dam and reservoir operation including sudden drawdown of the reservoir in the dam. Most of the failures of embankments were registered due to excessive deformation rate except the failures due to unexpected events like earthquake, overtopping, etc. according to *Ashok, 2014* among the dam failed incidentally, many of them failed due to deformation problem. Deformation failure is considered as the major mode of failure of embankments in such a way that 6% of the embankment dam failure is registered due to slope instability whereas deformation failure cause around 37% of embankment failure ***(Foster et al 1998, 2000).***Basically there are two kind of stress analysis which involve in the examination of the existed and the new embankments. The first one is about the total tress and effective stress is the second. The analysis of deformation requires consideration of stress conditions imposed during construction and the stress-strain relationship of the material used in the embankment construction.

3 MATERIALSandMETHOD

3.1 Description of the Study Area

3.1.1 Location of the Study Area

Gefersa reservoir is found in Oromia region 18 km far from Addis Ababa in west direction. This reservoir expanded in between Wechecha and Entoto mountains covering a catchment area of 5,500 ha***(Fasil, 2012)***.

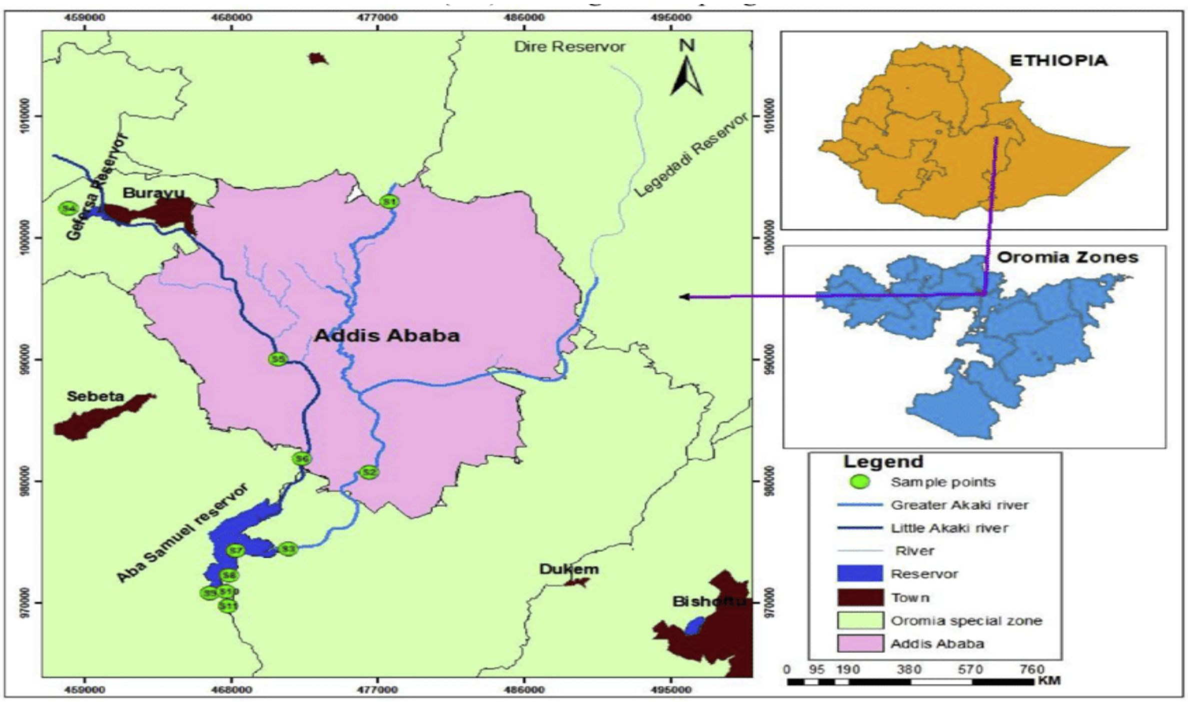

Figure 9: Gefersa Dam Site (https://www.researchgate.net/figure/map of Ethiopia top right Oromia)

3.1.2 Climate

The Gefersa catchment has an annual rain fall ranges from 1200 to 1300mm which is considered as high rainfall record. This catchment area has almost similar weather condition as Addis Ababa since it is near to the capital city. The season from July to September is considered as wet/summer season and the season from October to June taken as warm/winter season. This catchment area have temperature ranges 3^{o}c minimum to 25^{o}c maximum ***(Fasil, 2012).***

3.1.3 Land Cover and Land Use

The land use and land cover around the project area is dominated by grass and forests. Specifically, the grass and woods which cover the land in the project area are cultivated fields, Eucalyptus woodland (mature and young), wooded shrub and grassland, pine woodland, grassland (wet and dry) bare soil, built-up areas (paved road, dam, concrete buildings) and a water body. The cultivated land is located far from the project area, where as the project is surrounded by grasses and eucalyptus wood***(Fasil, 2012).***

3.1.4 Hydrology

The Akaki River is one of the major water source of population of AddisAbaba and its surrounding which cover a catchment area of 1500 square kilo meter. The initial location of this AkakiRiver is found around Entoto area and it goes about 95km to join a river called Awash near Dodota. The AkakiRiver is a combination of two independent rivers. The first one is big Akaki river in the eastern part and the second on is little Akaki river in the western part. Confluence of the two independent rivers is formed at an artificial lake called Abba Samuel reservoir which was reserved around 1930's***(Alema, 2009)***.

In 1981, Akaki river provided with gauging station around Addis Ababa- DebreZeit road for the first time without accounting the runoff from little Akaki river. The averageyearlyflow of Akaki River was recorded for successive 25 year starting from 1981 up to 2005 at the gauging station. As the recorded history at the station shows, the average annual flow of the river is about 9517l/s/km^2***(Alem, 2009).***

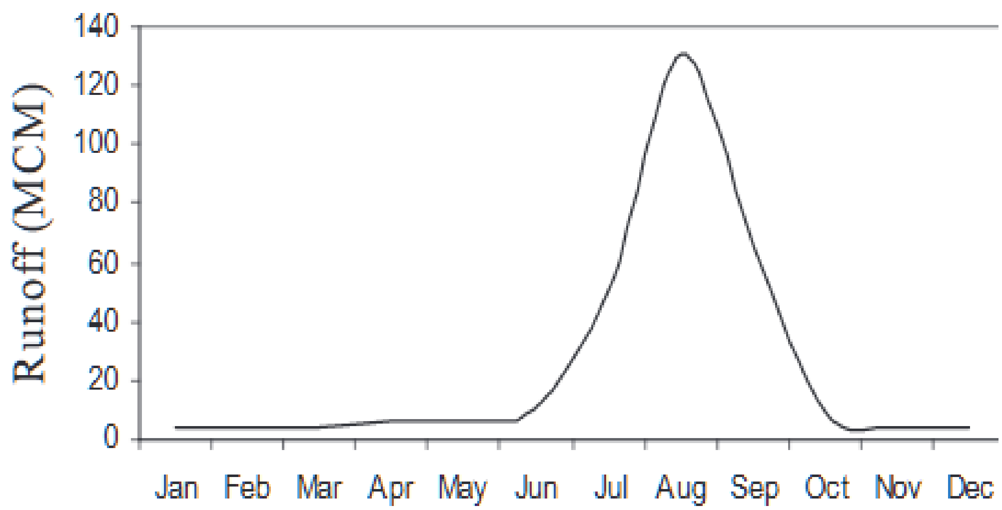

Figure 10: Monthly variation of runoff at Akaki Bridge ***(Alem, 2009)***

As the hydrograph in the above figure shows the rainy season is found in between June up to September and the peak flow is occur in the month of august. Since Addis Ababa is the capital city of the country and the center of commercial practices, significant sewerage from residential, industries and factories have been disposed to the main river of Akaki. This implies that the significant part of flow in AkakiRiver is sewage comes from different sources. Due to this sewage disposal to the river, the flow in the river increase with time**(Alem, 2009).**

In Akaki catchment, there are three basic reservoirs which facilitate the transfer of water from one section to another section of catchment in the form of water supply for domestic and industrial consumptions. So, it is clear that this human interruption has great impact on water balance determination of the catchment. On the other hand, sewage from Addis Ababa city is discharged to both big and small Akaki Rivers, hence this become another major factor which affect the hydrological process of the catchment**(Alem, 2009).**

3.1.5 Geological setting

The back history of the geological condition of the Akaki catchment comes from the gradual change and development of mountains and Plateau of our country Ethiopian***(AAWSA, 2000)***. The geology of the catchment area is dominated by volcanic rock and residual soil materials which contain black cotton soil***(Alem, 2009)*****.**

3.1.6 Geology

Different Studies was conducted which describe the geology of the river.Among those study Morton et al. (1979) and Girmay and Assefa (1979) can be mentioned as the most significant studies. As shown in figure 11 below, a hydro geological map was developed using the data from filed investigation and the past historical***(Alem, 2009)*****.**

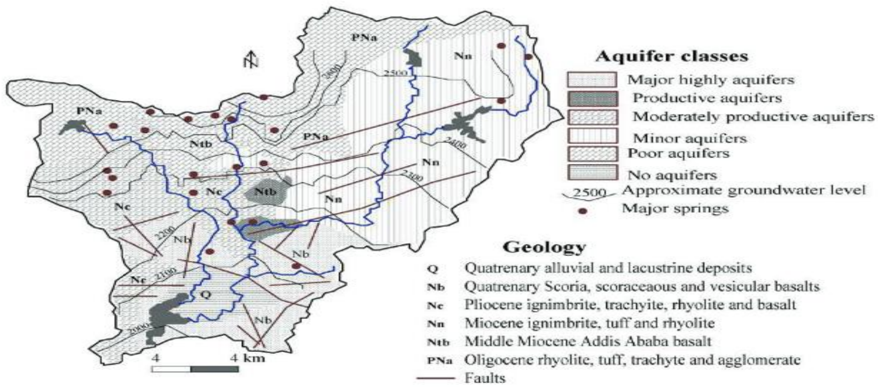

Figure 11: Simplified Hydrological Map ***(Alem, 2009)***

Alaji Formation: as***(Morton et al., 1979)***this type of formation mainly includes rhyolites, trachytes, tuff, agglomerate, and aphianitic basalt. This type of formation is abundant in the northern and central part. This rock type is future classified into Alaji rhyolites and Intoto Silicic. This formation mainly found in between Intoto Mountain and Blue Nile basin at the northern part***(Alema, 2009)***.

Addis Ababa Basalts:according to ***(Vernier, 1985)*** these basalts cover the Intoto Silicics and found in the central and southern part of Addis Ababa. Olivine porphyritic basalt outcrop in central Addis Ababa with a thickness varying from 1 m or less in the foothills of Intoto to more than 130 meters in central Addis Ababa***(Alema, 2009)***.

Younger Volcanic: this type of rock group is the combination of the Nazareth Group and Bishoftu Formations. The Nazareth Group rocks found mainly around south of the Filwuha Fault. Around 500m thickness of the lava sequence is considered as exposed section(***Girma, 1994***).Olivine porphyritic basalt, scoria, vesicular and scoraceous basalt, and locally trachy-basalt lava flows are the basic formation of younger volcanic***(Alema, 2009)***.

3.2 Gefersa Dam & Reservoir

3.2.1 Gefersa Reservoir

Gefersa reservoir is the first and the major potable water source for the populations of Addis Ababa capital city of Ethiopia. When Addis Ababa became the capital city of the country, Gefersa-I dam was implemented to treat the water that comes from springs of Entoto. Through time this dam was rehabilitated due tosedimentation problem in the reservoir. Because of this rehabilitation work, the dam was named as Gefersa-I/II dam rather than Gefersa-I. Even if the dam goes through rehabilitation work, sedimentation problem again become major issue in the reservoir. Due to this reason, another small dam called Gefersa-III homogeneous embankment dam was constructed at the upstream of the main dam (Gefersa I/II dam) to trap the incoming sediment and to store additional volume of water.Gefersa III reservoir, which is constructed in 1966, is located upstream of Gefersa I/II dam about 1km to the west. It is used as both storage and silt trap***(Alema, 2009)***.

The River of Gefersa and also the streams which confluence this river as a feeder are part of the Akaki river catchment. From this reservoir around 30,000 cubic meter of treated water addressed to the capital city for the partial fulfillment of population's daily water consumption. The total water storage capacity of Gefersa-I/II and Gefersa-III is 6,500,000 cubic meter and 1,500,000 cubic meters respectively.Sediment deposition is one of the major problems in reservoir impoundment, as it significantly affects their capacity and results in deficit of potable water for the population of capital city of Ethiopia. The sediment increment and cost of water treatment have direct relation, in such a way that when the rate of sediment deposition increase, the cost demanded for the treatment of sediment water becomes expensive. According to the surveys conducted in 1979 and 1998 the average sediment flow spilled out from the basin is estimated as 22,252m3/year or 575 tons/km2/year as soil loss from the catchment. A Bathymetric survey carried out in 1999 indicated that around 6.23MCM or 0.36% storage of Gefersa reservoir is lost due to the entrance of about 575 tons/Km2/year sediment***(Fasil, 2012).***

Basically Gefersa-III embankment dam heightening is proposed to increase its storage capacity which serves for both sediment trap and to store additional live storage (water). Once the construction of heightening part is accomplished, the heightening of the embankment is expected to increase the storage capacity of the reservoir by $3Mm^3$.

