Design and Analysis of a High Dam under Construction Material Constraint Condition

(Case Study: Wolkayite High Dam)

By

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Addis Ababa, Ethiopia

June 19, 2014

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Design and Analysis of A High Dam Under Construction Material Constraint Condition

(Case Study Wolkayite High Dam)

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Degree of Master of Science (M.Sc) in Civil Engineering.

(Major in Hydraulic Engineering)

By

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CERTIFICATION

The undersigned certify that he has read the Thesis entitled **Design and Analysis of a High Dam under Construction Material Constraint Conditions (Case Study: Wolkayite High Dam)** and hereby recommend for acceptance by the Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science.

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26-06-2014

Date

Abstract

In many countries, dams are currently under construction and to due constructed to impound water for different purposes such as flood control, water supply, irrigation, energy generation, and recreation as well as pollution control. Nowadays utilization of selected materials for making the dam body is vital and very common. In cases of high embankment dam projects, which require large volume construction material, availability and adequacy of selected material to the allotted quantity is crucial. Wolkayite high embankment dam project suffers by the presence of highly plastic clay and highly weathered shell material for the construction of its dam body. There is also abundant rockfill material, but require high processing to use resulting high project cost. In this paper, the author sought alternative material for the design and analysis of the dam by assessing different construction materials located within the economic distance of the project site satisfying different design standards in the area with comparable cost. As the dam is above 90% full throughout its design life, the u/s side of the dam was designed as free draining rockfill whereas the d/s side was designed as random fill surrounded by alluvial river gravel fill and rockfill materials. Using recommended design standards: basically USBR, USACE and other recommendations the design and analysis of Wolkayite High Dam was conducted using Geo-studio 2004 and PLAXIS 8.5 software products. Finally, the proposed material type was checked for different design criteria as fill materials making the dam body. Since the results gained after the analyses were in the recommended range, the author recommends this alternative design solution for the Wolkayite high dam project.

KEYWORDS: *Design of embankment dams, Material selection, Wolkayite Dam, seepage, settlement, stability analysis.*

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To My Family

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All praise is due to the Almighty GOD, His Virgin Mother Saint Marry, all His Angels and Saints without whom nothing good of this work can be ever accomplished.

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List of Abbreviations

a.s.l.	Above Sea Level
AAIT	Addis Ababa Institute of Technology
ACERD	Asphalt Core Earth-rockfill Dam
CFRD	Concrete Faced Rockfill Dam
D/S	Down Stream
E	East
FEM	Finite Element Method
FoS	Factor of Safety
g	Gravity
H	Height of Dam
i	Hydraulic Gradient
ICOLD	International Committee of Large Dams
IS	Indian Standard
km	Kilometer
Ltd	Limited
M	Meter
MCM	Mega Cubic Meter
MDE	Maximum Design Earthquake
Mm^3	Mega Cubic Meter
N	North
NFL	Normal Flood Level

No.	Number
OBE	Operating Basis Earthquake
PGA	Peak Ground Acceleration
q	Discharge per unit length
RCC	Roller compacted Concrete
t	Thickness
Tr	Return Period
USACE	United States Army Corps of Engineers
U/S	Upstream
USA	United States of America
USBR	United States Bureau of Reclamation
USSD	United States Society on Dams
USSR	Union of Soviet Socialist Republics
UTM	Universal Transverse Mercator
WWDSE	Water Works Design and Super Vision
α_h	Horizontal Seismic Coefficient
α_v	Vertical Seismic Coefficient

Chapter 1

1.0 Introduction

Because of lack of adequate design knowledge, construction equipment, new materials like cement and concrete and technology of construction, the establishment of large dam is a recent history. In addition, economic conditions and institutional capacity existing in countries that needed large dams did not enable them to take them up. The large dam construction became possible during the 20th century mainly because of advances made in Science and Technology, which enabled mechanization of construction processes and speedier construction (USACE, 2004). Improved design procedures and new construction materials enabled the design of large dams and their components to take on much higher loads and stresses.

Dams are vital elements in modern society and represent large economic values. There are a large number of dams and many of them are high with large reservoirs. Large dams are defined by ICOLD as dams exceeding 15 m in height or, in the case of dams of 10-15m height, satisfying one of certain other criteria, e.g. a storage volume in excess of $1x10^6m^3$ or a flood discharge capacity of over $2000m^3/s$ etc. ICOLD divide dams into embankment dams (about 70% of the total number), concrete dams (about 28 %) and masonry dams (about 2%) (ICOLD, 1988)

In Ethiopia currently, the Government laid a policy of agricultural leading industry, and has seen improvements in this perspective. Though previously the country was mainly involved in small-scale irrigation projects, that do not need sophisticated studies and techniques, recently there are large-scale irrigation projects Tendaho, Omo Gibe, Wolkayite and other projects are under construction.

1.1 Back Ground

Selection of dam type, among others, widely depends on geological condition of the site, foundation condition, economy, construction material availability and others. In view of the available construction materials, and the short dry season, a rockfill embankment with a central earth core is deemed optimum dam type. The most important advantage of proposing an earthen embankment for dam lies in the maximum use of locally available material with considerable low cost (Fell et al., 1992). However, in the case of Wolkayite high dam, there is a highly plastic clay soil and highly weathered shell material around the dam site as it is outlined in the problem statement below. Identifying alternative solutions of construction materials comparing based on technical, economic, construction complexity, project schedule and purpose criterion will be the main finding of this thesis.

1.2 Statement of the Problem

In Wolkayite dam and irrigation development project, the following are identified (Final feasibility study of WWDSE, 2012):

- Highly plastic clay material
- Highly weathered shell material
- Abundant rockfill material, but require high processing to use.
- Foundation conditions containing faults, joints and alluvial deposits.

The aforementioned conditions favor the use of embankment dam type to utilize the locally available material which also cost effective when compared to concrete dam types. Embankment dams are basically two types: Earthfill and rockfill. This thesis will examine which one or hybrid type comprising different zoning is suitable for Wolkayite dam project considering various, like functional, technical, construction, economic and project schedule related factors. Each type may also utilize different sections of impervious zones for seepage control case. In doing this, the project needs suitable design and analysis, assessing all the possible ways of different sections of the embankment making the proposed dam shall most suitable and implementable to the maximum possible extent using the available materials around the site. The design should involve extensive dam stability analysis, seepage modeling and determination of appropriate construction staging taking in to account both dynamic and static loading conditions.

1.3 Objective of the Research

1.3.1 General Objective

The overall goal of this research was to design and analyze a high dam under construction material constraint condition, case of Wolkayite irrigation development project.

1.3.2 Specific Objectives

In this study, the following salient points are going to be address:

- To develop safe designs of different sections of the selected dam for available materials.
- To assess the suitability of the existing materials for selected dam type.
- To analyze different stability, seepage, and various stress situations of the dam under the selected material.
- To assess the economic feasibility of materials making the dam.

1.4 Significance of the Study

Nowadays, construction material shortage problem becomes significant in the design of embankment dams. This may be due to the inaccessibility of suitable construction material in an economic distance of the project site. So incorporating appropriate materials in the design of embankment dams and their analysis will make a designer and analyst to go in depth of this area and solve a problem accordingly by the help of software like Geo-Studio and Plaxis. In this regard, the result and findings of the proposed research work will be expected to have a great importance in economic design of the dam. Based on the recommendation, the constraints of construction material will be solved. Besides, the study may also be utilized by the later researchers intending to work on the same subject or in the same study area. Moreover, this study will be a guideline for designers and professionals of this area in which construction material assessment, design, analysis and supervision of embankment works takes place.

1.5 Project Area Description

1.5.1 Location of the Project

The project area (the Dam axis, reservoir and command area) is located in the UTM Zone 37N geographic coordinates between 1451548 m N - 1562033 m N and 321244 m E – 394095 m E.

The Wolkayite project benefits of the water resources of Zarema River and the dam axis of the project is located downstream confluence of the drainage areas of Zarema and Dukuko rivers. At the Dam site, the catchment area is 2,133 km^2.

The Zarema River, originating from the Semien mountains hill slopes in between Debark and Zarema towns.

The Dukuko River originates more or less at similar altitude as that of the Zarema stream, but on the Western portion of the Semien Mountains. Both the Dukuko and Zarema after the confluence are named Zarema River. The Zarema River is a left bank affluent of the Tekeze River (Final feasibility study of WWDSE, 2012).

1.5.2 Accessibility of the Project

The project can be accessed by air and vehicle, and vehicle only. The best route by vehicle is Addis-Gonder-Humera-Tekeze and MayGeba village and dam site is found at about 1115 km from Addis Abeba .The second access is air and vehicle, air route is up to Axum town then travels by vehicle to Shire-Shiraro and then to MayGeba - Dam Site after traversing 265 km from Axum.

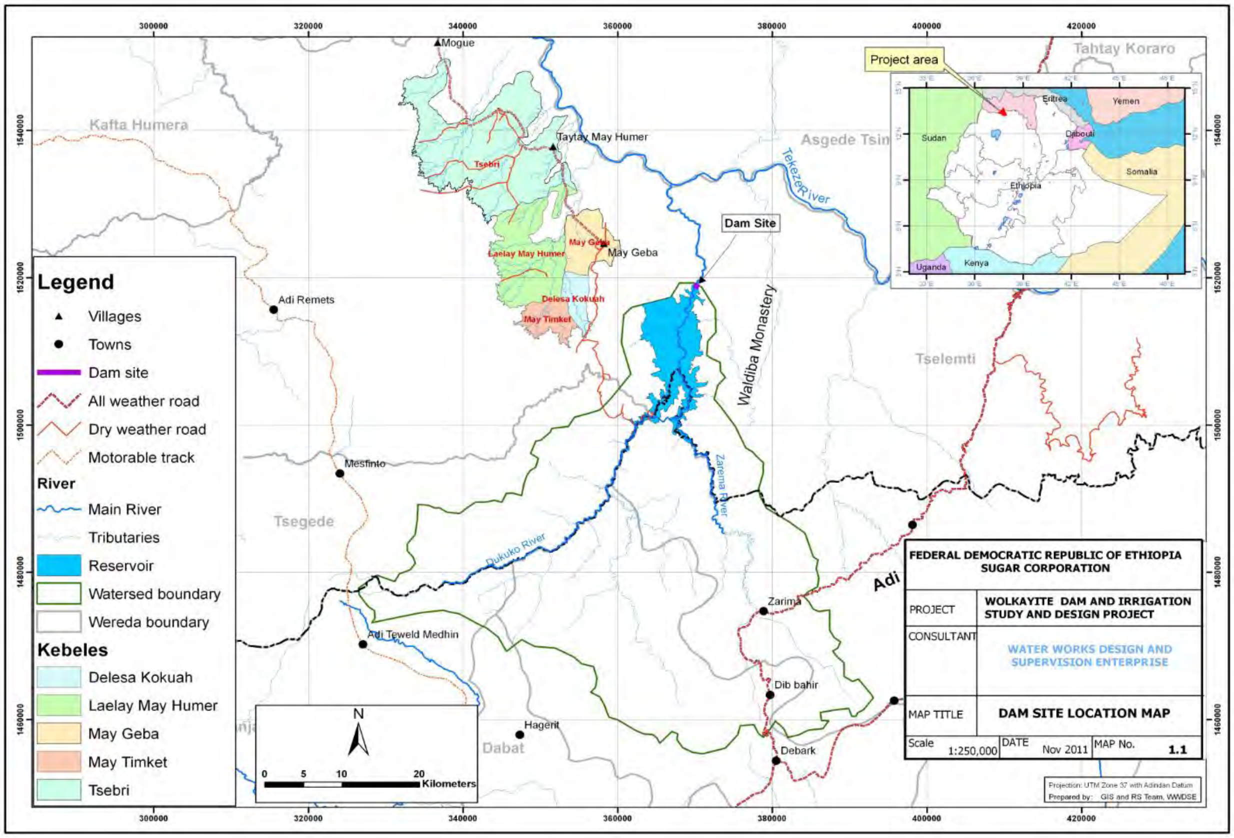

Figure 1: Dam site, Reservoir and Command Area location map

1.6 Geology of Dam Site

The major rock types at the dam foundation area are sedimentary rocks overlying low grade metamorphic rocks. The sedimentary rocks consist of sandstone, mudstone and silt stones. The metamorphic rocks are mainly meta-volcanics, meta-ultra maffics and minor meta-sediments.

Broadly the project area is underlain by low-grade Precambrian basement rocks consisting of basic meta-volcanics, meta-ultramafics, minor metasediments-banded ironstone, marble, calc-silicate, slate; Phanerozoic sedimentary rock successions (including polymictic conglomerates, sandstones, siltstones and mudstones), and Tertiary volcanic rocks consisting of different basalt flows.

Generally, the Dam Site is underlain by Precambrian basement rocks and by sedimentary successions, which are covered by colluvial and alluvial deposits of variable thicknesses.

1.7 Seismicity

Several studies show that the Red Sea and the Gulf of Aden oceanic rifts are propagating on land in Afar as a result of the separation of Arabia from Africa with a rate of about 16 mm/year and the dormancy of the Bab El Mandeb Strait in geologically recent times. The western margin of Ert Ale has been active in recent years. Starting on August 7, 2002, the Mekele area was shaken by a series of earthquakes (Figures 2 & 3) that occurred near the western margin of the Danakil micro plate. The activity continued until the end of the month and the larger earthquakes were widely felt in several towns in northern Ethiopia (Beyeda, Janamora, Adigrat, and Woldia) and Asmara in Eritrea, causing panic in particular in Mekelle town. The magnitude of the main shock was estimated to be 5.6 from (Ayele et al., 2007).