3.2.2 Gefersa Dam

3.2.2.1 Geometry of Existing Dam

Gefersa III dam is a 19m high homogeneous earth dam. It is constructed using clay soil. The dam is 5m wide at the crest and has upstream and downstream slopes of 1V: 2.58H and 1V:2.27H respectively. To facilitate proper drainage of the seeping water, a 45m long and 0.8m thick horizontal drainage filter is provided. Figure 8 shows the typical section of the dam at the deepest section. The detail of the engineering properties of the fill material and foundation has been presented in chapter four of this study (see Table 4, 5, and 6).

3.2.2.2 Foundation condition/treatment

The foundation condition of the dam site of Gefersa-III embankment is mainly covered by thick soil material. The total thickness of the soil foundation is about 20-25m below the ground surface. The outer most layers (about 8m) of the foundation thickness are dominated by organic maters. And the rest part of the foundation thickness was covered by a soil called clayey sandy silt. The safe bearing capacity of the foundation was ranges in between 67-128kpa.

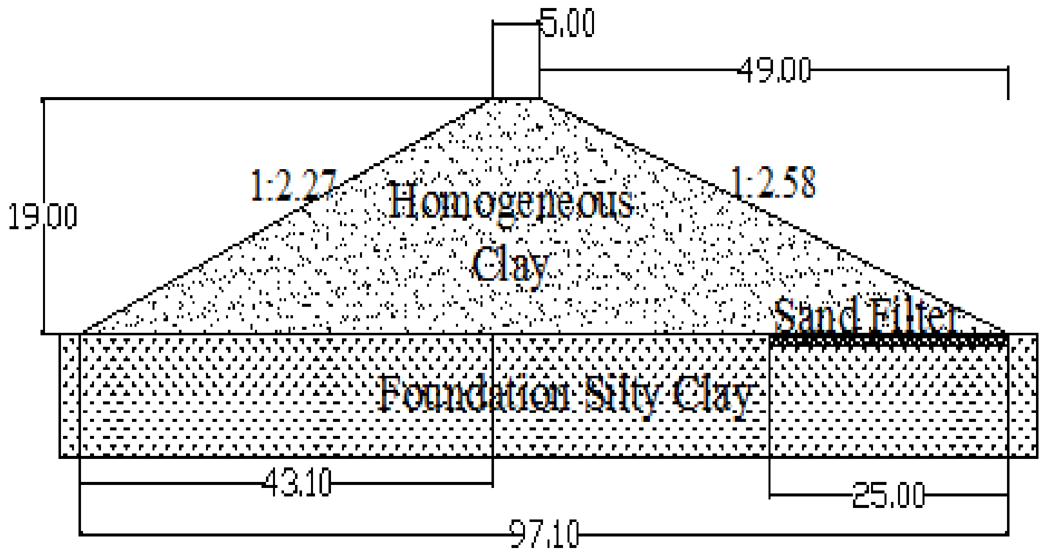

Figure 12: Existing Geometry of Gefersa III Dam

3.3 Materials Used

3.3.1 Geostudio Software

Geostudio is software developed on the base of finite element method which used to conduct analysis over soil and rock material. This software is suited with collection of different products which used to work on the wider range of engineering structure like dams, and other related structures. Among the products SEEP/W, SLOPE/W, SIGMA/W, and QUAKE/W can be mentioned as the basic component of Geostudio which used for modeling seepage, slope stability, ground deformation, and earth quake analysis respectively. the main advantage of this software is that, it generate tools like, graphing, contour, Isoline, and animations which help the user to understand what is done in the modeling clearly. In the following some article the main features of the software suits that are used in this study are highlighted:

SEEP/W: is one of a product in Geostudio, used to simulate the flow of waterthrough a pours medium especially through soil and rock material. The basic principle which applied to develop seep/w is the principle of Darcy called Darcy's low.The SEEP/W software mainly applied to solve two-dimensional flow problem in multiple layers of construction materials. Seep/w is not only used to modelthe movement of surface water, but it can model and analyze the movement of ground water too***(Seepage Modeling, 2012).***

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SLOPE/W: It is one feature of Geostudio software, which used to conduct analysis of sophisticated stability problems. The basic advantage of this software is that, it allows the user to use finite element computed pore-water pressures and stresses in a stability analysis***(Stability Modeling, 2012).***

Sigma/w: It is one of the components of Geostudio software which applied to analysis stress and deformation of earthen structures within the help of finite element principle. The basic character of this software is that, its flexibility to perform simple and complex analysis. This software allows the user to perform a simple linear elastic deformation analysis, and also it is flexibly to conduct a complex nonlinear elastic plastic stress analysis. Basically, this software helps to analyze deformation around the foundation and embankment of earth fill structure***(Stress-Deformation Modeling, 2012)***

3.4 Methodology

3.4.1 General

When an existing dam is proposed for heightening; the first task becomes selecting an appropriate heightening type. This is because the new heightened section has to be compatible with the existing fill type and also the heightened dam has to be compatible with the existing foundation condition of the dam.

3.4.2 Alternative OptionsforGefersa-III Dam Heightening, their advantage and disadvantage

In case of dam heightening the basic task is to deiced suitable heightening type which becomes compatible with the existing condition of dam body and its foundation. To come up with the best heightening type,different alternatives have to be proposed and the entire considered alternative has to be modeled to compare and contrast their merit and demerit. In the literature review section, different alternative heightening type with their merit and demerit are described. In this case study; among those available alternatives two basic alternatives (i.e. Homogeneous and Concrete Face Rock Fill heightening type) was considered and depending up on the modeling result the one which become more stable with regard to seepage, slope, and deformation failure has been selected as the suitable heightening type. It is to be recalled that homogeneous heightening type has a great advantage in that this type of heightening solve the complexity of the construction and reduce the need of transition material in between the old and the new section of the embankment. In contrast, concrete face rock fill heightening has the advantage of requiring small volume of fill material since rock fill embankment can be constructed with relatively steep slope.

3.4.2.1 Homogeneous heightening type

This type of heightening was proposed as a basic alternative to take the advantage of material compatibility.

Geometry of Homogeneous Heightening

The preliminary design of an earth dam is done on the basis of existing dams of similar characteristics and the design is finalized by checking the adequacy of the selected section for the worst loading conditions ***(Garge, 2005)***. According to ***Garge, 2005*** recommendations for selecting suitable values of top width, freeboard, upstream and downstream slope, drainage arrangements, etc. are given below:

- Increase dam height: the proposed overall height increment on the existing embankment was around 7m including the free board.
- Freeboard: It is the vertical raising between the crest of the spillway get and the top of the dam ***(Garge, 2005).*** It is used to control overtopping over the body of the dam. U.S.B.R recommend the following numerical value of freeboard with respect to dam heights as shown in table 2 below:

Table 2: U.S.B.R Recommendations for Freeboard for Earth Dams ***(Garge, 2005)***

Spillway Type	Height of dam	Minimum freeboard over MWL
Uncontrolled (i.e. free) spillway	Any height	Between 2m to 3m
Controlled spillway	Height less than 60m	2.5m above top of gates
Controlled spillway	Height more than 60m	3m above top of gates

The height of the new heightened Gefersa-III dam is around 24m with controlled spillway hence the free board was fixed as 2.5m above MWL.

- Top width**:** Itis the most upper section of dam in which roadway is provided**.** The top width is very important section to keep the phereatic line with in body of the dam and also to resist the effect of earth quake. The minimum roadway required on the dam highly governs the size of top width. According to ***(Garge, 2005)*** the top width (b) of the earth dam can be selected as per the following recommendations

$b=\frac{H}{5}+3$, for very low dams ***(Garge, 2005)*** **……………………………….…..…….** *(1)*

$b=0.55\sqrt{H}+0.2H$, for dams lower than 30m ***(Garge, 2005)*** **………………** *(2)*

$b=1.65(H+1.5)^{1/3}$, for dams higher than 30m ***(Garge, 2005)*** **…………….………** *(3)*

Where H is the height of the dam in meter

As it mentioned above the total height of the new heightened Gefersa dam is 26m which less than 30m. Hence top width has to be calculated using Equation (2)

$$b=0.55\sqrt{H}+0.2H$$

$$=0.55*19^{0.5}+0.2*19$$

=6m, the top width for the new heightened dam was fixed as 5m.

- Upstream and downstream slopes: the steepness or flatness of side slope is highly governed by different factors. The height of the dam is one of the factors which govern the side slope selection. As the height of the dam increase, the side slope has to be flatter enough to withstand the deformation problem. Beyond height of the dam, foundation condition and nature of the dam are also basic factors which govern the side slope selection. The recommended values of side slopes as given by Terzaghi are tabulated in Table3 below.

Table 3: Terzaghi's side slope for Earth Dams ***(Garge, 2005)***

Type of material	U/S slope (H:V)	D/S slope (H:V)
Homogeneous well graded	2.5:1	2:1
Homogeneous course silt	3:1	2.5:1
Homogeneous silty clay i) Height less than 15m ii) Height more than 15m	 2.5:1 3:1	 2:1 2.5:1
Sand or sand and gravel with a central clay core	3:1	2.5:1
Sand or sand and gravel with R.C. diaphragm	2.5:1	2:1

The Gefersa-III existing dam is homogeneous clay fill dam with height 19m, designed with upstream and downstream side slope of 2.27H:1V and 2.58H:1V respectively.

- Berm:This component of the dam is one of the important sections to prevent the formation of gullies due to rainfall over the downstream face of the dam***(Garge, 2005)*****.** For this case study two berms were provided. The first one was provided on the crest of the existed dam to create safe working space and the second one was provided at the downstream side of the dam to ensure that the downstream side is free of the formation of gullies
- Clearance: it is obvious that the most outer layer of the downstream of the dam may become decomposed or inorganic; therefore 0.5-1.0m (allowable limit) of the downstream side of the dam has to be clear before new material fill is started.

Accounting all the above geometric sections, the new heightened dam have the geometries shown in Figure 13 below.

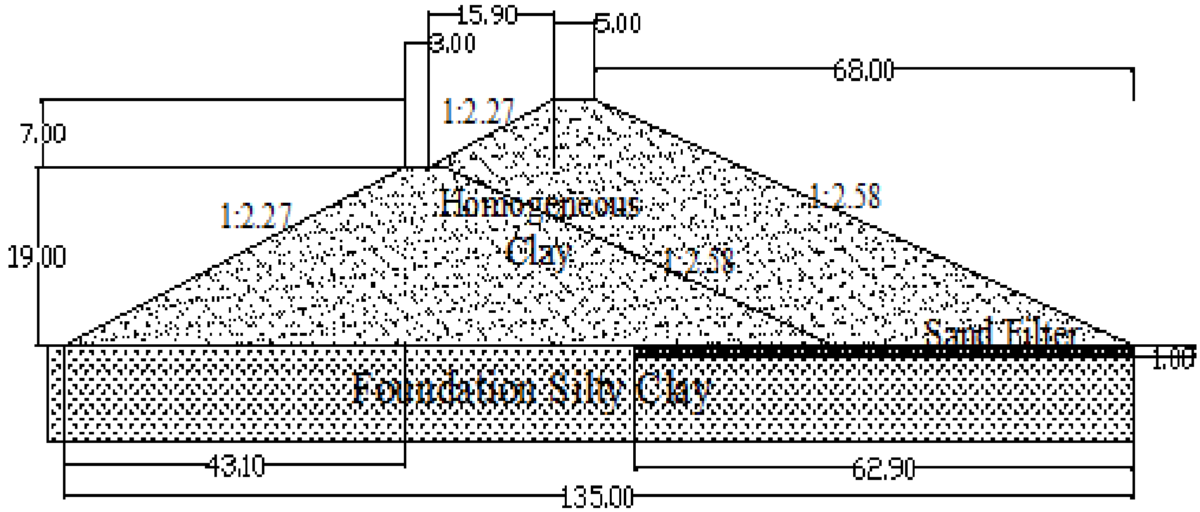

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Figure 13: Geometry of Homogeneously Heightened Gefersa-III Dam

3.4.2.2 Concrete face rock fill

As mentioned in the literature review, concrete face rock fill heightening has several advantages. This type of heightening can adopt relatively steep embankment slope leading to significant reduction in e volume of fill material required. Furthermore, since the construction is independent of weather and seasonal variation, the construction period is usually short.

Geometry of concrete face rock fill Heightening

While concrete face is provided for the heightened embankment, a berm is formed on the crest of the existing embankment which will be used as working space during construction of the heightening part.Foundation slab is provided on the crest of the existing embankment which is used as a base for the facing slab. The facing slab will have a dimension of 760mm at the base (old crest) and 500mm at the crest of the new heightened dam.Since the new fill material is different from the fill material of the existing embankment, transition material is required in between the rock and existing clay material. Sand filter material was used as transition material in between the two fill materials (rock and clay). To keep the continuity of the operation of reservoir in the existing embankment during the heightening, most of the fill is proposed to be loaded at the downstream side of the embankment. To do so the embankment is shifted to the downstream with 10.6m additional base width.

The thickness of facing slab was calculated as the equation given by:

d=488+0.066H……………………………………………………………… (4)

Where:

d: is the thickness of facing slab in mm (for 488mm≤d≤760mm)(***Ljubomir, 2005).***

H: the height from the crest

On the crest of the old dam (when H=7m from the crest):

d= 488+0.066*7000=950mm>760mm, hence provide 760mm

On the crest of the newdam (when H=0m from the crest):

d= 488+0.066*0=488mm hence provide 500mm

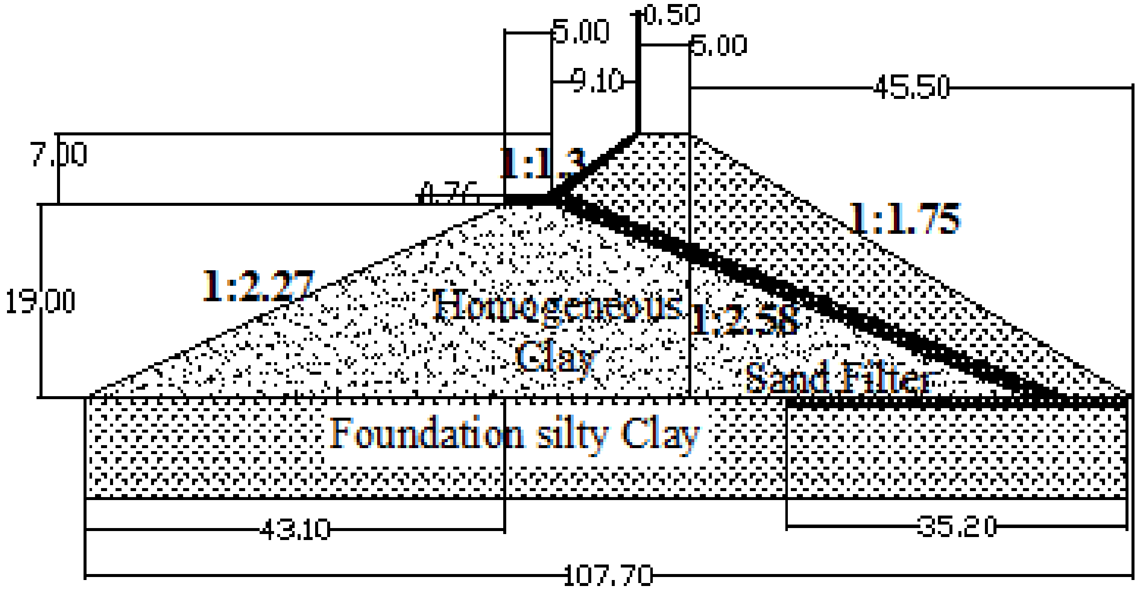

Figure 14: Geometry of concrete face rock fills Heightened Gefersa-III Dam

4 RESULTS AND DISCUSSION

4.1 Dam materials

The different engineering properties of the embankment material are very essential input to run Geostudio software. As it is mentioned in the methodology section, two different heightening alternatives are proposed with different fill type. The materials used for those heightening alternatives including the existing embankment are listed with their engineering properties as shown in the three successive tables below.

Table 4: Engineering properties of material for existed embankment***(J.E. Bowles, 1996)***

S.No	Material Type	Permeability (m/sec)	Water content (%)	unit weight (KN/m3)	Cohesion (Kpa)	Poisson's ratio	angle of friction(^{0})	modulus of elasticity (Kpa)
1	clay/ embankment	$1.69*10^{-7}$	50	17.5	90	0.45	7	32,500
2	sand, Silty clay/foundation	$1.8*10^{-7}$	45	16	75	0.325	10	20,000
3	sand/ filter	0.001	20	18.28	11.17	0.2	35.5	17,500

Table 5: Engineering properties of material for homogeneously heightened embankment***(J.E. Bowles, 1996)***

S.No	Material Type	Permeability (m/sec)	Water content (%)	unit weight (KN/m3)	Cohesion (Kpa)	Poisson's ratio	angle of friction(^{0})	modulus of elasticity (Kpa)
1	clay/ embankment	$1.69*10^{-7}$	50	17.5	90	0.45	7	32,500
2	sand, Silty clay/foundation	$1.8*10^{-7}$	45	16	75	0.325	10	20,000
3	sand/ filter	0.001	20	18.28	11.17	0.2	35.5	17,500

Table 6: Engineering properties of material for concrete face rock fill heightened embankment***(J.E. Bowles, 1996)***

S.No	Material Type	Permeability (m/sec)	Water content (%)	unit weight (KN/m3)	Cohesion (Kpa)	Poisson's ratio	angle of friction(^{0})	modulus of elasticity (Kpa)

1	clay/ embankment	$1.69*10^{-7}$	50	17.5	90	0.45	7	32,500
2	sand, Silty clay/foundation	$1.8*10^{-7}$	45	16	75	0.325	10	20,000
3	sand/ filter	0.001	20	18.28	11.17	0.2	35.5	17,500
4	Gravel/ Rock	0.01	0.09	26.4	200	0.35	22.5	150,000
5	Concrete/ Asphalt	$1.8*10^{-10}$	0.01	23	600	0.4	10	15,000,000

4.2 Seepage Analysis

To check whether the seeping water is in the allowable limit or not seepage modeling was conducted using seep/w of the Geostudio software.

4.2.1 Boundary conditions

In addition to engineering properties of the fill material, boundary condition has to be defined while seepage analysis is conducted. In case of steady state seepage analysis, three basic boundary conditions are defined. Since the upstream face of the embankment is always in contact with water, this upstream boundary condition has to be described.Even if the phereatic line has safely passed through the pipe in the filter, this may hold true for theoretical case only. But for practical case, some seepage may pass to the downstream face of the embankment. Hence the downstream face is always described as potential seepage face boundary condition. Another boundary condition is about zero pressure boundary condition. This boundary condition is always located at the drainage of the embankment in which the pressure near to negligible. In this specific case study those three boundary conditions were prescribed while steady state seepage analysis was conducted.

The steady state seepage analyses is conducted both for the existing embankment condition and for the proposed heightening alternatives as shown in figure 13, 14, and 15below.As the result shows, the numerical value of the seep water through the dam body and foundation is estimated to be$1.3713*10^{-6}$m/sec for the existing embankment, $3.05*10^{-6}$m/sec for homogeneously heightened embankment, and $8.255*10^{-8}$m/sec for concrete face heightened embankment.

Obviously, as the cross section of an embankment is increased, seepage through the embankment also increases. This logic also holds true for this case study as the numerical value of the seepage in the above paragraph shows. When the seepage in case of the existing embankment is compared with the seepage in the proposed heightening alternative embankments, the seepage in homogeneously heightened embankment is increased by $1.68*10^{-6}$m/sec whereas the seepage in concrete face rock fill heightening was reduced by $1.29*10^{-6}$m/sec.

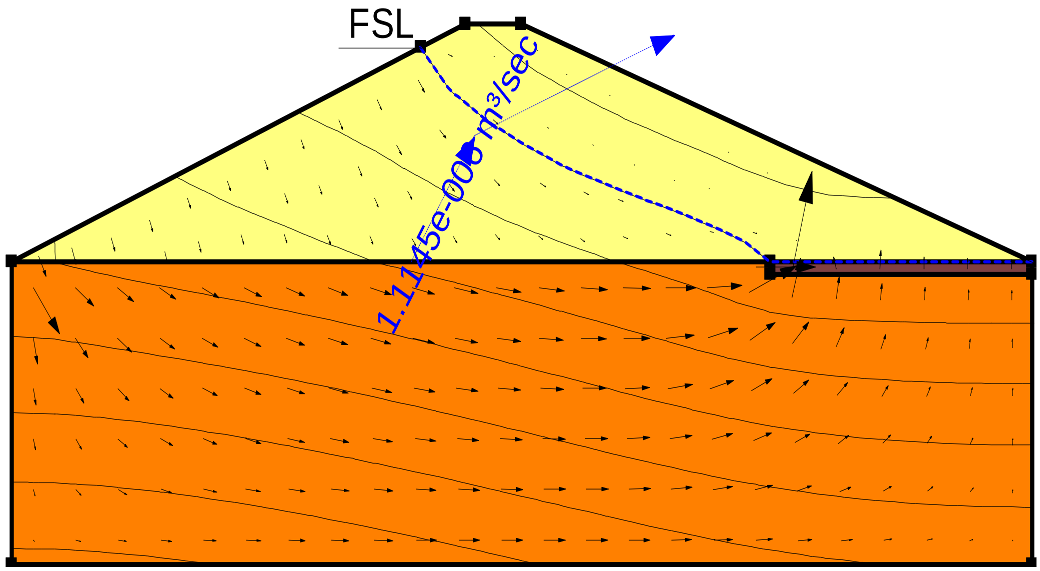

Figure 15: steady state Seepage analysis before heightening

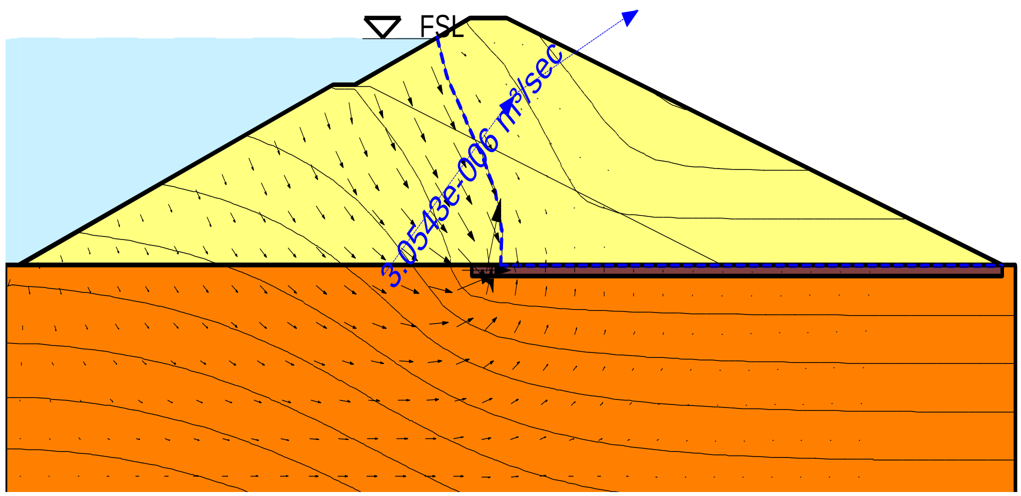

Figure 16: Steady state seepage analysis of homogeneously heightened embankment

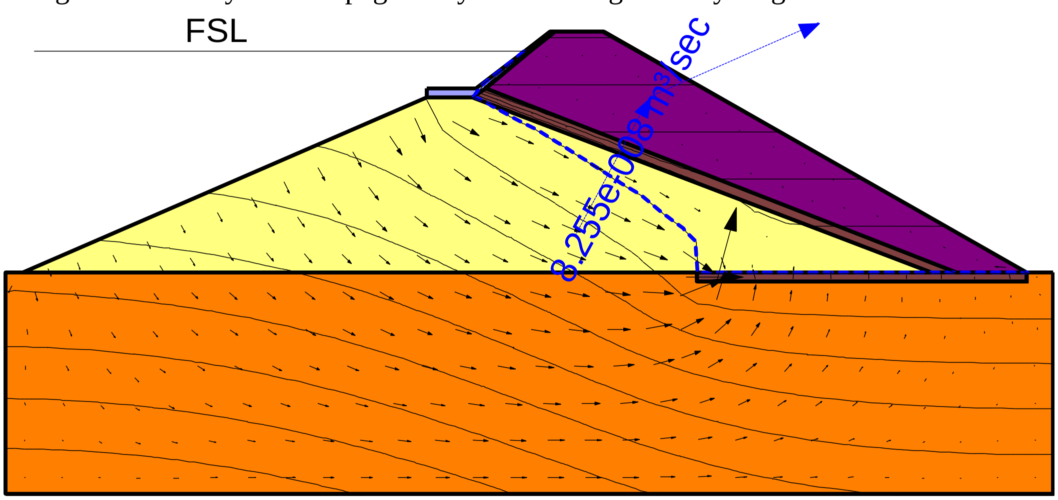

Figure 17: Steady state Seepage analysis of concrete face rock fill heightening

4.2.2 Comparison between the heightening alternatives in terms of seepage

Actually the magnitude of seep water is greater in case of homogeneous heightening than concrete face rock fill heightening. But, as embankment dam is constructed using naturally available material, it is impossible to control seepage of water hundred percent. Due to this reason limited or controlled seepage may be allowed in any embankment dams. As the seep/w result shows, the seep water through the body of the embankment is minimum and the phereatic line also safely pass through the pipe of filter drainage in case of both homogeneous and concrete face rock fill heightening. This implies the seepage water in both alternatives was become under control. But, for comparison, the magnitude of seep water to the body and foundation is greater in homogeneously heightened embankment than concrete face heightened embankment. Therefore concrete face heightening alternative was preferred as a better heightening type with controlled seepage.

4.2.3 Porewater distribution

The pore water pressure is the overall pressure in the dam/embankment due to the impounded water in the upstream side of the dam. Usually the distribution of water pressure decreases from bottom to top in which the water pressure becomes negligible or zero at full supply level and phreatic surface. The pore water distribution modeling was conducted for the existing, homogeneously heightened and concrete face rock fill heightened embankments as shown in figure (16, 17 and 18). As the figure shows, due to its large cross section, the pore water pressure become greater in case of homogeneously heightened embankment than the pore water pressure of concrete face rock fill heightening. This is because in case of concrete face rock fill embankment water may not simply move from one section of the embankment to another section due to the barrier (concrete face) provided as water proof material in the heightened section (refer figure 20).