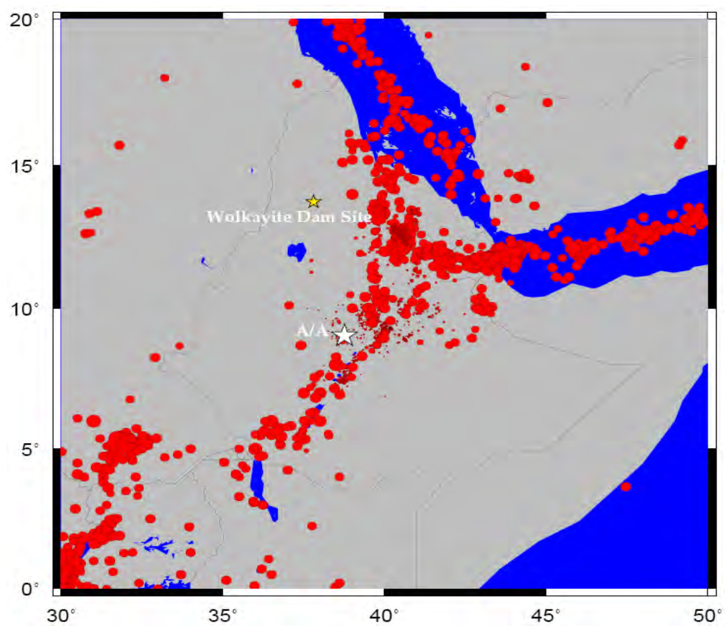

Figure 2: Seismicity data for the Horn of Africa; red circles represent earthquakes that occurred for the last 110 years in the region and size of the circles is proportional with magnitude. The yellow star shows the location of Wolkayite Dam Site and the white star shows the location of Addis Ababa.

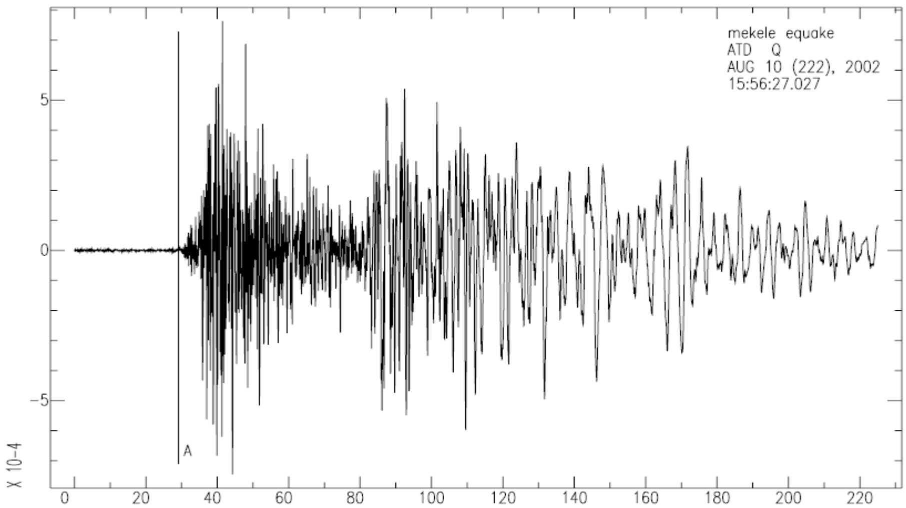

Figure 3: The vertical component weak motion accelerogram of the August 10, 2002 earthquake of magnitude 5.6 that ruptured east of Mekelle which is recorded by a broadband seismic station in Djibouti at 405 km distance from the source.

The shallowness of the depth estimate agrees with the macro seismic reports available from a wide area in northern Ethiopia. Potential future shallow crustal deformation may cause significant loss of human life and damage to property in the densely populated highland region close to the rift margin unless measures are taken in improving building standards of houses, roads, bridges and dams.

1.8 Climate

The project area is characterized by a plain lowland and high rugged mountainous terrain in the highland. The lowlands are hot while the highlands are cold. The average elevation of the woreda lowland has failed about 800masl and the highland is about 2800masl. It could be distinguished by Woina Dega and Kola. Mean annual rainfall is 1100mm with 90% of it falling between June and September. The annual mean temperature is 20 degrees Celsius with little variation across the months.

Chapter 2

2.0 Literature Review

2.1 Dam Type Selection

2.1.1 General

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

A material whose composition is satisfactory for use in embankment construction is a suitable material. The moisture content of the material has no bearing upon such designation. In general, any mineral (inorganic) soil, blasted or broken rock and similar materials of natural or manufactured (i.e. recycled) origin, including mixtures thereof, are considered suitable materials. Consideration should be given to whether the material contains vegetable or organic matter, such as muck, peat, organic silt, topsoil or sod. Certain manufactured deposits of industrial waste, sludge or landfill may be determined to be unsuitable.

Inorganic, fine-grained soils are classified as either clays or silts. Clays and silts typically exhibit different engineering behavior with regard to compressibility, strength and permeability. The principal characteristics that distinguish fine grained soils from coarse grained soils for purposes of embankment dam design are that fine grained soils have lower permeability, lower shear strength, and higher compressibility. Soil plasticity serves as an initial indicator of the potential behavior of a clay or silt when placed in an embankment dam.

The most common index tests, the Atterberg limits, help to classify and characterize fine-grained soils according to the degree to which moisture changes impact soil consistency and behavior. Atterberg limits are water contents at defined transitions in soil consistency, as measured by standardized tests. The liquid limit and the plastic limit are the most commonly used Atterberg limits in engineering work. The liquid limit is the water content at the transition between a liquid and plastic solid state, and the plastic limit is the water content that defines the lower limit of the plastic solid state. The plasticity index (PI = liquid limit - plastic limit) indicates the magnitude of water content range over which the soil remains plastic.

Compressibility of embankments depends on the soil properties and the placement conditions. Under lower consolidation stresses, such as in the upper sections of an embankment, compressibility appears to correspond with placement moisture. Fill placed at relatively low average water contents shows low initial strain and moderately increasing or constant compressibility under higher pressures. Gradation and plasticity of embankment soils are considered to be more important than placement moisture conditions under high consolidation pressures, such as in the lower portions of a high dam (Sherard, et al., 1963).

According to Jansen, 1988, fills placed against small irregularities in rock foundations or concrete surfaces, which must be sufficiently plastic to accommodate differential movement across the discontinuity without cracking. However, this is not suitable utilizing this highly plastic material in large amount for making body of the dam. This is often achieved by selecting materials with high plasticity, and compacting the materials wet of optimum moisture in these critical areas.

Foundations of competent rock with high bearing capacity and resistance to percolation and erosion are required for concrete arch, multiple arches, and buttress dams because these structures are sensitive to foundation deformations. With arch dams, a prime requirement is to accommodate the thrust of the arch onto the abutments. Concrete gravity dams including RCC dams, drive their stability from the weight of the structure and the shearing strength of the foundation. These dams and rockfill dams with upstream impervious facing are built on competent rock and as well on higher quality weak rock, provided sliding stability and a secure subsurface water barrier can be obtained.

Embankment dams, i.e. rockfill or earthfill dams with impervious core, induce lower stresses in the foundation than concrete dams, and can be built over a wide range of foundation materials, on rock and on soil foundations. A well-designed embankment dam makes the best use of local materials to fit site conditions. Quality construction is necessary to transform the design concepts into a successful project.

2.2 Embankment Dams

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

In spite of these developments it is difficult to establish mathematical solutions to the problems of design, and many of its components are still guided by experience or judgment. For a realistic design of an earth dam, it is necessary that the foundation conditions and materials of construction be thoroughly investigated. It is also necessary that controlled methods of construction are used to achieve necessary degree of compaction at predetermined moisture.

The embankment dam is popular because:

- Materials available within short haul distances are used;

- ❖ The suitability of the type to sites in wide valleys and relatively steep sided gorges alike;
- ❖ Subject to satisfying essential design criteria, the embankment design is extremely flexible in its ability to accommodate different fill material e.g. earthfill and/or rockfill. if suitably zoned internally;
- ❖ Properly designed, the embankment can safely accommodate an appreciable degree of settlement deformation without risk of serious cracking and possible failure;
- ❖ The embankment dam can accommodate a variety of foundation conditions, and;
- ❖ Often, the embankment dam is least costly when compared to other dam types.

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

Depending on the available materials at the proposed dam site, embankment dams can be either earthfill or rockfill. An embankment may be categorized as an earthfill dam if compacted soils account for over 50% of the placed volume of material. On the other hand, if over 50% of the fill material is granular then it is classified as rockfill. I.e. coarse grained frictional material, the ideal choice will be rockfill type. In conditions of high dams such as Wolkayite high dam, the dam body must be constructed as heterogeneous fill, having impervious and pervious zones. The impervious zone may be either on the upstream face or within the embankment, in which case it is called core and it may be made of masonry, concrete, geomembrane, steel sheet piles, timber or other material. The shell zone, which is made of granular materials, is provided mainly to support the impervious zone. Present embankment dam design practice preserves both principles. Compacted fine-grained silty or clayey earthfill, or in some instances manufactured materials, e.g. asphalt or concrete, are employed for the impervious core. Subject to their availability, coarser fills of different types ranging up to coarse rockfill are compacted into designated zones within either shoulder, where the characteristics of each can best be deployed within an effective and stable profile.

2.3 Dam Design Alternatives

If suitable earthfill materials are available with in economic haul distance of the selected dam site, it is ideal to be selected for the construction of an embankment dam. If suitable earthfill is not available, then the construction of dam will be made with rock materials with appropriate impervious membrane or concrete. Of course, the latter is not economical as compared to embankment dams due to their huge volume concrete requirement and other foundation requirements.

For a given embankment dam project site, there are several different design options; e.g., a dam with: earth core; upstream facing of reinforced concrete or asphalt or synthetic geomembrane; or asphalt core.

There are different factors that determine the optimum possible dam design alternatives. The following bullets pinpoint the major ones:

- Construction costs,
- Maintenance costs
- Safety,
- Total construction time,
- Construction expertise,
- Potential dam overtopping during construction, and
- Weather conditions.

2.3.1 Concrete Faced Rock-fill Dam, CFRD

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

The main advantage is utilization of less volume of rockfill consequently considerable economy is saved. Some of the disadvantages are design for leakage through opened joints and tension cracks, large compression cracks can occur for considerable height in narrow valleys, needs competent foundation, and cannot provide storage during construction. As in the case of Wolkayite dam site, the foundation condition shows that fresh and very sound rock is located at a maximum depth of 15m; use of upstream faced concrete would require removal of the foundation material from the dam seat area and use of a non-compressible rock fill shell.

2.3.2 Rolled Compacted Concrete Dam, RCC

These types of dams are widely used and becoming common for high dams as is evident in the construction of Gilgel Gibe 3, which has a height of 243 m, is soon to be the highest RCC dam in the world. Such type of dams can accommodate spillways within the dam body, allow overtopping, and safely discharge considerable flow during construction and operation. This would minimize size of diversion structures. Volume of dam is lower making construction time shorter. Compared to conventional concrete dams, RCC dams save considerable cost. The availability of sufficient Pozolana cement locally might be difficult.

Considering the severe shortage of cement (this is now being solved) in the country and non-availability of pozzolanic materials at an economical distance in the area, RCC dams would not be the best option. Another problem with this type of dam is lack of experience by the local contractors.

2.3.3 Geomembrane Faced and/or Core Rockfill Dam, GF/CRD

Geomembranes are typically comprised of polymeric materials that are thin, flexible, and watertight. In general, the main types of geomembranes used as seepage controller in dam construction include poly vinyl chloride (PVC) and high-density polyethylene (HDPE). According to ICOLD, 2011, many places around the world, geomembranes are used as water barrier for temporary or permanent cofferdam and as a liner within main canals to reduce seepage losses. Some of the advantages of using such an impervious membrane: rockfill zone is unsaturated and downstream slope can be constructed with steeper slopes, plinth and grouting can take place independently, and membrane’s flexibility to accommodate rockfill deformations. Whereas the following disadvantages play major role in their selection particularly in high dam; vulnerable to impacts, ice loads, sabotage, effects of weathering and aging, and this require partial or fully covered protective layer that increase cost, and cannot provide storage during construction.

For Wolkayite dam, due to its considerable height use of this geomembrane as water barrior results more problematic conditions in relation to maintenance and sometimes liable to thefts, if it is placed as along the upstream side, by the surrounding community. By the reasons mentioned above, the option to utilize geomembrane as the main water barrior for the main Wolkayite dam construction is discarded. However, may be used for the temporary cofferdam construction as water barrior material in fact later it is also part of the main dam.

2.3.4 Asphalt Core Earth-Rockfill Dam, ACERD

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

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

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

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

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

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

No noticeable leakage was recorded, which means that the core was able to adjust to the significant foundation settlements, differential settlements, the large tension stresses and shear strains without cracking or noticeable increase in permeability.

According to the Indian standard, IS 8826, 1978, the recommended thickness of asphaltic core is about 1% of the head of water at the base. In typical works, the asphalt concrete core wall thickness ranges between 0.5 and 1.0 m wide, depending on the height of the dam, seismicity of the site, foundation conditions, and quality of the embankment fill. Because of the considerable height of Wolkayite dam 1.5 m thickness of asphaltic core is used at its bottom and may be tapered towards to its crest with a minimum core thickness of 0.5m.

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

2.3.5 Wolkayite Dam Alternative Design

Relative merits of four embankment design alternatives were considered. CFRD, GF/CRD and RCC were not practical alternatives for the Wolkayite dam site. ACERD option was selected from the point of view of economy, ease of construction and safety considerations.

The reason why was an ACERD dam design alternative considered?

- ❖ Can be built with lower grade rockfill.
- ❖ Core can be built in rainy cold weather.
- ❖ Core construction does not slow down the rest of the embankment zones.
- ❖ Suitable natural fine grained low permeability material of substantial quantity for construction of a core zone is not available close to site.
- ❖ Reduction in construction schedule.