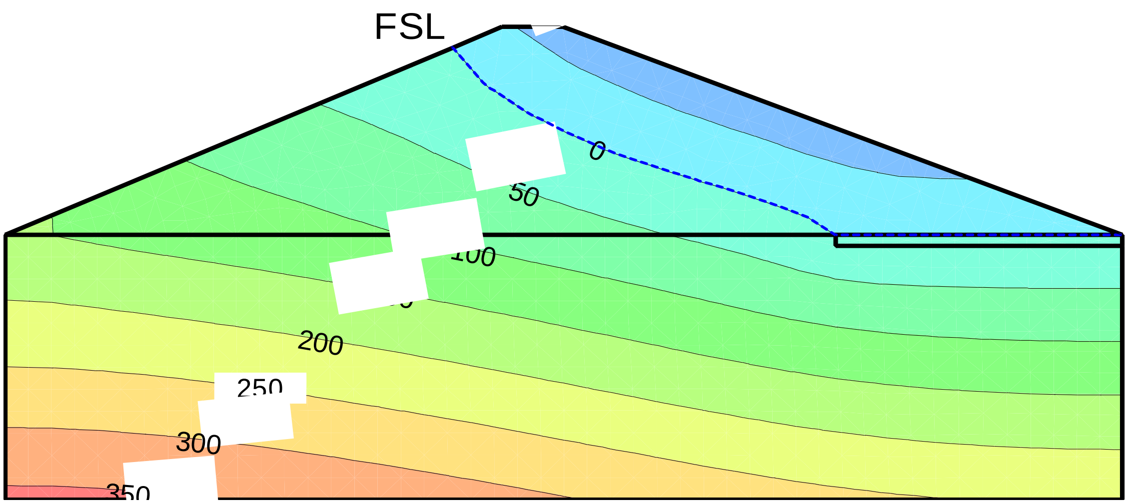

Figure 18: Steady State Pore Water Distribution before heightening

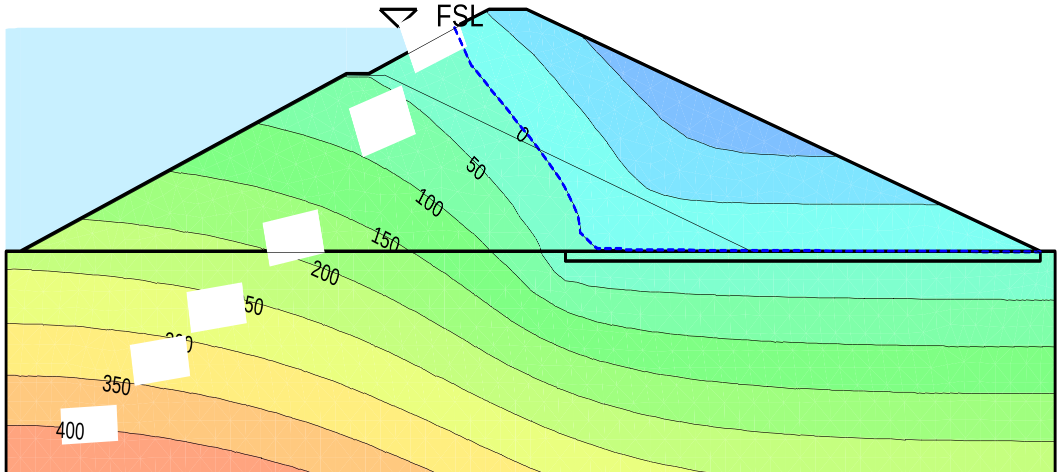

Figure 19: Steady State Pore Water Distribution for homogeneous heightening

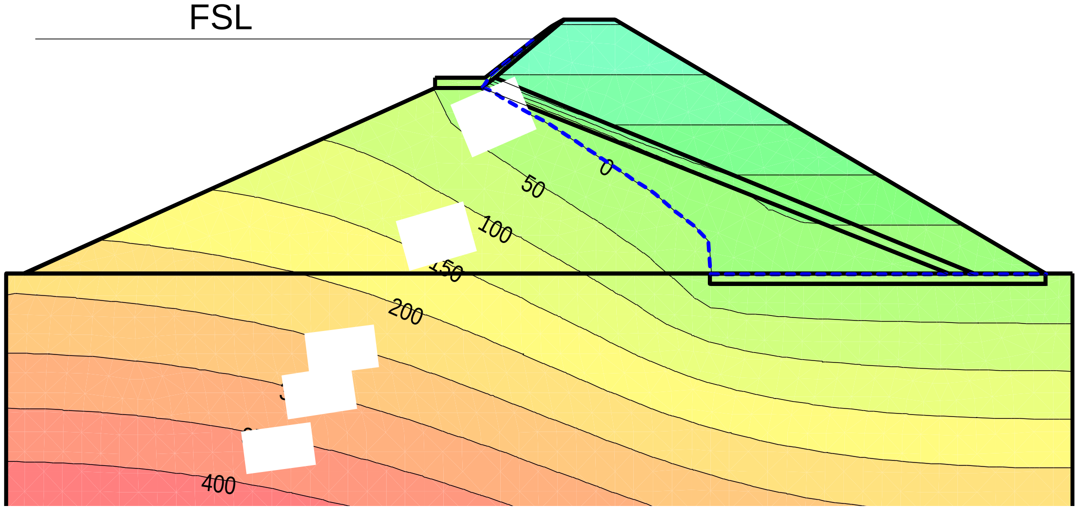

Figure 20: Steady State Pore Water Distribution for concrete face rock fill heightening

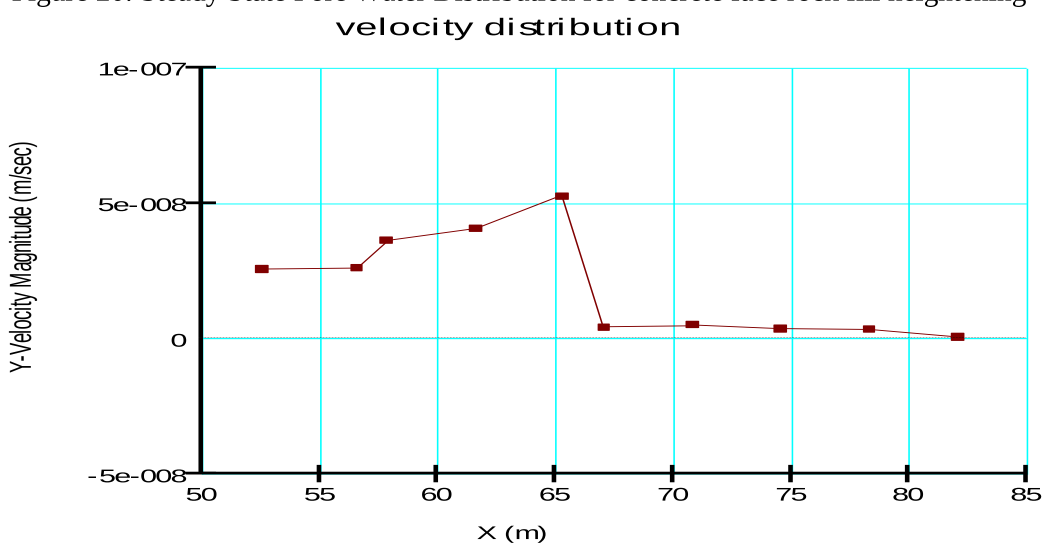

Figure 21: Y-velocity vs X-coordinate (before heightening)

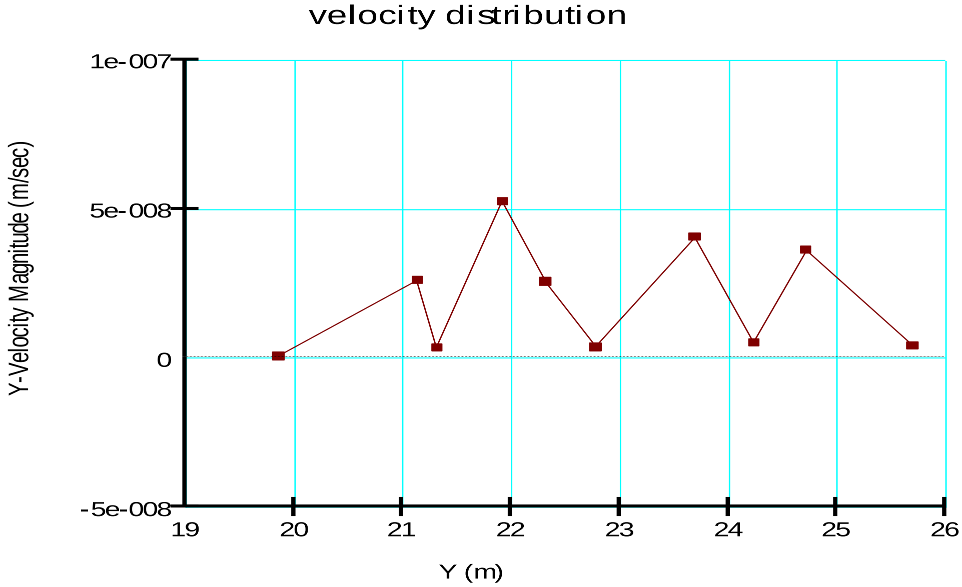

Figure 22: Y-Velocity Distribution vs Y-Coordinate (before heightening)

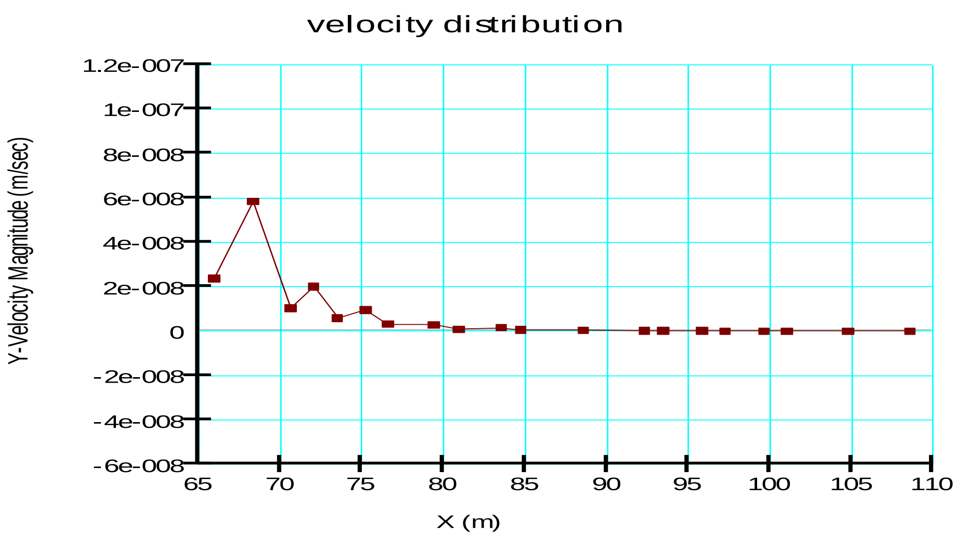

Figure 23: Y-velocity vs X-coordinate (homogeneous heightening)

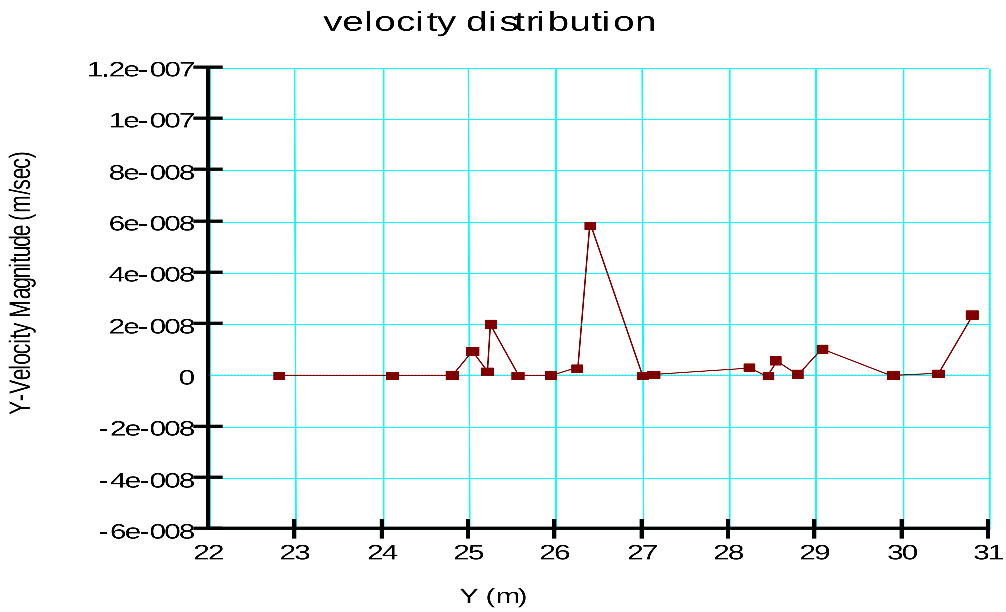

Figure 24: Y-velocity vs X-coordinate (homogeneous heightening)