Chapter 3

3.0 Construction Material

3.1 General

According to USBR, 1986 design standard, nowadays rock of less desirable characteristics has been accepted for use within the embankment section because of the simplified technology with advent of thinner lifts and more efficient compaction techniques. Rounded cobbles and gravels are currently considered the most durable type of rockfill because more angular rock under high stress levels tends to break at the points or fractures causing more settlement and more deformation. The use of rock from excavations for spillways, outlet works, tunnels, and other appurtenant structures has reduced the construction cost of rockfill dams without impairing the embankment's usefulness or stability.

As it is pointed out in the design criteria, for minimum cost the dam must be designed to make maximum utilization of the most economical materials available, including materials which must be excavated for the dam foundation, spillway, outlet works, canals, and other appurtenant structures. In planning the use of these materials, one must recognize that stockpiling, moisture control, processing, and special size requirements may add to the project cost. For these reasons, material from spillway excavations ordinarily is used primarily in the main structural zones (shells) of dam embankments.

The use of materials from the spillway or cutoff trench directly into the embankment without having to stockpile and re-handle requires providing an adequate placing area. The placing area is usually restricted early in the job; hence, the designer is faced with the choice of specifying that spillway excavation be delayed until space is available for it, of requiring extensive stockpiling, or of permitting large quantities of material to be wasted. Zoned dams provide an opportunity to specify the use of materials from structural excavation. The zoning of the embankment should be based on the most economical utilization of materials which can be devised; however, the zoning must be consistent with the requirements for stability and seepage control conditions.

An important use of materials from structural excavation is in portions of the embankment where the permeability and shear strength are not critical and where weight and bulk are the major requirements. Areas within the dam in to which such excavated material is placed are called “random zones.” Because estimates of the percentage of structural excavations usable within the embankment are subject to variation, variable zone boundaries to accommodate any excess or deficiency are sometimes required.

Embankment slopes are generally estimated during the early stage of design on the basis of experience with previous construction materials and foundation and then verified by stability analyses and adjusted as necessary during final design. Flat upstream slopes are sometimes used in order to eliminate or reduce expensive slope protection. A berm is often provided at an elevation about 1.5 m below the inactive capacity water surface elevation to form a base for the upstream slope protection, which does not need to be carried below this point.

The rate of reservoir drawdown is an important factor which affects the stability of the upstream part of the dam, because pore pressures in the upstream zone and foundations of the embankment may not have time to dissipate if the drawdown occurs at a fast enough rate. This may result in failure of the upstream slope. A storage dam subject to rapid drawdown of the reservoir should have an upstream zone of sufficient size and permeability to dissipate pore water pressures during drawdown. If free draining sand and gravel or sound and durable rock are available for construction of the upstream zone, a steeper slope may be used provided that the foundation has adequate strength. If rockfill is used, a transition layer of sand and gravel between the rockfill and the surface of the impervious embankment may be necessary to prevent fine grained material from migrating into the rockfill.

3.1 Construction Material Availability

Suitable materials can be obtained locally from the required excavations and from quarrying within the dam site. These quarry sites can be used to produce good quality rock fill, concrete aggregates and graded granular materials. There is a cohesive material at the project site, but due to its high swelling potential the use of such material will make it difficult to use it as construction material, even though large amount of quantity is available. Therefore, dam types that rely on impermeable cohesive materials have not been considered.

Along the river channels of the Zarema River, there are terraces which are dominated by sand, gravel and pebbles. Major construction materials are pervious embankment (rock fill or gravelly shell material), impervious core material, sand for filter/fine aggregate, and rock quarry for crushed aggregate production as well as rip rap.

A major consideration for selection of dam type is the requirement to minimize importation of materials to the site. Therefore, dam types that require less imported materials have been favored in the screening assessment.

Different types of construction materials that have been assessed for possible use in the Wolkayite high dam are:

- Fine impervious materials;
- Gravely materials;
- Rocky materials;
- Filter, gravel and riprap materials;

3.1.1 Fine Impervious Materials

The impervious material that is present in the dam site and its surrounding, for use as a core, is clay material. Due to its high swelling potential the use of such material will make it difficult to use it as construction material, even though large amount of quantity is available.

3.1.2 Gravely Material

3.1.2.1 Metavolcanic Rocks

a. Weathered Metavlocanic Rocks

Weathered metavolcanic rocks considered as potential sources of pervious gravely shell material. The quarries are located at 0 to 10 km from the dam axis. The volume of shell material to be excavated from this quarry was estimated to be in excess of 16 Mm^3. This material is characterized by its more fine content and may produce more fine content upon compaction if used as construction material.

b. Fresh Metavolcanic Rocks

These fresh rocks have good quality for use as shell materials. However, in order to obtain these materials, the moderately to highly weathered top part (estimated to be in the depth range of 15-25 m perpendicular to the slope face) need to be excavated. Moreover, systematic blasting will be required to obtain the right fragment size from the slightly weathered to fresh metavolcanic rocks.

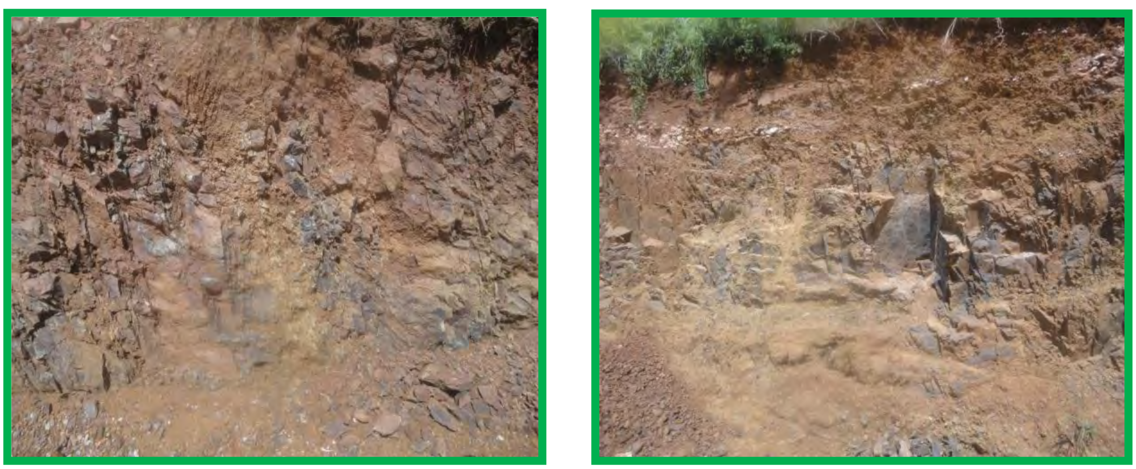

Photo 1: Weathered Metavolcanite as seen from the road cut at the dam axis

3.1.2.2 Basalt Rocks

The catchment, reservoir and outlet areas are underlain by basalt flows. Moreover, most of the ridges found in the project area are igneous bodies (mostly basalts with some dolerite intrusive). Both the weathered and fresh basaltic rocks were evaluated for their suitability as shell materials.

a. Weathered Basalt Rock

The closest basalt ridges are located upstream at an average distance of 5.5 km from the dam site and they composed of highly to moderately weathered basalt. The volume of this quarry site is estimated in excess of 30 Mm^3. This material is also characterized by its more fine content and may produce more fine content upon compaction if used as construction material.

b. Fresh Basalt Rock

Results of the laboratory analysis show that these fresh rocks have good quality to be used as shell materials. However, in order to obtain these materials, to moderately to highly weathered part (15-20 m depth) need to be excavated. Moreover, systematic blasting will be needed to obtain the right fragment size from shell basalt.

Photo 2: Weathered Basalt/Dolerite

3.1.2.3 River Deposited Material

River gravel (alluvial deposit) are often considered among the best construction materials as shell materials for embankment dams because of their free draining and ease mining. In this line, several areas were assessed and identified as potential sources for river gravel/alluvial deposits. Though there are river deposits in many sites in the project area, considering the required volume of free draining shell materials for the dam, areas with good potential zones have been identified. Summary of the location and volume estimate is shown in the table below (Final feasibility study of WWDSE, 2012).


Photo 3: River deposited rounded Gravely Material at the dam axis and upstream of dam

3.1.3 Rocky Material

a. Weathered Sedimentary Rocks

The top part (not exceeding 0.3 m depth) of the sedimentary rocks is highly fragmented due to weathering. Results of the laboratory test revealed that these weathered sedimentary rocks have gravel content which range from 60.35% to 70.19%, sand content from 16.36% to 19.75%, and fine fraction (silt and clay) in the range 13.45-19.91%.

Upon compaction and when subjected to moisture some of the components of these materials (mainly sandstone and conglomerate) could have low resistance to gradation. In addition to the quality problem, the weathered part is only the top 0.30 m and this makes the naturally occurring fragmented material to have limited volume.

b. Fresh Sedimentary Rocks

The fresh sedimentary rocks were also assessed for their suitability as construction material as well as for their geotechnical properties. Samples were collected from boreholes cores and surface exposure on fresh rocks. Results revealed that these materials have good quality in most parameters except for the abrasion resistant which gave marginal results for Los Angeles Abrasion value.

The main challenge with the quality of fresh sedimentary rocks as shell materials is the fact that these rocks are comprised of alternating layers of sandstone, siltstone/mudstone and conglomerate. These conglomerates are weakly cemented and the sedimentary rocks are not homogeneous; this makes it difficult to process. Moreover, to break the rock systematic and controlled blasting will be required to obtain the required fragment size.

Photo 4: Sedimentary Rocks

3.1.4 Rock for Aggregate, Filters and Riprap

The possible sources of materials for quarry (crushed aggregate) are intrusive rocks (dolerites) at a distance of 10kms from the dam site. As riprap, the sedimentary rocks are identified as potential sources.

Chapter 4

4.0 Asphalt Core Earth-Rockfill Dam Design

4.1 Introduction

The basic consideration in the design of Dams in general is to achieve safety consistent with economy. Consideration must be given to maintenance requirements so that economies achieved in the initial cost of construction will not result in excessive maintenance costs. For minimum cost, the dam must be designed for maximum utilization of the most economical materials available, including materials, which must be excavated for its foundation and for appurtenant structures. Based on this ground, and assessing different design standards basically United States Army Corps of Engineers, USACE, Design Standards Embankment Dams No. 13, USBR, International Commission of Large Dams, ICOLD, and Fell, Geotechnical engineering of dams, the selected type of dam is earth-rockfill dam with central asphalt core that gives rise to the lesser cost, lesser time of construction as well as best use of rock and shell materials.

4.2 Design Criteria:

An earth-rockfill dam must be safe and stable during all phases of construction and operation of the reservoir ICOLD, 2011.

To accomplish this, the following design criteria must be satisfied:

- The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet capacity;
- The slopes of the embankment must be stable during construction and all conditions of operation, including rapid drawdown of the reservoir;
- Seepage flow through the embankment foundation and abutment must be controlled, so that no internal erosion takes place and so there is no sloughing in the area where the seepage emerges. The amount of water lost through seepage must be controlled so that is does not interfere with planned project functions;
- The embankment must be safe against overtopping by wave action;

- The upstream slope must be protected against erosion by wave action, and crest and the downstream slope must be protected against erosion due to wind, rain and cattle.

As more than 90% of the reservoir water head acts on the dam and foundation areas throughout the life of the dam, seepage analysis and control measures shall receive the maximum attention. No major loss or damage to the seepage control measures shall be allowed since remedial measures are difficult to be undertaken in this dam due to the long time it takes for depleting and filling the reservoir.

4.3 Dam Zoning and Foundation

4.3.1 Dam Zoning

In most cases, high embankment dams are zoned to use as much material as possible from required excavation, from borrow areas with the shortest haul distances and the least wastage, at the same time maintain stability, and control seepage. For most effective control of through seepage and seepage during reservoir drawdown, the permeability should progressively increase from the core out toward each slope (USACE, 1986). For the fact of utilizing local material, the Wolkayite dam formed as a zoned embankment with an impervious central "Core" composed of Asphaltic Concrete. A central core has the advantage of providing higher pressure at the contact between the core and the foundation, thus, reducing the possibility of leakage and piping.

A zoned dam is, however, preferred where different types of soils are available from borrow area. It also facilitates the use of compulsory excavation from different works of the dam. In zoned dam, the weaker materials are often utilized most economically in the form of random zone. Random zones are generally provided below minimum drawdown level on upstream side and on downstream. When the random zone is of relatively impervious material, horizontal filter at different elevations on upstream and downstream sides are provided (Fell et al., 2005; IS 8826, 1978). But for this special case of Wolkayite dam since the reservoir is almost 90% full throughout its life, due to the position of its command area, upstream side of the fill must be free draining to overcome liquefaction problem and reduce uplift pressure. Downstream side of the fill is designed on the bases of optimization to utilise most economical fill materials of the shell as random fill surrounded by alluvial gravel and rock fill which are quite good in their free draining nature.

4.3.1.1 Upstream and Downstream Slopes

The side slopes of the dam is determined by Stability Analysis Program, Geostudio 2004, accounting for each of the different construction materials and considering all design criteria conditions. The side slopes depend upon various factors such as the type and nature of the dam, foundation materials, height of dam, economic requirement, etc. Embankment slopes used for rockfill dams have evolved from very steep slopes, usually 0.5 to 0.75: 1 (H:V) which were used on early rockfill dams, to the flatter slopes of 1.3:1 to 2.0:1 used in current practice (USBR, No.13, 1986). The recommended values of side slopes as given by Terzaghi (Terzaghi, 1943) are given below.

Table 1: Recommendations of Side Slopes of Embankment Dams

Type of material	Upstream slope (H:V)	Downstream 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 central clay core	3:1	2.5:1
Sand or sand and gravel with R.C diaphragm	2.5:1	2:1
Rock fill	1.6 to 2:1	1.6 to 2:1

Based on the recommended values different trial slopes were taken and final side slope comes after checking the stability and economic consideration as 1.5H:1V upstream slope and 1.46H:1V downstream slope with 5 downstream berms.

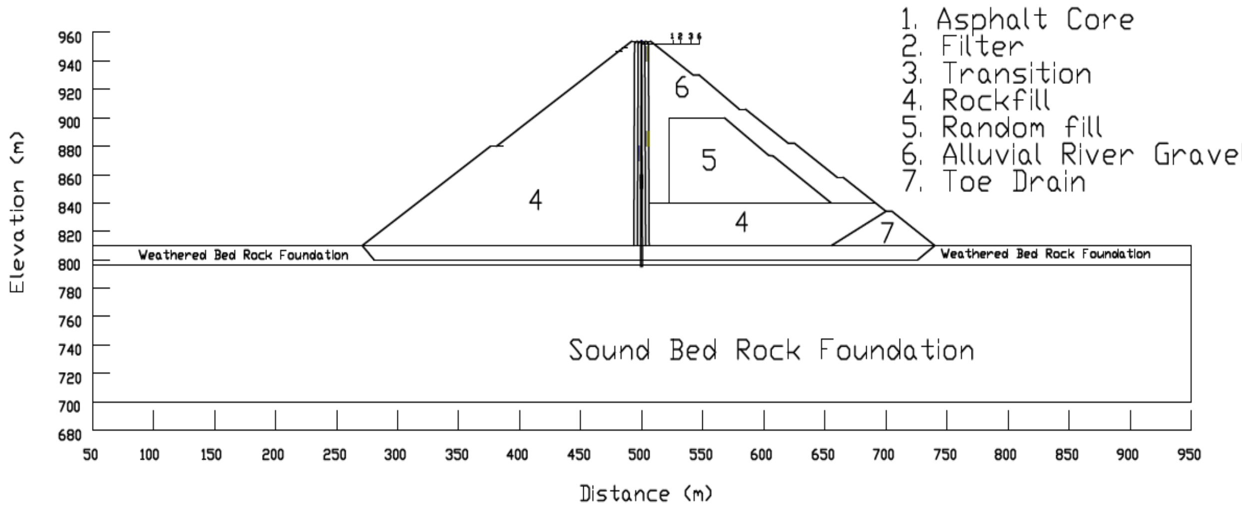

Figure 4: Dam profile section-Model

4.3.1.2 Provision of Berms

For the safety of the main dam against slope erosion and for later operation and maintenance cases, provision of berms is necessary. Therefore, as Wolkayite dam has considerable height and in order to break the height at convenient levels for later operation and maintenance of the dam, a total of 5 berms; 4 of them at 30 m and 1 at 25 m vertical interval are provided on the downstream slope. All berms are 5 m wide and will be inclined at 2% slope towards the fill. Moreover an 8m wide berm is provided at elevation 880 m a.s.l on the upstream slope for a reason of as a road service during the construction of the main dam on the crust of coffer dam and stability of the upstream slope.

4.3.1.3 Camber

Major portion of earth fill dam consolidation takes place during the construction period. Therefore, camber shall be provided along the crest of an embankment dam to ensure that the free board provided is not reduced due to foundation settlement or consolidation of the embankment. Therefore, camber shall be provided along the axis of the dam from zero height at the abutments to maximum at the central section of the dam. Generally, a provision of 1% and 2% of the embankment height above the designed top level may be provided to account the settlement in respect of rockfill and earthfill dams respectively (USBR, 1987). For Wolkayite Dam, a camber of 1% i.e. 1.54m has been provided at the maximum section for settlement purpose.

4.3.2 Foundation Condition

The foundation area of Wolkayite dam is overlain by colluvial and alluvial deposits. Underneath these deposits the highly weathered meta volcanite is encountered about 14m deep in the abutment areas. Fresh sound rock is encountered in the river bed area below the alluvial deposits and the weathered meta-volcanites in the abutment area. The permeability of upper section is in the range from impervious to pervious whereas, the bedrock is impermeable and sound.

For the boreholes drilled over the general area of the dam site the following general considerations can be expressed about the permeability of the foundation:

- ❖ in 70% of the cases values lower than 2 L units, indicating essential imperviousness of the formation, were registered;
- ❖ in the remaining 30% of the cases the values fall in the 3-11 L range (medium permeability), with the exception of a value of 51 L recorded in borehole WDBH-2 (located in the lower portion of the left abutment) in the interval 32-37 m.

4.3.3 Provision of Cutoff

The main importance of provision of cutoff in a rockfill dam is the prevention of seepage beneath the dam and the effecting of a watertight seal between the impervious core and the foundation. To prevent seepage beneath the dam, foundations are usually grouted. The need for grouting and the extent required should be based on careful study of the site geology, on a visual examination of the drill cores from the rock foundation, and on drill-hole water-loss values (USBR, 1987).

Cutoff is required for the following functions:

- ❖ To reduce loss of stored water through foundation and abutments; and
- ❖ To prevent subsurface erosion by piping.
- ❖ Provide sufficient working space for compaction equipment as required ;and
- ❖ Provide sufficient working space for curtain grouting;

Considered criteria to fix the cutoff excavation level are:

- ❖ At least one meter depth shall be anchored to the impervious hard continuous stratum.
- ❖ Side slopes of 2H: 1V is the minimum adopted for this project.

4.3.4 Grouting of Foundation

Generally, foundation grouting comprises consolidation/blanket grouting and curtain grouting. Depths for a grout curtain should be sufficient to minimize seepage and assist in the reduction of uplift and the need for extensive drainage facilities. Where conditions permit, grout holes should bottom in sound, relatively impervious rock. In general, it may range from 30 to 40 percent of the head of the water on good foundation and to 70 percent of head on poor foundations. Consolidation grouting is introduced by drilled shallow holes with different patterns, depending on the type of the dam and the geological conditions and it is usually restricted to the upper 5m to 20 m (Fell et al., 2005).

As reported in the Draft Feasibility Design and confirmed during the Site inspection carried out from 14 to 16 June 2012, most of RQD values found in the geological cross section of the river indicates fair to very poor rock quality up to 25 m maximum depth throughout the dam axis. Deep curtain grouting must be carried out down to a maximum depth equal to 50-60% of the maximum hydrostatic head, to control and minimize seepage through the foundation.

For this particular dam site, good rock quality is found up to a maximum depth of 25 m. Curtain grouting is considered assuming maximum depth of which 50% of maximum hydrostatic head (0.5 x 136 m) of maximum head up to the level of 728 m.a.s.l.

Chapter 5

5.0 Material and Methodology

5.1 Materials Used

Geostudio 2004 was used to carry out the modeling, including seepage analysis, stability analysis and deformation analysis under static and dynamic conditions. PLAXIS 8.5 was also used for the analysis of stability and deformation of the dam. For the computation of volumes of the respective materials was done by using AutoCAD 2012.

5.2 Methodology

‘’ The attraction of modeling is that it combines the subtlety of human judgment with the power of the digital computer.’’ Anderson and Woessner, 1992. The computer program Geo-Slope International Ltd., 2004 was used to estimate seepage, stability and settlement conditions through transverse sections of the dam. Deformational analysis, with determination of the displacements and stress distribution throughout the dam were carried out by FEM modeling, under static and seismic conditions. Stability analysis of the dam was also carried out by Slices Method analysis, under static and seismic conditions.

Another numerical modeling conducted in this study was the finite element code developed by PLAXIS BV, which is developed for two-dimensional analysis of deformations and stability. Version 8.5 has been used in the numerical modeling.

5.3 Data Availability

Before conducting of any research, it is imperative to make a tough search for the data. Therefore, the primary assignment of the study is getting relevant information and data of the study area. For this thesis, geological and hydrological data are taken from Ministry of Water and Energy and Water Works Design and Supervision Enterprise and some others adopted from typical works of Duncan et al., 1980.

5.4 Constitutive Model for Dam Materials

The Duncan-Chang Hyperbolic model is widely used in the numerical analyses of embankment dams for the modeling of soil behavior, and is capable of modeling the non-linear, stress-dependent and inelastic behavior of cohesive and cohesion less soils. Whereas the original model assumes a constant Poisson's ratio (Duncan and Chang, 1970), the revised model accommodates the variation of Poisson's ratio by means of stress-dependent bulk modulus (E-B model, Duncan et al. 1980). In Geostudio 2004, due to basaltic bedrock, the foundation is modeled as linear elastic material with Young's modulus and Poisson ratio. However, fill materials are modeled as nonlinear elastic model. The parameters used for analysis was taken from WWDSE, 2012 final feasibility study and Duncan et al., 1980 Report. These parameters were used in a finite element numerical model to evaluate the seepage, stability and settlement of Wolkayite dam.

5.5 Model Simulation and Analysis

The material properties used for different analysis are given in table 2, as shown below. Using Geostudio 2004, the foundation was modeled with a linear elastic model specified using values of Young's Modulus and Poisson's Ratio. Also the dam body was modeled with a nonlinear elastic model. In Plaxis 8.5, both the embankment and foundation was modeled with Elastoplast behavior Mohr Coulomb Model respectively.

5.5.1 Analysis Limitations

Due to the complex nature of soil behavior, all observed behaviors of an embankment dam might not necessarily lend themselves to a precise analysis. In such instances, reliance on engineering judgment, based on professional experience of responsible engineers, is generally considered prudent and acceptable.

Table 2: Parameters of Material Properties

Material Type	γ_d (KN/m^3)	C' (KN/m^2)	ϕ ' (deg.)	k (m/s)	E (KN/m^2)	ν	k_L	n	R_f	k_b	m	References
Lower Bed Foundation	26	10500	35	$5.0x10^{-7}$	$2.0x10^6$	0.15	-	-	-	-	-	*1*
Upper Bed Foundation	18	3000	34	$1.0x10^{-6}$	$2.0X10^5$	0.3	-	-	-	-	-	*1, 2*
Asphalt Core	24	800	35	$5.0x10^{-10}$	$2.6x10^5$	0.45	360	0	0.7	-	-	*1, 3*
Filter	21	0	40	$1.0x10^{-3}$	$4.0x10^4$	0.3	900	0.4	0.7	-	-	*1, 4*
Transition	21	0	40	$2.5x10^{-3}$	$4.0x10^4$	0.3	1000	0.42	0.7	-	-	*1*
Alluvial Gravel	19	0	51	$1.0x10^{-3}$	$5.0x10^4$	0.3	1700	0.39	0.67	1300	0.16	*4, 5*
Crushed Basalt	19.7	0	51	$1.0x10^{-1}$	$5.0x10^7$	0.15	1000	0.22	0.7	390	0.14	*2, 5*
Random Fill	19	4.7	35	$1.1x10^{-4}$	$5.0x10^4$	0.28	400	0.5	0.7	600	0	*2, 5*

Where:

γ_d = Dry Unit Weight, C' = Cohesion, ϕ' = Internal Friction angle,
k = Permeability, E = Young's Modulus,
ν = Poison's Ratio, k_L= Modulus Number,
n = Modulus Exponent, R_f = Failure Ratio;
k_b = Bulk Modulus Number, m = Bulk Modulus Exponent

Note: References

1. WWDSE, 2012
2. Gorden, 1978
3. Hoeg, 1993
4. Nikoloviski et. Al, 2011
5. Duncan et al., 1980;

5.5.2 Seepage Analysis

One of the basic requirements for design of embankment dams is to ensure safety against internal erosion, piping and development of excessive pore pressures.

Steady state seepage analysis is carried out to estimate the amount of seepage through the embankments and foundation materials. The analysis is conducted using the state of the art Finite Element Method based computer program – Seep/W from Geo-slope international, 2004.

The permeability coefficients of materials for different zones listed in table 2 above are the ones under saturation condition. When seepage flow calculation, in order to simulate the seepage characteristics of material under unsaturation condition and above saturation line the materials other than the foundation should be given a permeability coefficients of 1/100 of the coefficients under saturation condition in case of a pressure head less than zero.

5.5.3 Slope Stability Analysis

The stability of an embankment depends on the characteristics of the foundation and fill materials, on the geometry of the embankment section, and additional factors such as presence of water, loading conditions etc.

The procedures for static slope stability are well established, and have been concisely documented by Duncan et al, 1987. Duncan recommends that evaluation of a slope focus on defining geometry, shear strengths, unit weights, and pore water pressures. The various methods available for slope stability analysis yield similar results (within ± 6%), and selection of the method is less important than choosing accurate parameters.

Seismic slope stability is evaluated using a "pseudo static" analysis where the failure mass is assumed to be horizontally accelerated by the seismic coefficient, α_h as percentage of gravity, appropriately chosen for the expected seismicity of the site.

The stability of Wolkayite Dam is analysed using both state of the art software – Slope/W from Geo-Slope International Ltd of Canada and Plaxis version 8.5. The stability analyses had been conducted in order to determine the factor of safety for various slip surfaces:

- Upstream and downstream slopes under steady state seepage condition with or without earthquake;
- Upstream slope under sudden drawdown condition; and
- Upstream and downstream slopes under during construction and end of construction condition.

5.5.3.1 Loading Conditions

As the reservoir is 90% full, the upstream reservoir condition is not critical. Table 3 below summarizes the loading conditions and corresponding minimum factor of safety, FoS requirements proposed by USACE, 2003.

Table 3: Various Load Cases and Minimum Required Factor of Safety for Full Supply Condition.

Case	Loading Condition	Critical Slope	FoS
I	End of construction	Upstream	1.3
		Downstream	1.3
II	Sudden drawdown	Upstream	1.3
III	Steady state seepage	Upstream	1.5
		Downstream	1.5
IV	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

5.5.3.2 Method of Slope Stability

The slope stability investigation will be carried out using the Slope/W computer program based on the limit equilibrium method and the Morgenstern-Price method will be used to obtain the factors of safety. This particular method is to be adopted because, unlike Fellenius or Bishop's or Janbu's methods, the Morgenstern-Price method satisfies both the force and moment equilibrium conditions. Spencer's method also satisfies both moment and force equilibriums and gives factors of safety values very close to those obtained by the Morgenstern-Price method.