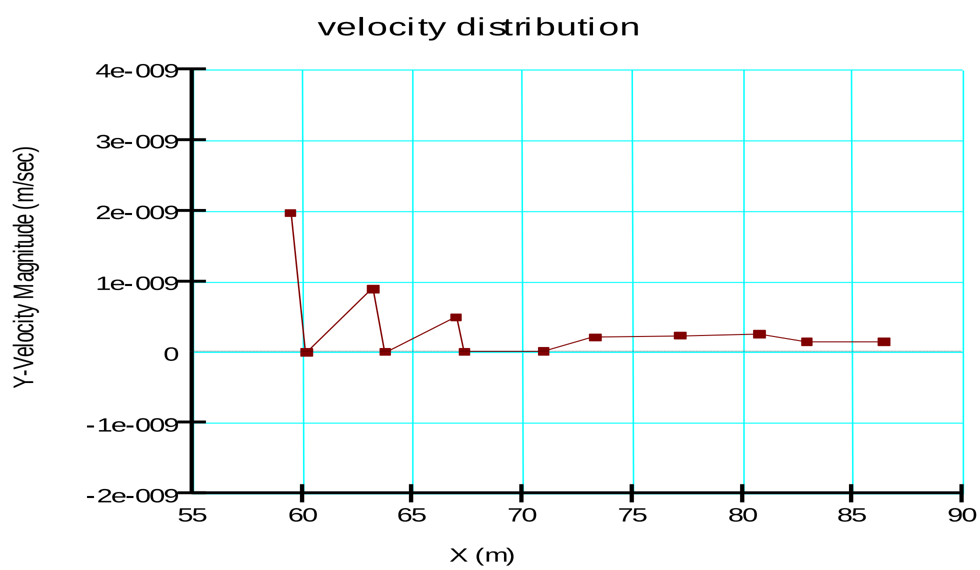

Figure 25: Y-velocity vs X-coordinate (concrete face rock fill heightening)

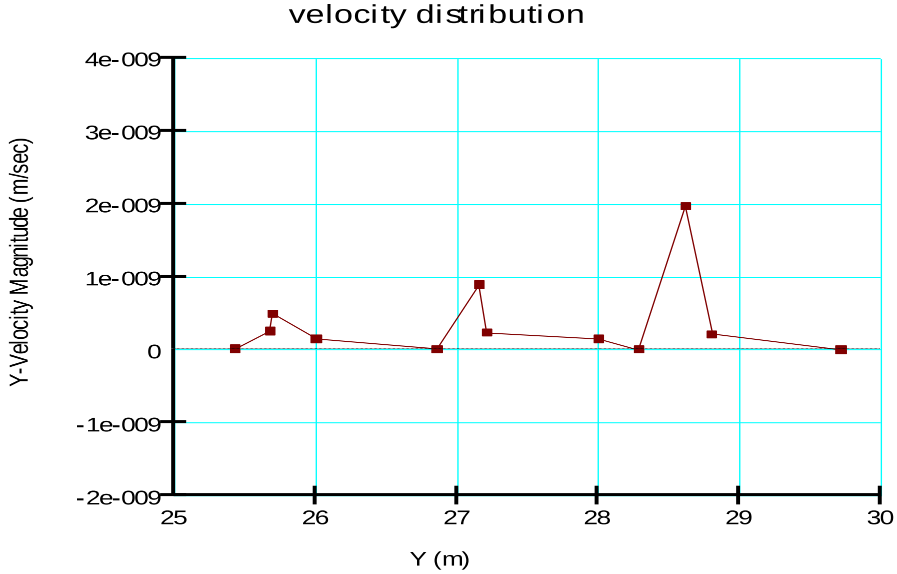

Figure 26: Y-Velocity Distribution vs Y-Coordinate (concrete face rock fill heightening)

4.3 Slope Stability Analysis

The factor of safety of the dam slope under different loading condition (steady state seepage, and during rapid draw down) is calculated using SLOP/W of the Geo-studio 2012 software. The computed factors of safety under the different loading conditions against their slope failures are summarized follows.

4.3.1 Slope stability analysis of existing embankment

Before going through slope stability analysis of the heightened embankment, the stability of the slopes of the existing embankment was checked for two scenarios (i.e. for steady state seepage and reservoir rapid drawdown condition) and numerical values of factor of safety were obtained for both upstream and downstream embankment side slope. As the numerical value of factor of safety shows, the overall embankment side slope fulfill the minimum requirement of factor of safety and the existing embankment side slope was become safe for both steady sate seepage and reservoir rapid drawdown loading condition(see Table 7).

Table 7: computed factor of safety for different loading conditions of existing embankment

	FoS min	Computed factor of safety	
Loading condition		Upstream	Downstream
Steady sate seepage	1.5	3.156	2.184
During rapid drawdown	1.3	2.336	-

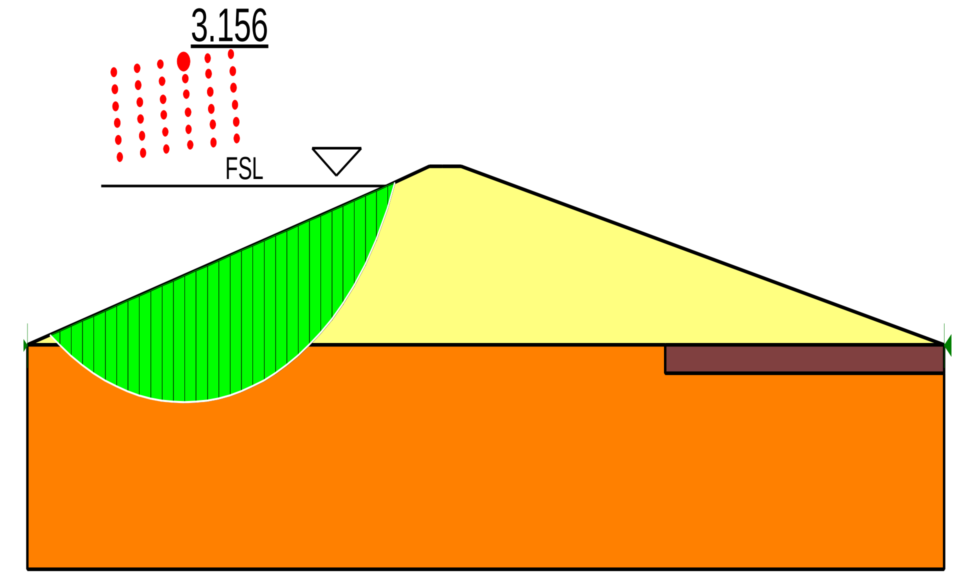

Figure 27: Slope Stability Analysis before heightening under Steady State (Upstream Face)

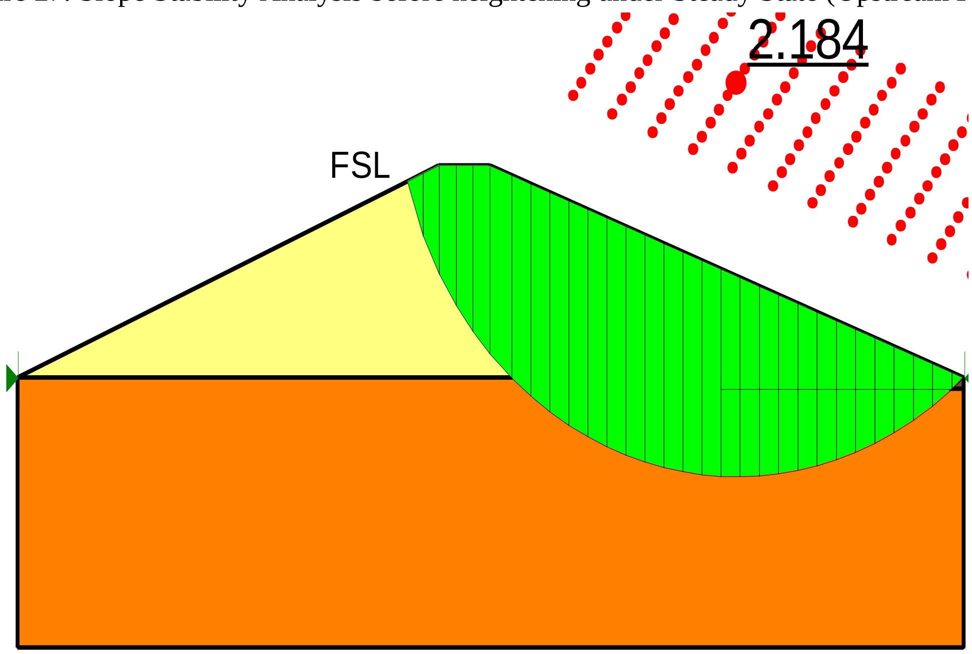

Figure 28: Slope Stability Analysis before heightening under Steady State (downstream Face)

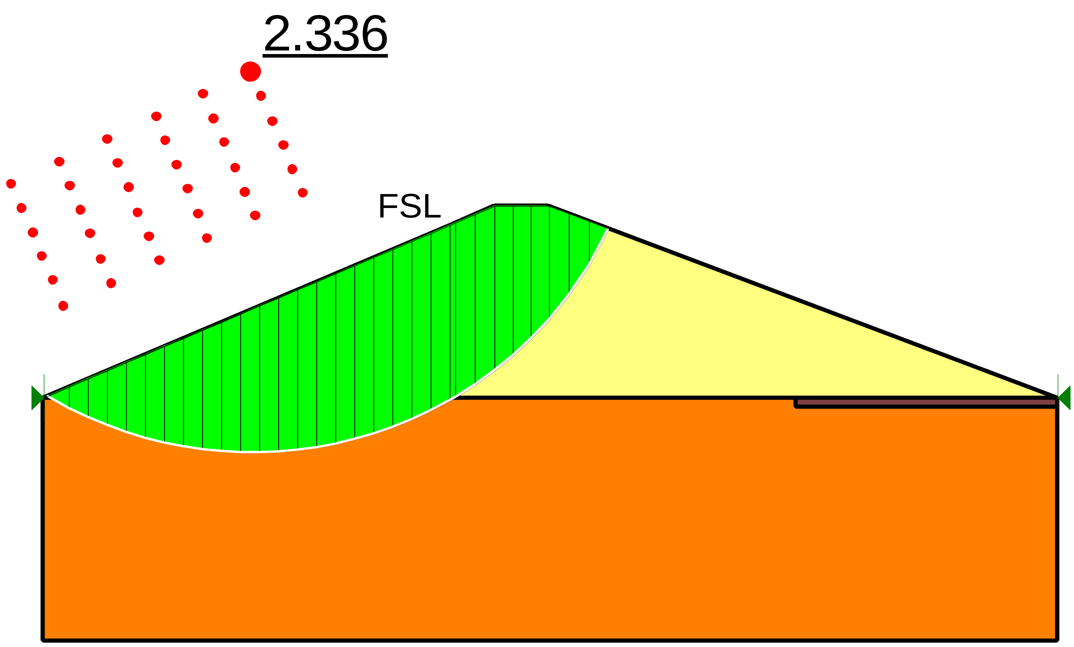

Figure 29: Slope Stability Analysis existed embankment during rapid drawdown (Upstream Face)

4.3.2 Slope stability analysis of homogeneously heightened embankment

As in the case of the existing embankment, slope stability analysis also done for homogeneously heightened embankment with similar loading conditions and the numerical values of factor of safety was obtained for both upstream and downstream side slopes. As the obtained numerical value shows, all the value become greater than the minimum required factor of safety, hence the homogeneously heightened embankment was become safe for both loading condition (see Table 8).

Table 8: computed factor of safety for different loading conditions of homogeneously heightened embankment

	FoS min	Computed factor of safety	
Loading condition		Upstream	Downstream
Steady sate seepage	1.5	3.645	1.912
During rapid drawdown	1.3	1.943	-

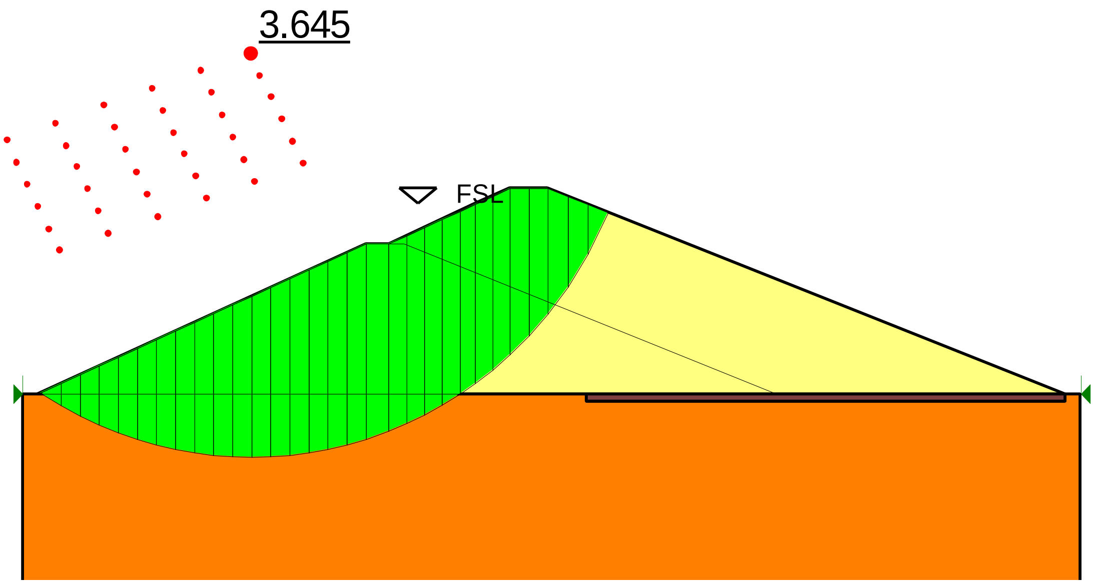

Figure 30: Slope Stability Analysis of homogeneous heightening under Steady State (Upstream Face)