5.5.4 Deformation Analysis

5.5.4.1Types of Stress Analyses

In general, there are two types of stress analyses that are used in the evaluation of existing and proposed embankments. These are the total stress analysis and the effective stress analysis. The total stress analysis is used in the design of embankments for loading conditions during construction, rapid drawdown, and earthquake. The effective stress analysis should be used only in cases where the soils behave drained and piezometer data are available. The cases that can be analyzed by the effective stress method are partial pool and steady seepage.

5.5.5 Design of Earthquake

The criteria adopted for the selection of design earthquake loads were based on the recommendations of USACE, 1995.

- ❖ Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated.
- ❖ Operating basis earthquake (OBE). The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of exceedence during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years.) The associated performance requirement is that the project functions with little or no damage, and without interruption of function.

According to Feasibility Report, it appears that an earthquake may be expected in the dam zone with peak ground accelerations (PGA).

Table 4: PGA Values as per Available Feasibility Report

Design Earthquake	PGA (g)	
	Horizontal, α_h	Vertical, α_v
OBE (T_r 144 yrs.)	0.08	0.027
MDE (T_r 1,000 yrs.)	0.11	0.037

Where g = gravity acceleration, Tr return period or average recurrence interval, OBE operating basis earthquake and MDE maximum design earthquake.

The project area is confirmed to be of relatively limited seismicity, mainly because of its relatively far location from the active margin of the Ethiopian Rift. The dam must be safe for the operating basis earthquake and must be check with the maximum design earthquake value for its adequacy.

5.5.5.1 Pseudo-Static Stability Analysis

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

According to Seed, 1979, the acceptable design criterion for a rockfill embankment dam exposed to earthquakes, is a pseudo-static factor of safety greater than 1.15 for an earthquake coefficient of 0.1 for a magnitude 6.5 event, and an earthquake coefficient of 0.15 for a magnitude of 8.25 event.

The results are presented in Figure 18 and Figure 19. Respectively this satisfies the recommended design standard.

5.6 Liquefaction Potential

The phenomenon of liquefaction of loose saturated sands, gravels, or silts having a contractive structure may occur when such materials are subjected to shear deformation with high pore water pressures developing, resulting in a loss of resistance to deformation.

According to Youd T.L, 1998, soils may be considered to pose no potential liquefaction hazard, if it can be demonstrated that any potentially liquefiable soil types present at a site:

a. are currently unsaturated (e.g., are above the water table),
b. have not previously been saturated (e.g., are above the historic high water table), and ,
c. cannot reasonably be expected to become saturated,

Additionally, if the presence of potentially liquefiable soil types cannot be reduced, and if it cannot be shown that such soils are not and will not become saturated, then the absence of significant liquefaction hazard may still be demonstrated.
Table 5 shown below summarizes historical data relating water table depth to liquefaction susceptibility (Youd T.L, 1998).

Table 5: Groundwater Table Depth and Liquefaction Susceptibility (Youd T.L, 1998).

Groundwater Table Depth (m)	Relative Liquefaction Susceptibility
<3	Very High
3 to 6	High
6 to 10	Moderate
10 to 15	Low
> 15	Very Low

The presence of the waterproofing asphalt core guarantees that the downstream side of the embankment is dry and the location of the random fill is far beyond the phreatic level. Hence, earthquake shaking cannot cause pore water pressure buildup and strength degradation. Hence, liquefaction is not a problem.

Chapter 6

6.0 Results and Discussion

6.1 Results of Seepage Analysis

The Finite Element Models used in the analyses and the computed discharges are shown on respective Figures. As can be seen from SEEP/W results, phreatic level vertically drops down indicating that the use of asphalt core as a water barrior material is quite effective. It is one of the tasks of design and construction to make the structure functional in the sense that the water is properly drained away and the quantity of drained water is tolerable and small. The seepage flow was calculated based on the seepage flux through the embankments, and multiplied by the average crest length of the dam. The expected seepage for the embankment was estimated to be 0.029 l/s per meter length of the dam.

The pressure distribution was also resulted as expected with minimum at the top and maximum at the bottom along with the depth of water increment. In the graphs of velocity and hydraulic gradient, the distributions of their values displayed centrally as expected.

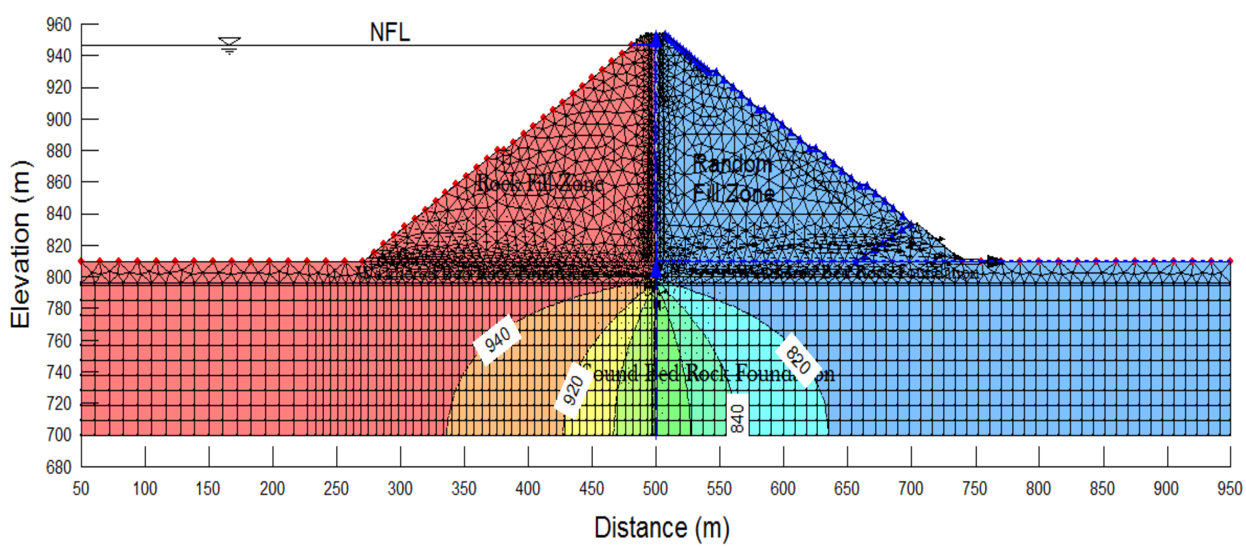

Figure 5: Total head distribution and location of phreatic line.

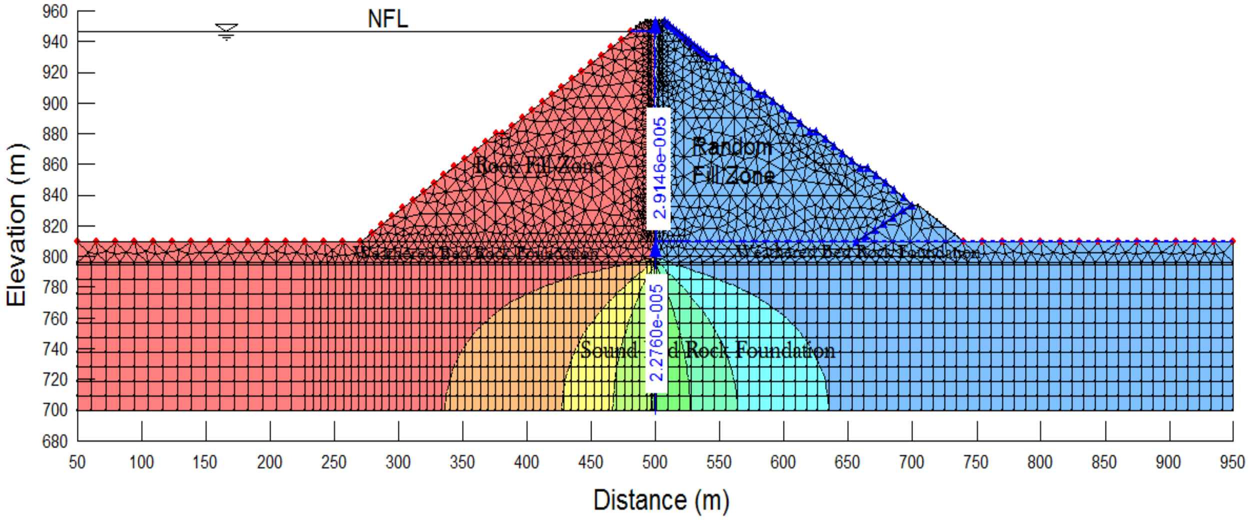

Figure 6: Seepage through the dam body and foundation.

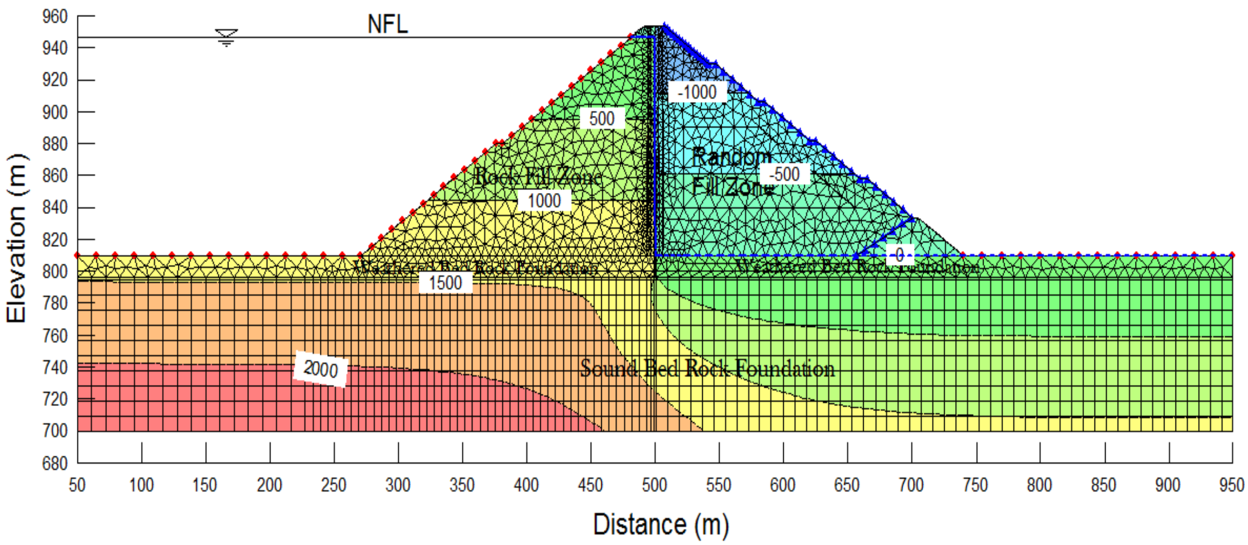

Figure 7: Pressure difference through the dam body and foundation.

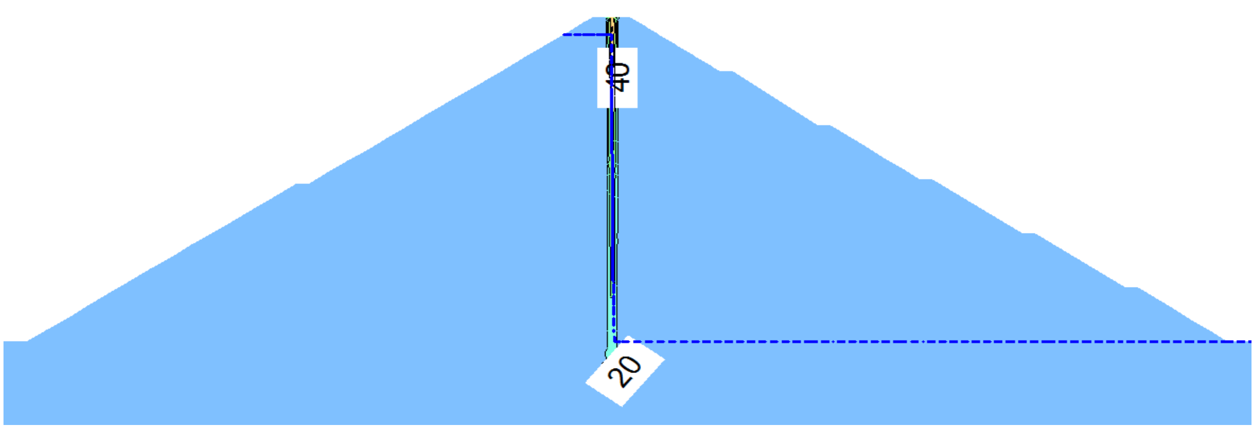

Figure 8: X-gradient through the dam body and foundation.

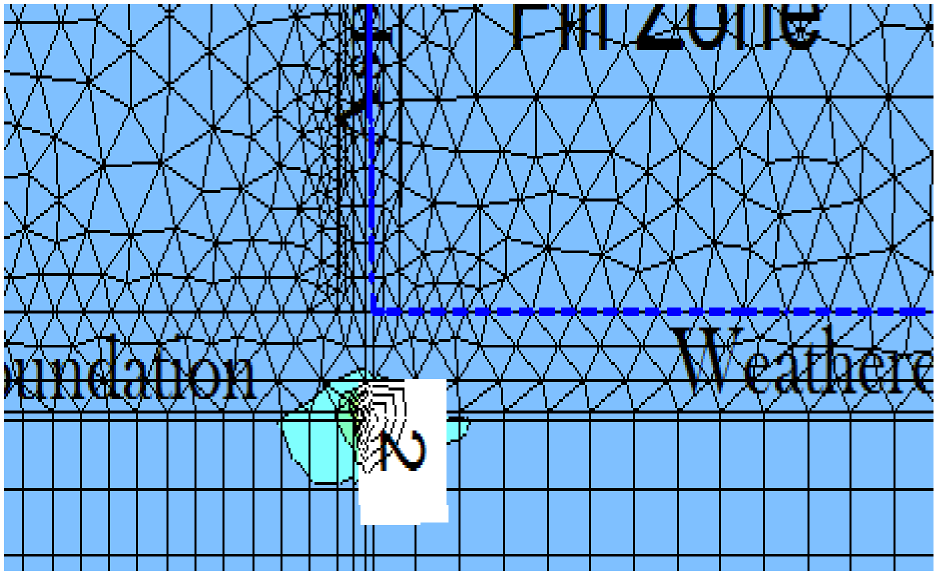

Figure 9: Y-gradient through the dam body and foundation.

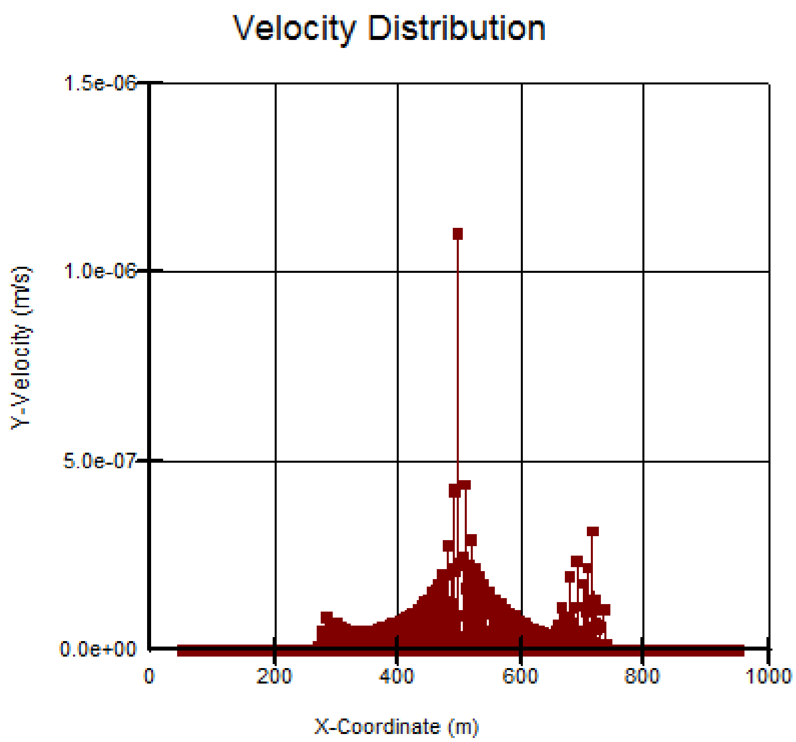

Figure 10: Velocity Distribution Y-Velocity vs. X-Coordinate in the dam body and foundation.

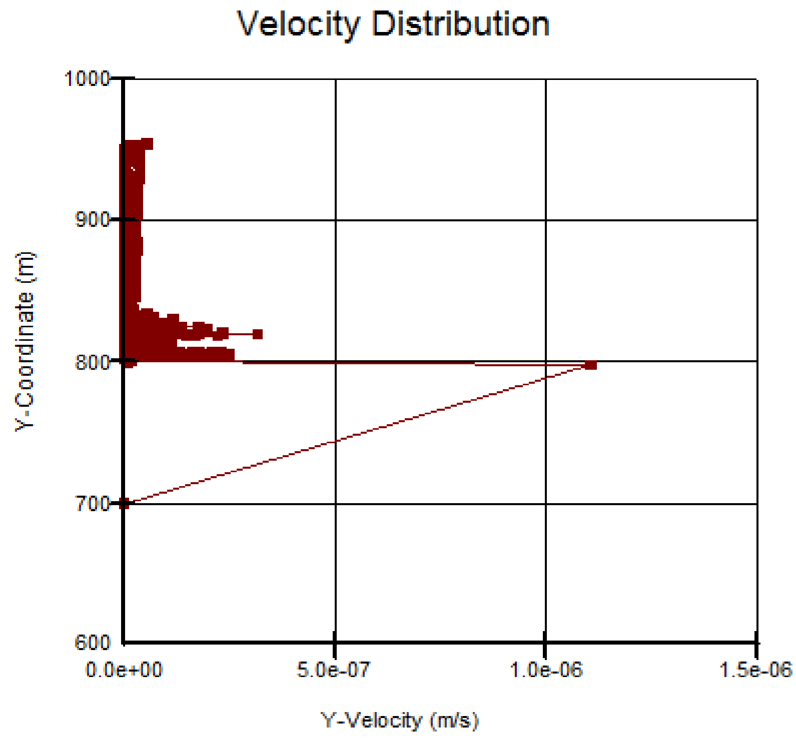

Figure 11: Velocity Distribution Y-Coordinate vs. Y-Velocity in the dam body and foundation.

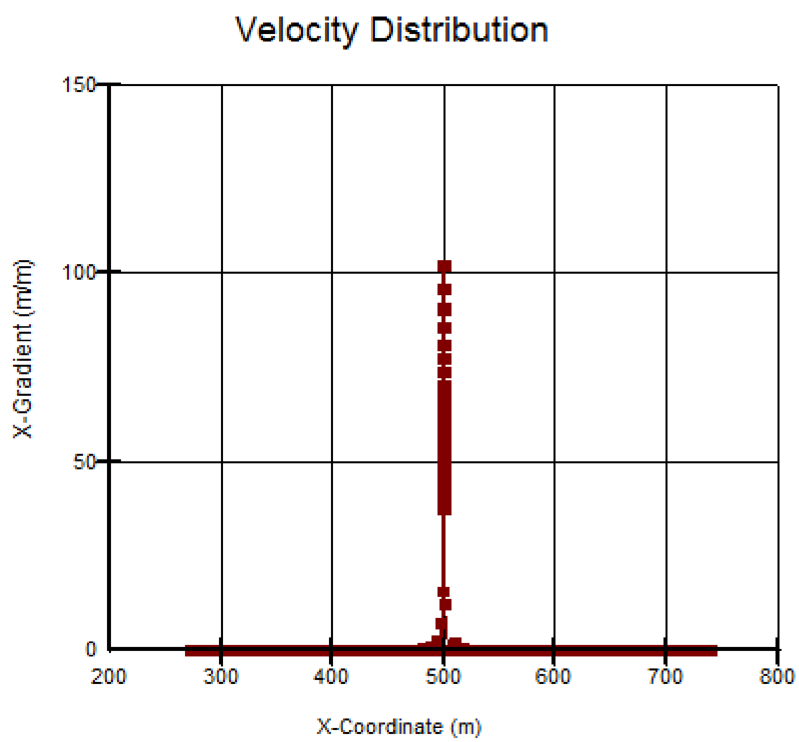

Figure 12: Hydraulic Gradient Distribution X-Gradient vs. X-Coordinate in the dam body and foundation.

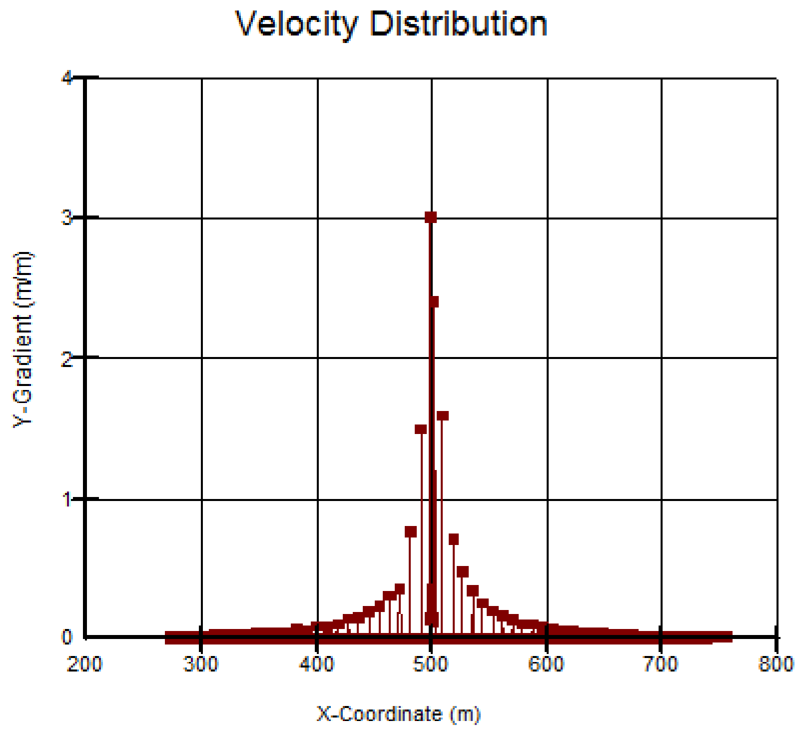

Figure 13: Hydraulic Gradient Distribution X-Gradient vs. X-Coordinate in the dam body and foundation.

6.2 Results of Stability Analysis

6.2.1 Using Geo-Studio 2004

The computed factors of safety against slope failures under the different loading conditions are summarized in table 6 below.

Table 6: Computed Factors of Safety

Loading Condition	$FoS_{min.}$	Computed Factors of Safety		Status
		upstream	Downstream	
End of construction	1.3	2.240	1.714	Ok.
Steady state seepage	1.5	2.293	1.716	Ok.
Steady state seepage with earthquake	1.1	1.188	1.388	Ok.

The computed critical slip surfaces with their corresponding factors of safety are shown in Figures below. Slope stability is controlled by the shear strengths of the dam materials and the foundation (Coulomb, 1776). According to Kutzner, 1997 with a homogeneous dam on an equivalent foundation the slope itself is the critical slip plane, provided the shear strength is given by friction only. In contrast, the critical plane cuts through the dam if the shear strength is given by friction and cohesion. With a zoned dam with earth core on a foundation of equal or higher shear strength, the critical slip plane touches the foundation line. In any cases if the foundation is stiffer than the embankment the slip plane may not touch the foundation line depending of the rigidity of the foundation.

As upstream dam body is subjected to varying stress and strain conditions due to reservoir operation, the use of different materials there is limited. Earthquake demands free draining materials all over the upstream shell at a permeability of k ≥ 10^{-2} m/s (Kutzner, 1997). Therefore, the upstream shell material is selected as rockfill. As depicted in figure 21, the variations in pore water pressure at different condition strengthens the use of free draining material to alleviate the problems caused by this pore water pressure development.

Embankment stability during earthquake loading has been assessed by performing a pseudo-static analysis, whereby a horizontal force (seismic coefficient) is applied to the embankment to simulate earthquake loading to determine the critical (yield) acceleration required to reduce the factor of safety to 1.0.

From the results of the performed analysis, it can be concluded that the dam satisfies all the requirements of USBR recommendations as shown in table 6 above.

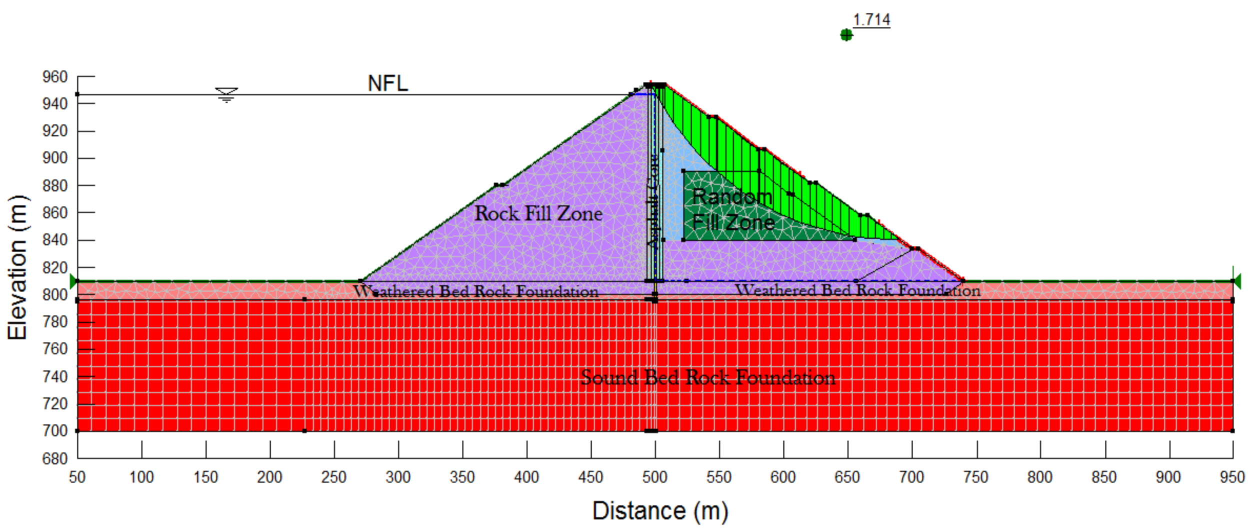

Figure 14: End of construction condition (downstream slope).

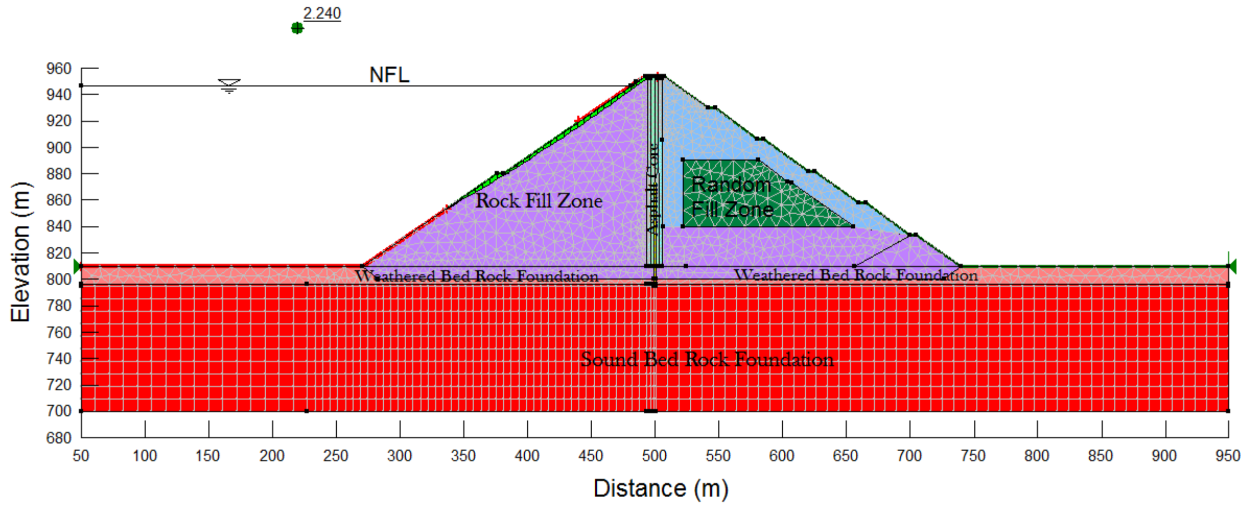

Figure 15: End of construction condition (upstream slope).