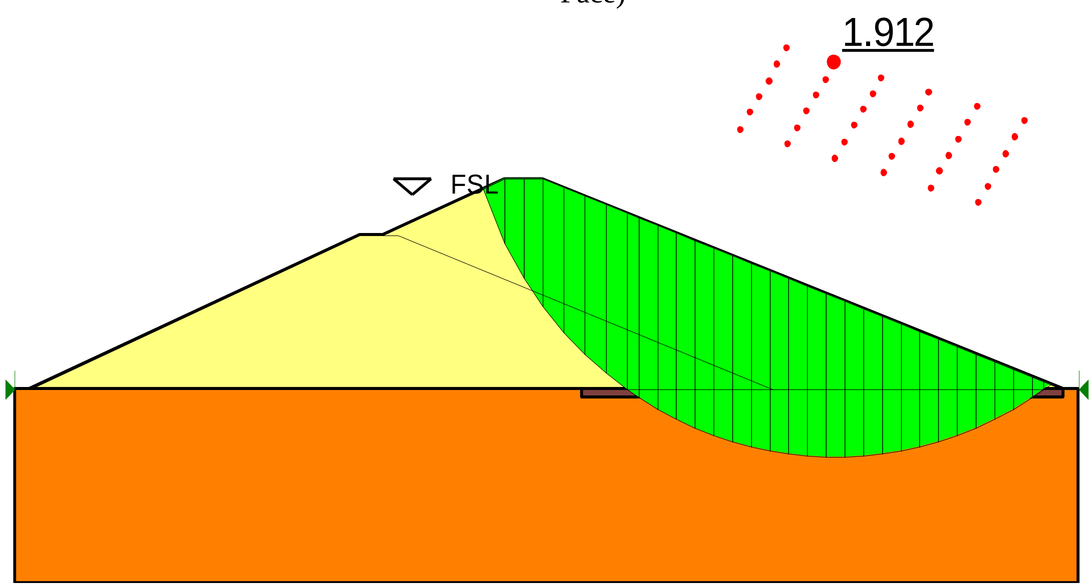

Figure 31: Slope Stability Analysis of homogeneous heightening Under Steady State Seepage (Downstream Face)

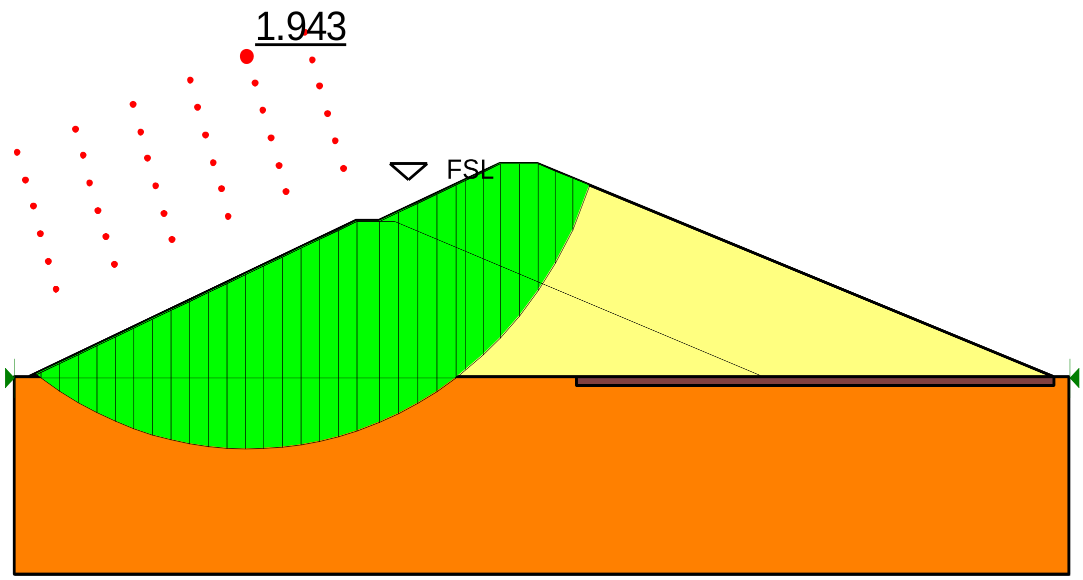

Figure 32: Slope Stability Analysis of homogeneous heightening during rapid drawdown (Upstream Face)

4.3.3 Slope stability analysis of concrete face rock fill heightened embankment

Slope stability analysis was continued to concrete face rock fill heightened embankment with the similar loading condition(steady state seepage and reservoir rapid drawdown loading condition) as existing and homogeneously heightened embankment. Factor of safety on the upstream and downstream side of the embankment were obtained for those loading conditions. As the result indicates, the concrete face rock fill heightened embankment was become safe against slope failure due to having greater factor of safety than the minimum requirements see (Table 9).

Table 9: computed factor of safety for different loading conditions of concrete face rock fill heightened embankment

	FoS min	Computed factor of safety	
Loading condition		Upstream	Downstream
Steady sate seepage	1.5	2.949	1.531
During rapid drawdown	1.3	1.800	-

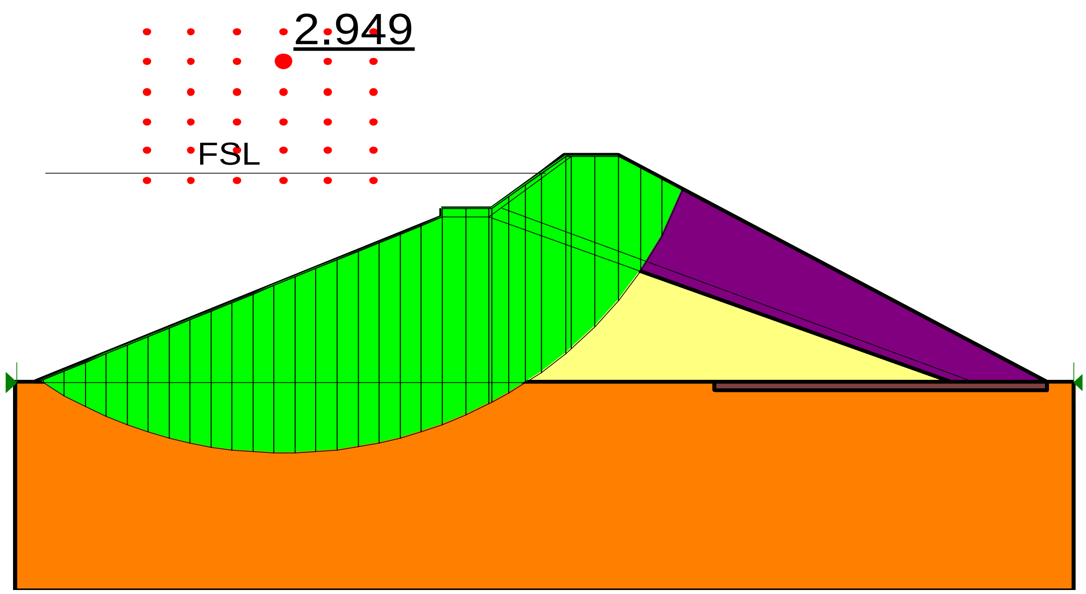

Figure 33: Slope Stability Analysis of asphalt concrete face heightening under Steady State (Upstream Face)

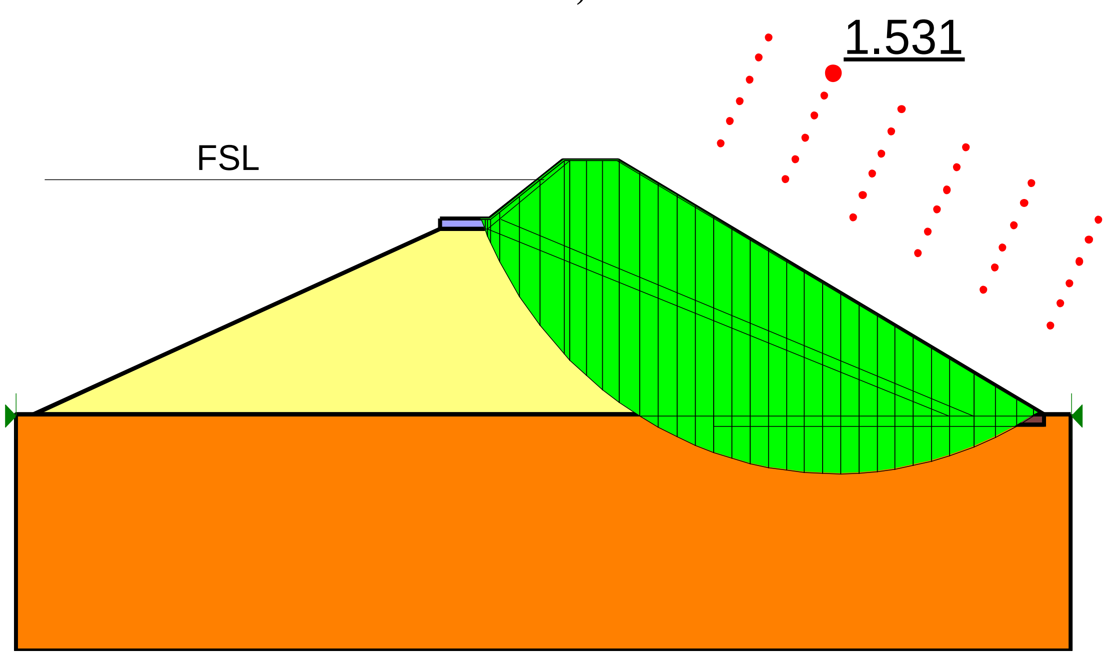

Figure 34: Slope Stability Analysis of asphalt concrete face heightening under Steady State (downstream Face)

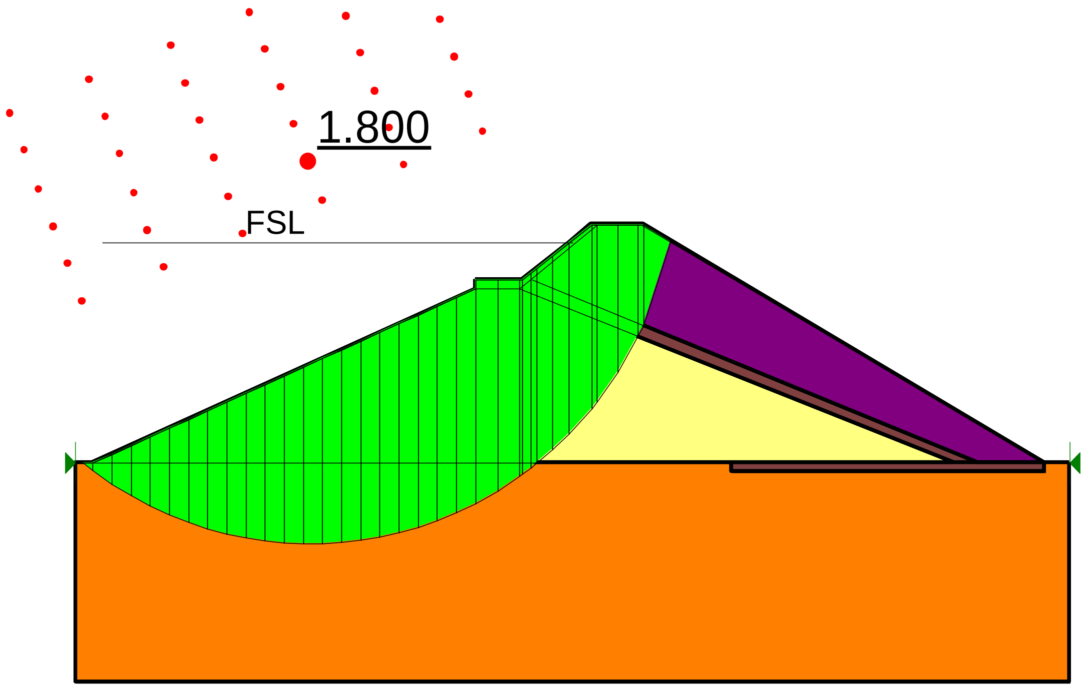

Figure 35: Slope Stability Analysis of concrete face rock fill heightening during rapid drawdown (Upstream Face)

4.3.4 Comparison in between the heightening alternativesin terms of slope stability

As it mentioned in the above section, the overall slope of the embankments (existed and heightened) are safe against failure for all loading conditions. When the heightening alternatives are compared with respect to slope stability, both homogeneously heightened and concrete face rock fill heightened embankment are stable in all loading conditions. Since both heightening alternative become stable with respect to slope, the comparison point of view used in case of seepage analysis may hold true in case of slope stability analysis.

4.4 StaticDeformation Analysis

It is clear that; as the height of the dam increase, its weight is also increase due to the fill material used for heightening. Not only the weight of the dam, but as the height of the dam is increase, the water load in the existing reservoir is also increase due to the formation of additional storage. Here this study was basically focused on conducting the deformation analysis due to the increase in the weight of the dam and thewater load due to the additional storage in the reservoir.