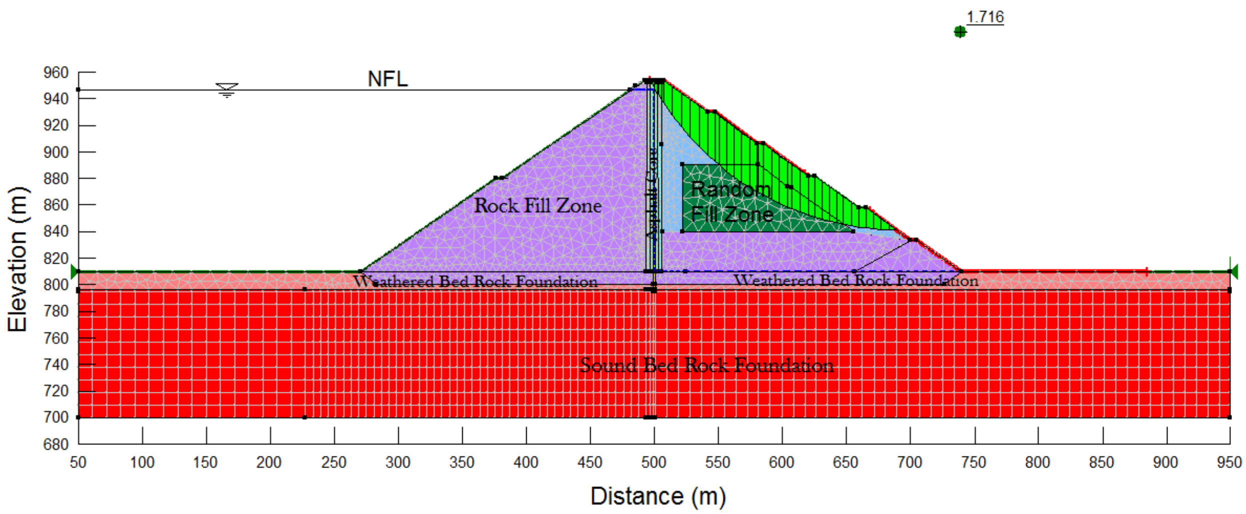

Figure 16: Steady state seepage without earthquake (downstream slope).

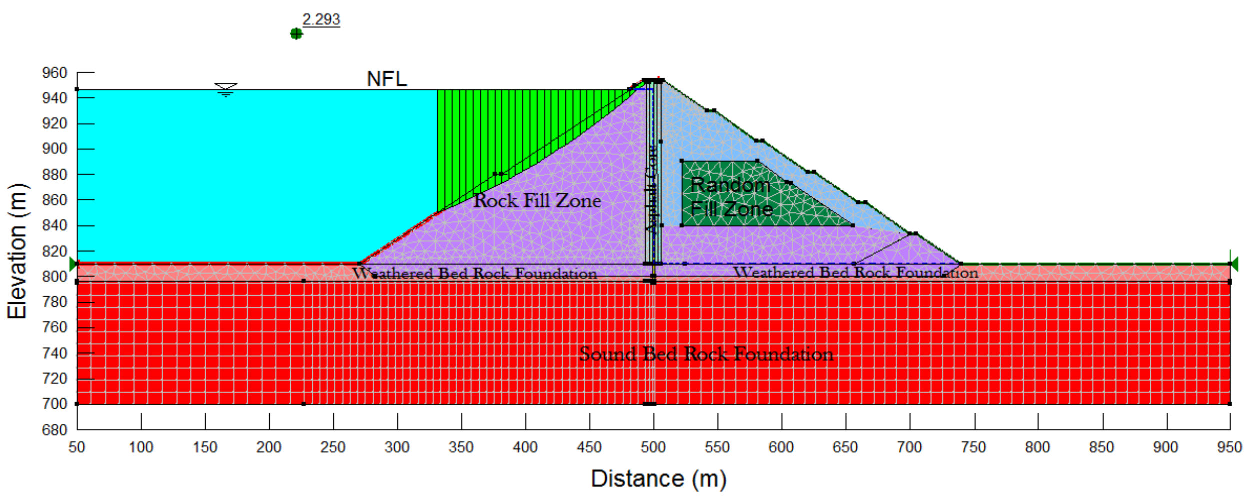

Figure 17: Steady state seepage without earthquake (upstream slope).

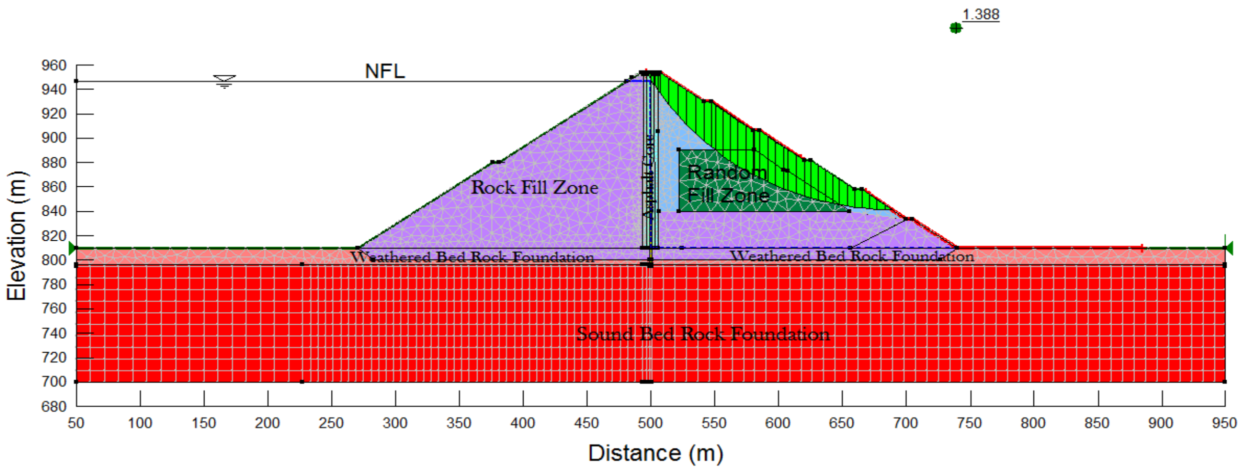

Figure 18: Steady state seepage with earthquake (downstream slope).

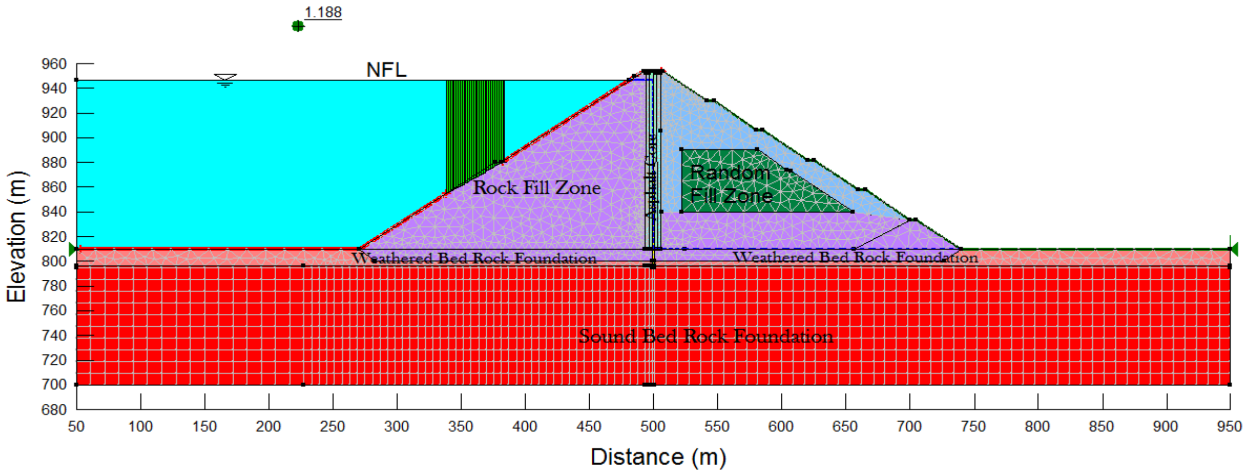

Figure 19: Steady state seepage with earthquake (upstream slope).

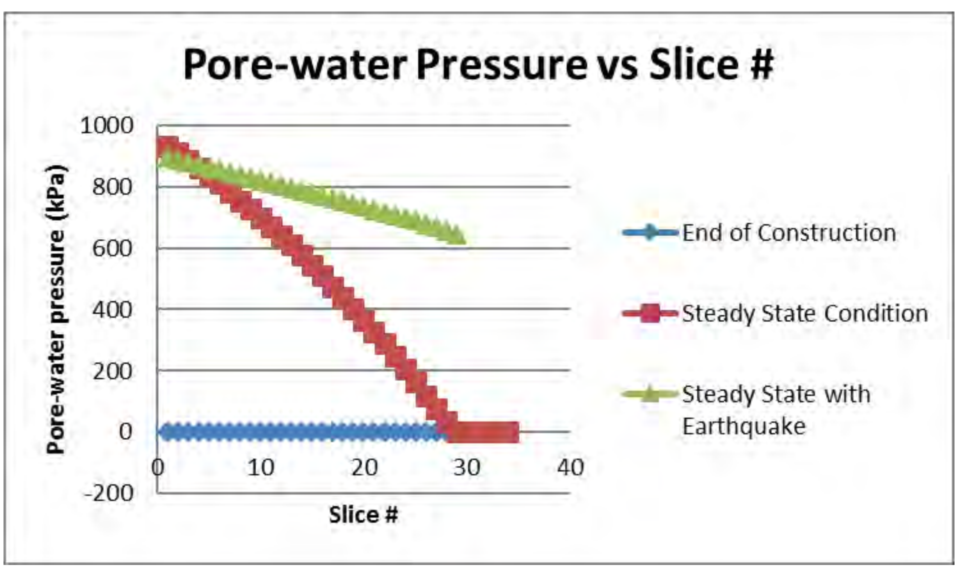

Figure 20: Pore water pressure distribution (upstream slope).

6.2.2 Using Plaxis V8.5

In the design of an embankment it is important to consider not only the final stability, but also the stability during the construction. For the purpose of the deformational and stability analyses, the construction staging (15 lifts of 10m depth) was selected. This type of construction staging implies the preliminary execution of diversion works and grouting works and the construction of cofferdam and the toe drain. After the cofferdam is completed, the construction of the remaining portion of the dam body is characterized by the execution of horizontal layers at elevation z, spreading from the cofferdam downstream slope to the final dam downstream slope, to be completed before stepping to the next layer at elevation z + Δz complete lift. This applies to the possibility to easily place and use haul roads within the fill and, in principle, to stage the construction of the upstream portion of the embankment ahead of the downstream portion.

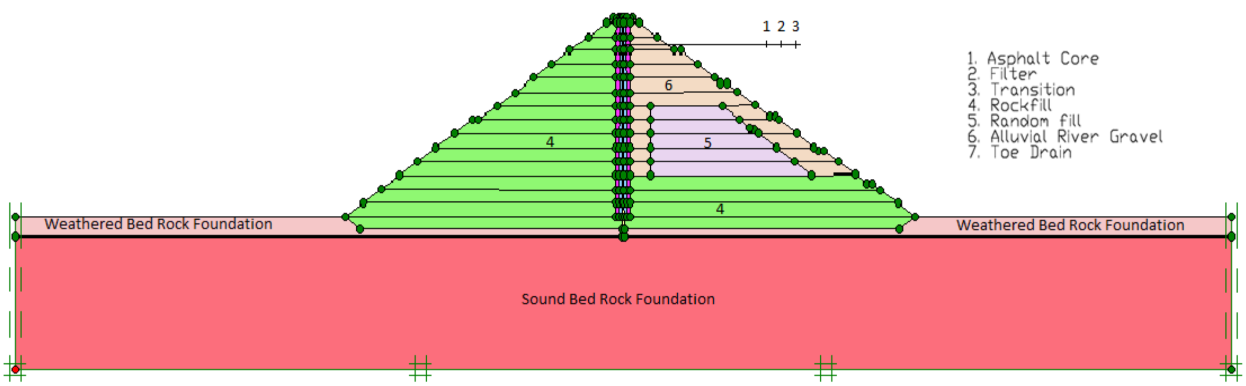

Figure 21: Complete lift Stage Construction.