4.4.1 Boundary Condition in Deformation Analysis

4.4.1.1 Stress Boundary Condition

This is a special boundary condition called hydrostatic pressure boundary condition. This boundary condition is used to indicate the change in pressure or force due to the increase or decrease in level of the reservoir. In this case study, the following stress boundary conditions were used while conducting deformation analysis:

A. Reservoir Pressure: this type of stress boundary condition is used to define the pressure of water act on the embankment in case of reservoir full condition.
B. Removing Reservoir with Time: as the level of reservoir is changed (rapid drawdown); definitely there is the change in force which represents the removal of the weight of reservoir. Here this boundary condition was used to defineload variation in case of reservoir drawdown condition.
C. Rapid Drawdown: this is one of the important boundary condition which used to indicate the potential seepage face at the upstream face of the embankment during rapid drawdown condition. Even if the reservoir is drop down from full supply level, some seepage is expected above the drawn reservoir level. Hence; this rapid drawdown boundary condition was defined at the upstream face of the embankment to show the existence of potential seepage face above the level of drawn down reservoir.

4.4.1.2 Displacement Boundary Condition

A. Fixed x/y boundary condition:this boundary condition is used to indicate that there is no horizontal and vertical displacement at all.This type of boundary condition is defined at the lower base of the foundation of the embankment to show horizontal and vertical displacements are not allowed at the lower level of the embankment foundation.
B. Fixed x:it is used to indicate that there is no horizontal displacement. This boundary condition was defined at the right and left edge of the foundation to indicate that the horizontal displacement is not allowed in both edges of the foundation.
C. Fixed y:this boundary condition is basically used to indicate that the vertical displacement in not allowed in the location wherever this boundary condition is defined.

4.4.2 Types of Material Model

In this case studybasically linear-elastic typeof material model was used to carry out deformation analysis through the embankment.If a certain material return to its original shape after the load is removed, then the material has a property of elasticity.In case of linear elastic property, the stress and strain have a direct relationship.Embankment material may face deformation problem and change their shape due to the pressure comes from the reserved water and the weight of the dam. While the water load is removed in case of rapid drawdown, the material in the embankment starts to return their original shapes linearly this is called the linear- elastic property of material.

4.4.3 Stress DistributionBefore and After Heightening

The stress distribution before and after heightening was modeled and the results were taken as contour lines shown in the figure 36, 37, and 38 below. As the modeling shows the maximum stress are registered as 550kpa for existing embankment, 700kpa for homogeneously heightened embankment, and 700kpa for concrete face rock fill heightening alternative.

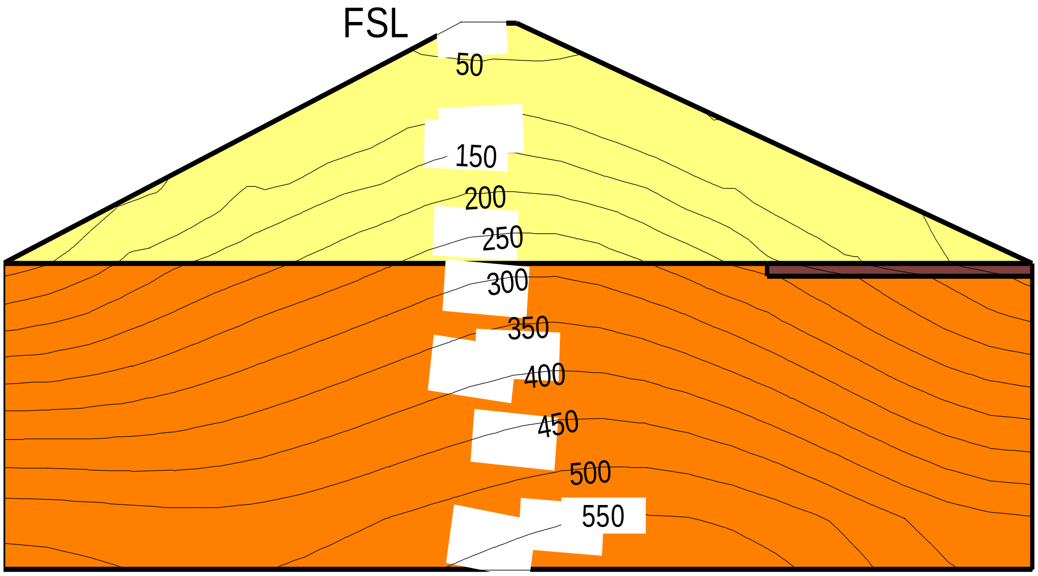

Figure 36: Stress distribution before heightening

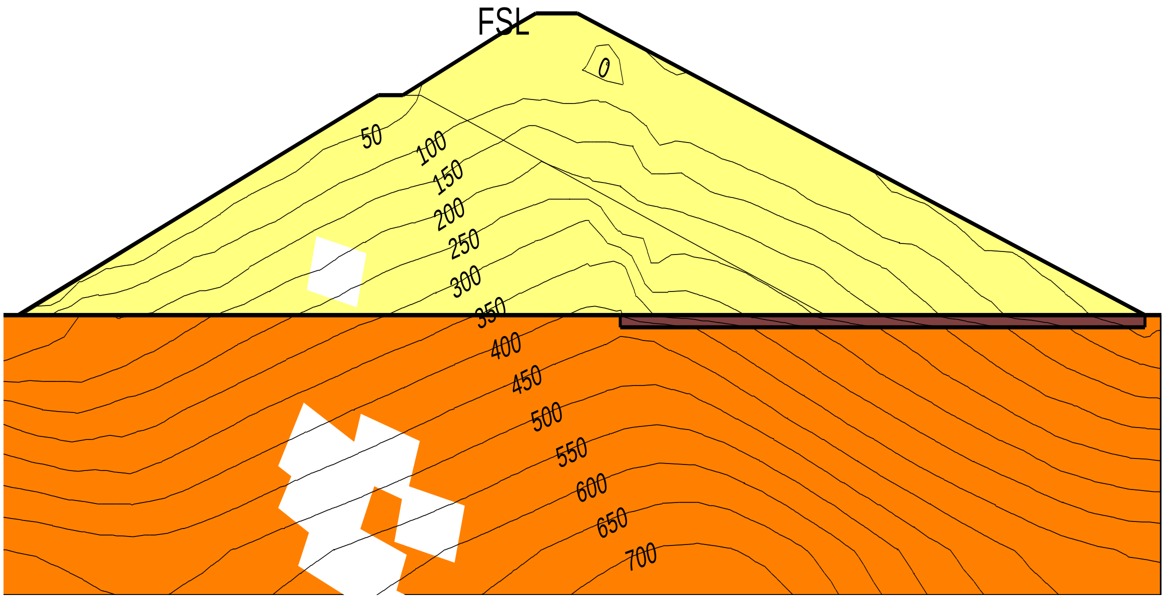

Figure 37: Stress distribution in case of homogeneous heightening alternative

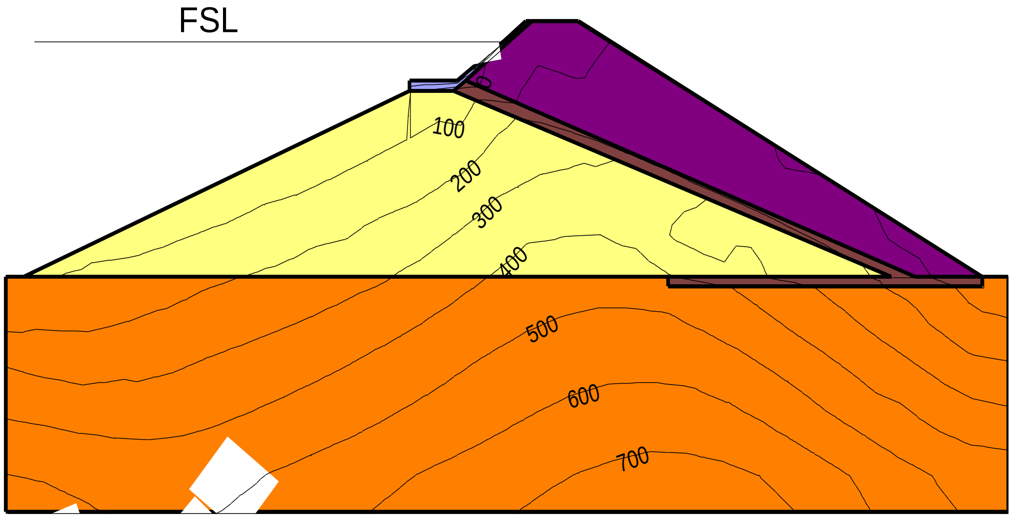

Figure 38: Stress distribution in case of concrete face rock fill heightening alternative

4.4.4 Horizontal and Vertical Displacements

4.4.4.1 Horizontal Displacement

Actually deformation is always caused due to the instability of upstream side slope. The upstream side of the dam is one section of the embankment sensitive for settlement/or displacement. Even if the settlement is started from the upstream side of the embankment, it may lead to the failure of the whole section of the dam.

The horizontal displacement was conducted for the existing embankment and the two heightening alternatives. According to the modeling result, the horizontal displacement is become negligible for existing condition of the dam. Whereas, the maximum horizontal displacement on the crest of the dam was obtained as 0.0275m (27.5mm), and 0.038m (38mm) for homogeneously heightened and concrete face rock fill heightened embankments respectively (see figure 40 and 41*)*.The horizontal displacements of the embankment before and after heightening are presented using contour line as follows.

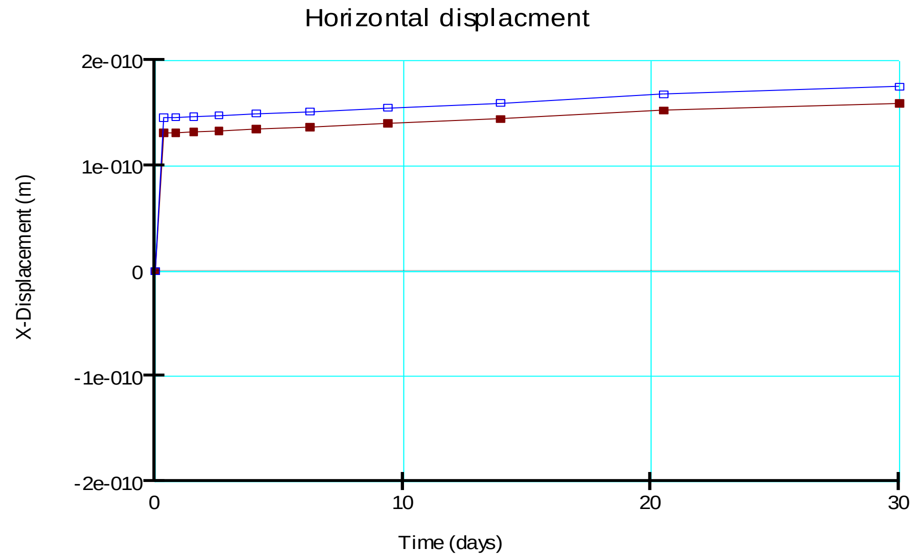

Figure 39: Horizontal displacement before heightening at the crest of existing dam

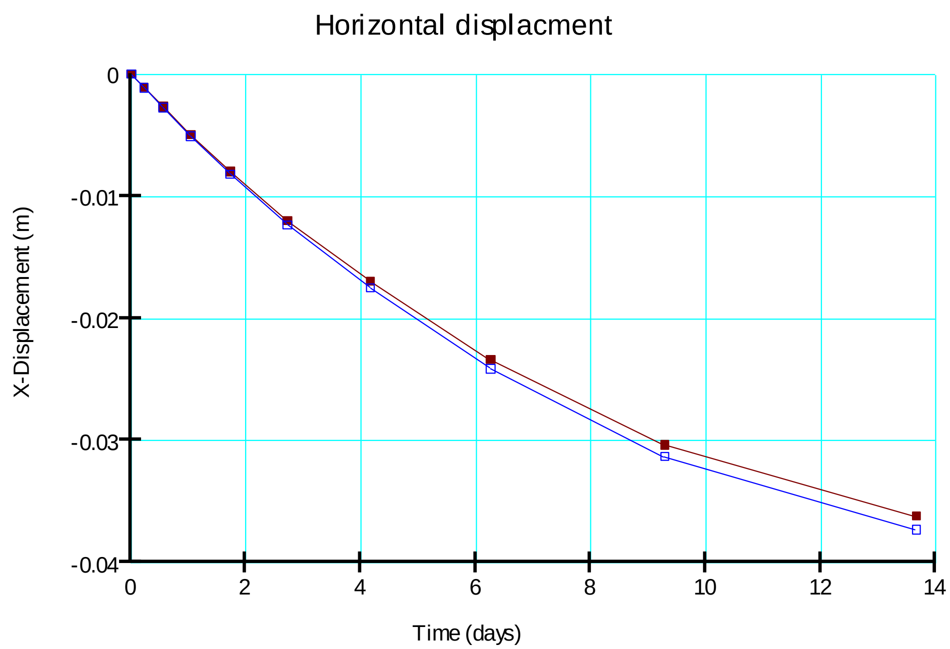

Figure 40: Horizontal displacement in case of homogeneous heightening at the dam crest

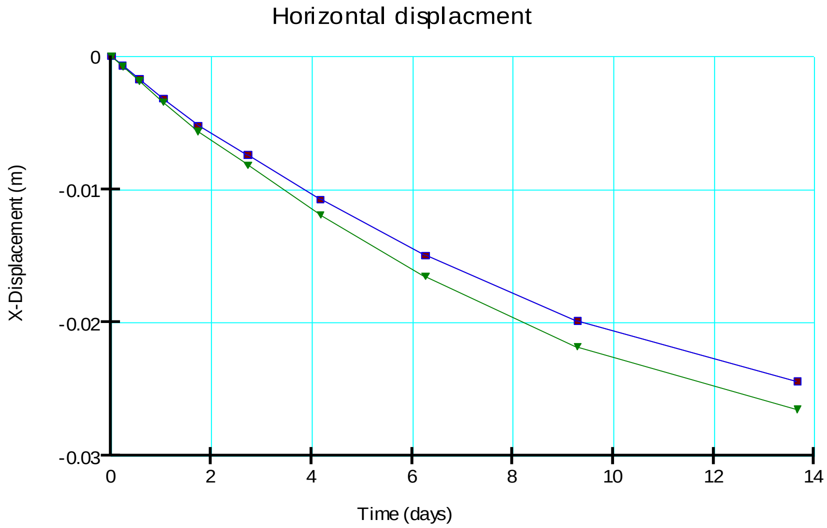

Figure 41: Horizontal displacement in case of concrete face rock fill heightening at the crest of the dam

4.4.4.2 Vertical Displacement

Obviously; one of the major factors which reduce the height of the embankment is excessive vertical displacement.The vertical displacement was modeled for the existed and heightening alternative embankments as shown in the figure 42, 43, and 44. As the horizontal displacement, the vertical displacement also becomes insignificant in case of existing embankment. On the other hand; in case of homogeneously heightened embankment around 0.0028m (2.8mm) settlement was resulted. Whereas, about 0.0122m (12.2mm) vertical displacement was register in case of concrete face rock fill heightening embankment.

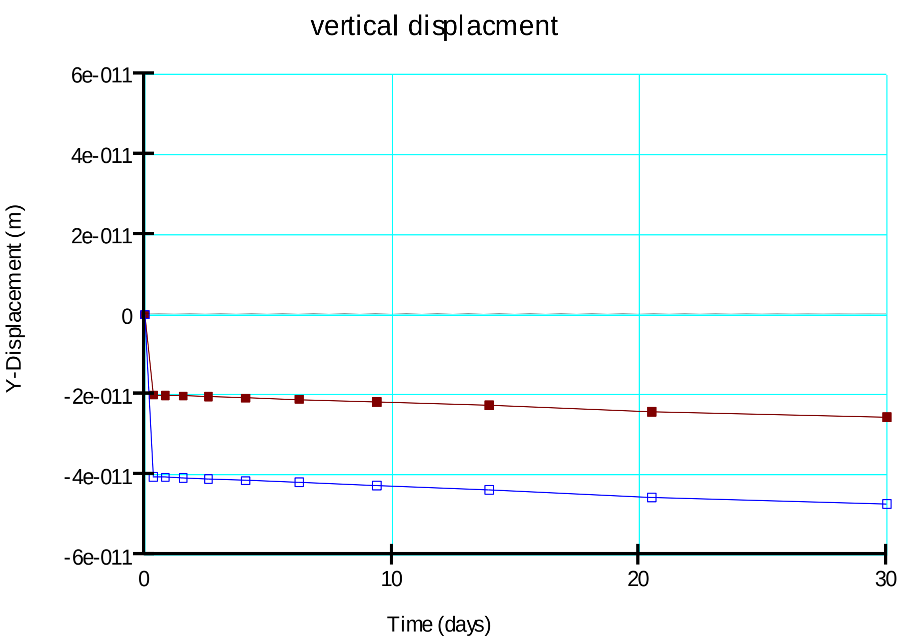

Figure 42: Vertical displacement on the crest before heightening

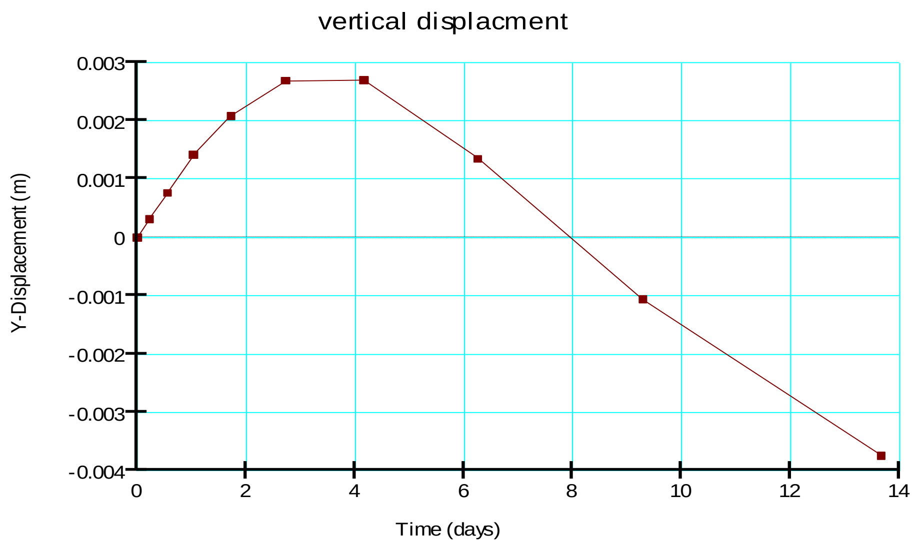

Figure 43: Vertical displacement on the crest of homogeneous heightening alternative

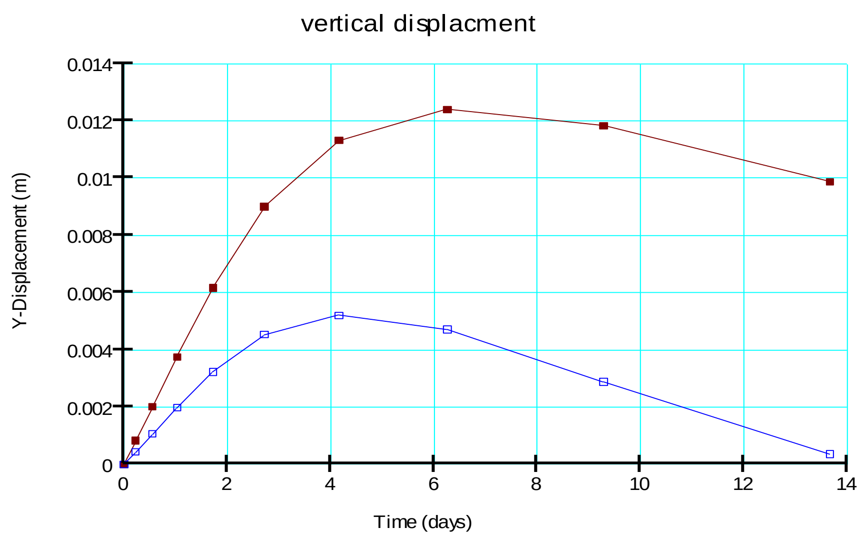

Figure 44: Vertical displacement on the crest of concrete face rock fill heightening alternative

4.4.5Comparison in between the heightening alternativesin terms of deformation

In both alternative heightening, the horizontal and vertical settlements are found within the recommended rang of 50-75mm. this shows that, both homogeneous and concrete face rock fill heightening alternatives were become safe against deformation failure. But, for comparison, greater horizontal and vertical displacement is developed in case of concrete face rock fill heightening than homogeneous heightening alternative. This implies that the homogeneous heightening is become more stable than concrete face rock fill heightening against deformation failure.

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Sediment deposition is a major problem which reduce the live storage capacity of the reservoir in turn the life span of the structure.Gefersa- I/II dam is one of gravity dam found in our country Ethiopia in which sedimentation become a major problem. Even if Gefersa-III embankment dam was builton the upstream of this dam to trap sediment and to reserve live storage, the coming sediment beyond its capacity and reach to the main dam (Gefersa-I/II gravity dam). Due to this problem increasing the height of the structurewas seen as an effective way than dredging the reservoir to increase the storage capacity. Hence it is proposed to heighten the existing dam up to 7m additional height.

This study mainly focused on selecting and designing the best and compatible heightening type for the existing Gefersa-III embankment dam and analyzing the stability of the existing and heightened embankment dam.

This study was considered two different possible heightening alternatives, namely homogeneous heightening and concrete face rock fill heightening.In both heightening alternatives; heightening were started on the crest of the existing embankment and the fill was done on the downstream face of the embankment to keep the operation of the existing reservoir.For both heightening a berm was created at the crest of the existing dam. In addition to berm on the existing dam crest, downstream berm was also provided in case of homogeneously heightened embankment to incease the stablility of the downstream slope of the embankment.

Both heightening alternatives have their own advantage and complexity. Comparativly; homogeneous heightening may minimize construction complixity due to the fact that, during its heightening there is no need of transition material between the existing and new fill material. Whereas in case of concrete face rock fill heightening, providing transition material in between the existed clay material and the new fill material (rock) is a must. Not only providing transition material; incase of concrete face rock fill heightening providing a joint between for the facing to ensure that the it can serve to proof water flow to the body of the embankment. hence creating this bondage may have its own complexity in construction of the heightening section.

Despite of construction complexity, concrete face heightening have its own advantage with respect to minimizing the volume of fill material than homogeneous heightening. This is because rock fill embankment can adopt steep side slope, whereas clay fill embankment require guntle/ flat side slope for their sides. Hence this point of veiw makes concrete face rock fill heightening more effecive than homogeneous heightening.

Since construction complexity or simplicity couldn't become the end point of veiw, the effectivness of heightening alternatives were ckecked with respect to stability in different mode of failure of embankmnet dam. One of the cause of failure of embankment is excessive seep water to the body or foundation of the embankment. actually; incase of embankment dam limited or controlled seepage water is always invited due to the fact that it is impossible for outomatic control of movment of water using naturally available material (soil and rock). Hence steady state seepage analysis was conducted and numerical values were taken for existed and heightening alternatives to check wether seepage is excessive or not. As the result of seep/w shows, the seep water to the body of the embankment were taken as $1.3713*10^{-6}$, $3.055.*10^{-6}$, and $8.255*10^{-8}$m/sec per unit length of existed, homogeneously heightened and concrete face rock fill heightened embankment respectivily.Obviously; the seep water increase as the cross section of the embankment increase and this hold true for this case study. When the seep water of heightening alternatives were compared, it is greater in case of homogeneously heightened embankment than concrete face rock fill heightened embanlment due to the reason that concrete face play great role to drop the phereatic line with high gradient before it reachs to the downstream of the embankment. Hence it is possible to say that concrete face rock fill heightening become more effective than homogeneously heightened embankment with respect to seepage or piping. But as it mentioned above controlled seepage is always invited in case of embankment dams therefore this may not lead to the final decision.

The another major part which covered in modeling section was slope stability analysis for existed and heightening alternatives. As the slope stability analysis section shows, the upstream and downstream slope of existed and heightening alternatives were become safe againest failure in case of both steady state and rapid drawdown loading condition. As any embankment dam designer knows, while guntle side slope is provided for embankment, it is to increase the cross section of the embankment to guide the seep water to the foudation before it reach to the downstream face and to increase the stability of the side slopes. Here even if the side slope of concrete face rock fill heightening is more steeper than the side slope of homogeneously heightened embankment still it become stable for both steady state and rapid drawdown loading conditions. Henec it is cleare that concrete face rock fill heightening become more effective with smaller cross section (cost effective plus stable) than homogeneously heightened embankment.

Deformation is also one of the major mode of failure of embankments. Deformation analysis also conducted for existed and heightening alternatives to check wether the embankments are safe with respect to displacment and stress distribution. As the result of Sigma/w shows the horizontal displacment values were registered as 0.0275m (27.5mm) and 0.038m (38mm) for homogeneously heightened and concrete face heightened embankment respectivily. Whereas the vertical displacments were taken as 0.0028m (2.8mm) and 0.0122m (12.2mm)forhomogeneously heightened and concrete face rock fill heightened embankment respectivily. But, the recommended settlement value is around 50-75mm whch used as standard. Hence both homogeneously heightened and concrete face rock fill heightened embankment become safe with respect to both horizontal and vertical displacments.

Finally; except construction complexity, confidently concrete face rock fill heightening become more effective in most point of views which was considered as comparison bases. Therefore; concrete face rock fill heightening was decided as a best heightening way for the proposed Gefersa-III embankment dam.

5.2 Recommendation

A number of project works were conducted as remedial action to eliminate/ or reduce the problem of sedimentation in the main reservoir behind Gefersa-I/II dam. For instance; in 1954 rehabilitation is done as the first remedial action, but the problem is continued. After a decade Gefersa-III dam is also built at the upstream face of the main dam to trap the sediments before enter into the reservoir, again this action also couldn't be effective solution to solve the existed problem in the reservoir. Right now, for the third time heightening the existing Gefersa-III embankment damis being proposed as asolution to overcome the problem. But it may not be as better solution as the remedial actions which were taken before. Hence better solution has to be identified which may serve as solution for sufficient period of time.

Before construction for heightening is started, the rip rap and top clay soil at the downstream face of the existing dam has to be removed to compact the fill material with existing material.

While concrete face rock fill heightening embankment was designed, most of the fill for heightening is loaded on the downstream face of the dam, and the base of existing dam is shifted to the downstream side by 10.6m. Due to this reason the pipe through the filter of existing dam was covered by the fill material, hence it has to be extended to the allowable length for well-functioning of the pipe.

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