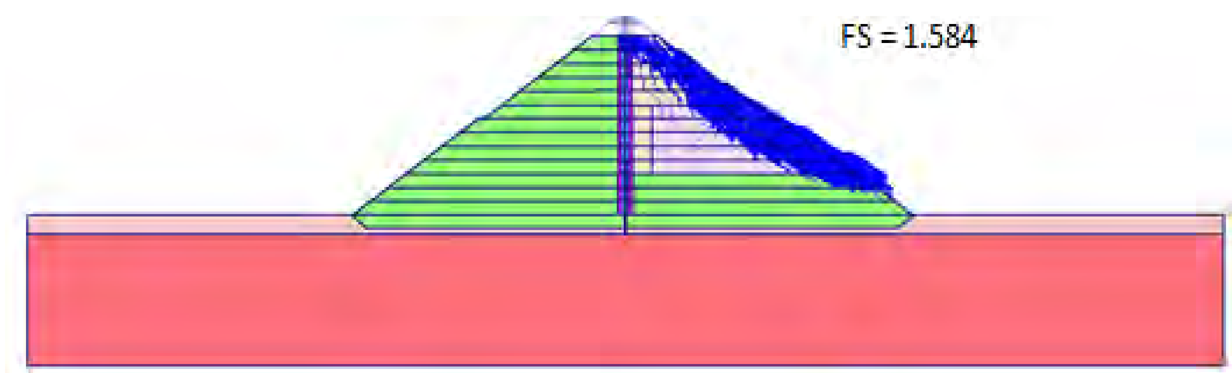

Figure 22: Plot of total increments (arrows) - (Phase: 32, Intermediate phase)

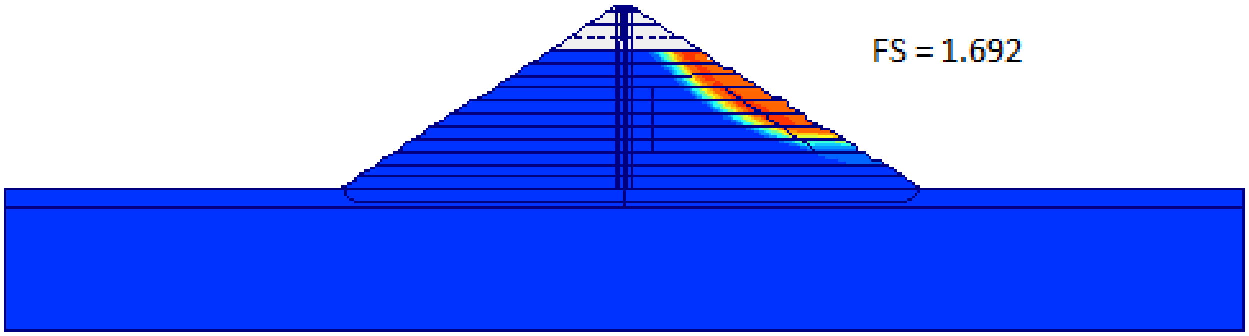

Figure 23: Plot of total increments (shadings) - (Phase: 30, Intermediate phase)

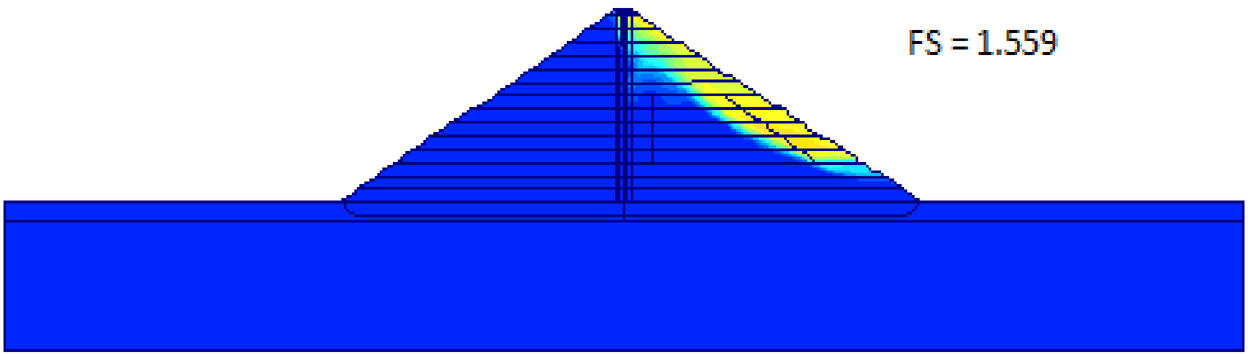

Figure 24: Plot of total increments (shadings) - (Phase: 34, Last phase)

6.3 Results of Settlement Analysis

6.3.1 Using Geostudio 2004

The magnitude and distribution of the vertical and horizontal displacements are shown in Figures below for the maximum cross section and full reservoir condition.

In Plaxis 2D analysis, additional displacements are generated during a Safety calculation. The total displacements in each stage (at failure) give an indication of likely failure mechanism. The shading of the total displacements indicating the most applicable failure mechanism of the embankment in the final stage.

Displacement results shows in both Geostudio and Plaxis gave almost similar result in both vertical and horizontal directions. As the impervious layer ranges from 0.5 m at the top to 1.5 m at the bottom, the horizontal displacement must be at most less than 0.5 m in order not to have cracking through the core.

The maximum vertical and horizontal displacements were 1.5957 m and 0.325 m for steady state condition; and 1.608 m and 0.3205 m for end of construction conditions respectively.

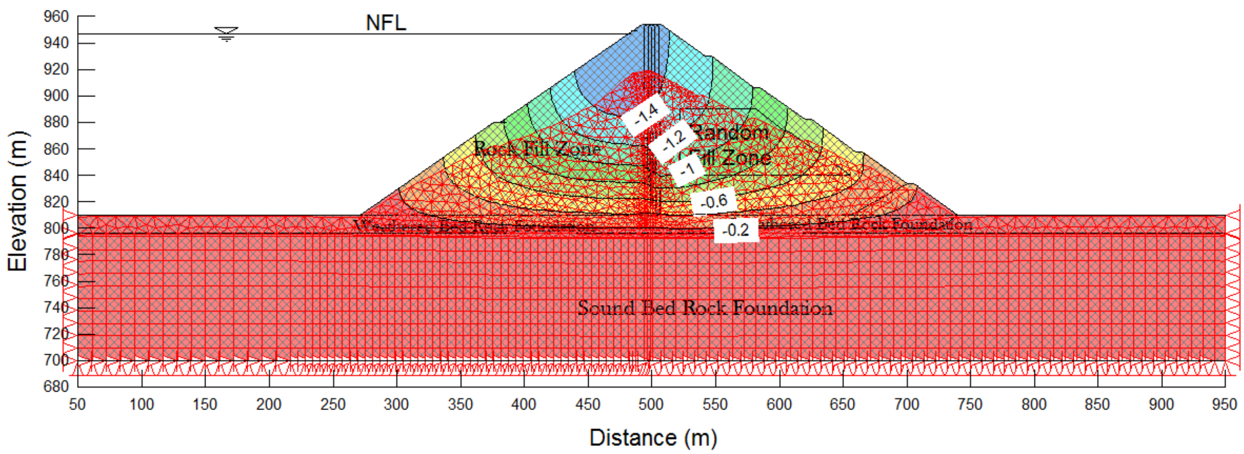

Figure 25: Vertical settlement in the dam body and foundation (Steady state condition).

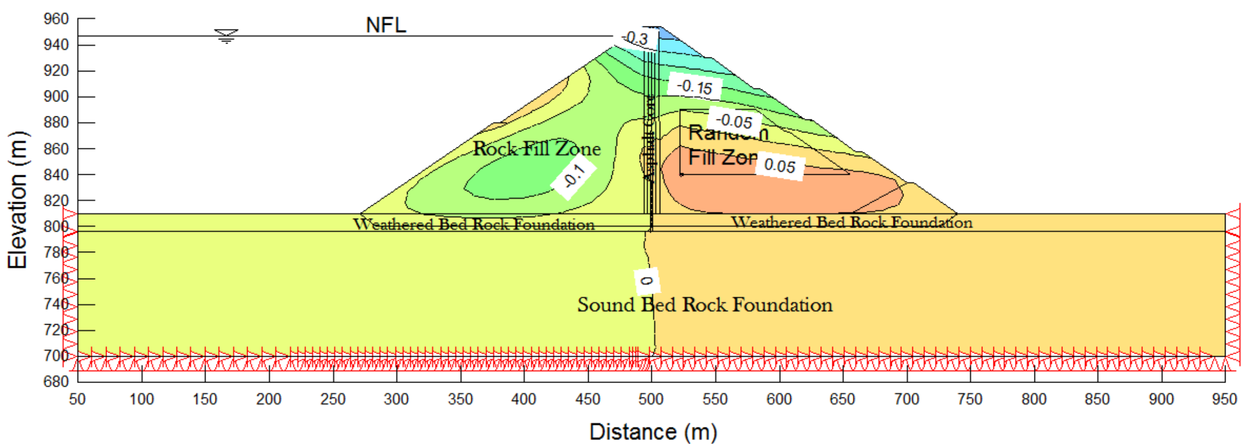

Figure 26: Horizontal Displacement in the dam body and foundation (Steady state condition).

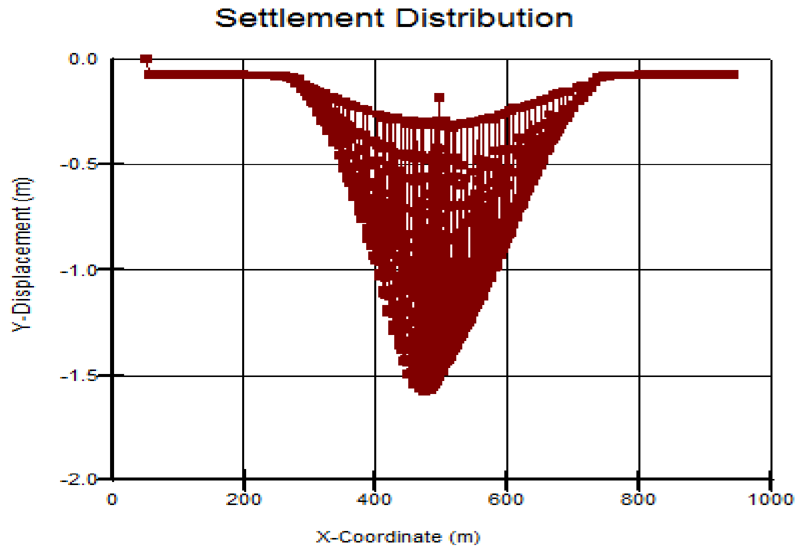

Figure 27: Vertical settlement Distribution in the dam body and foundation (Steady state condition).

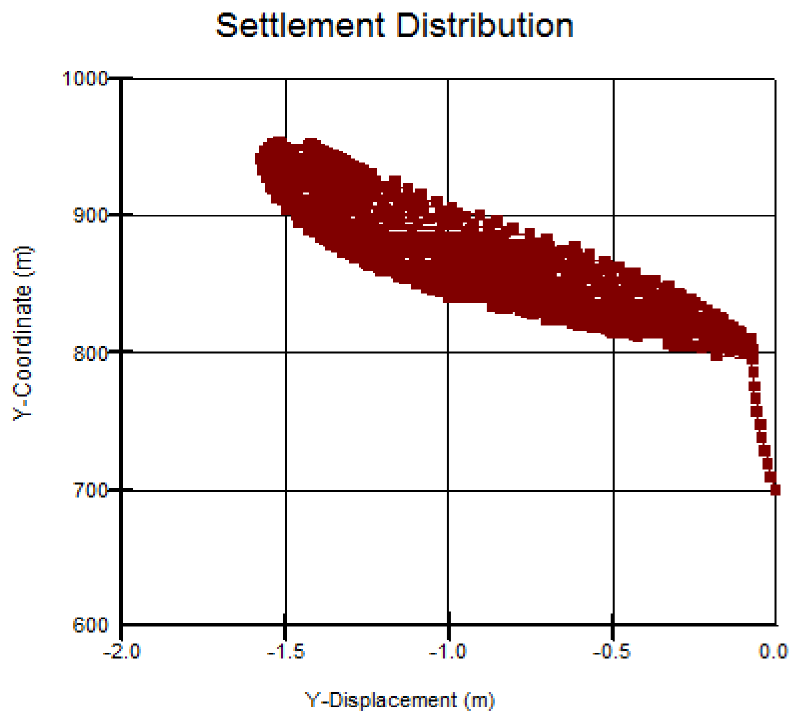

Figure 28: Vertical settlement Distribution in the dam body and foundation (Steady state condition).

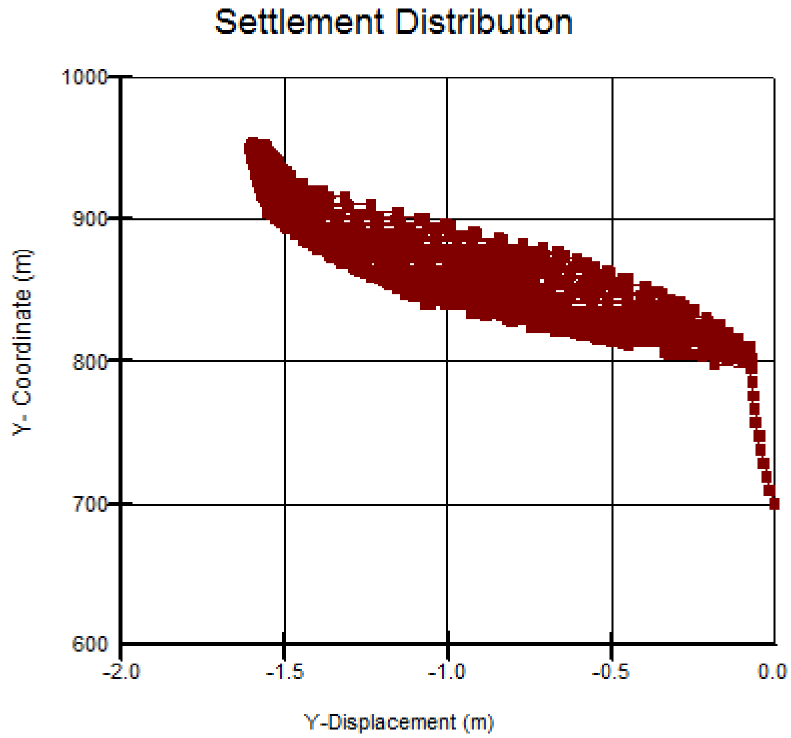

Figure 29: Vertical settlement Distribution in the dam body and foundation (End of construction).

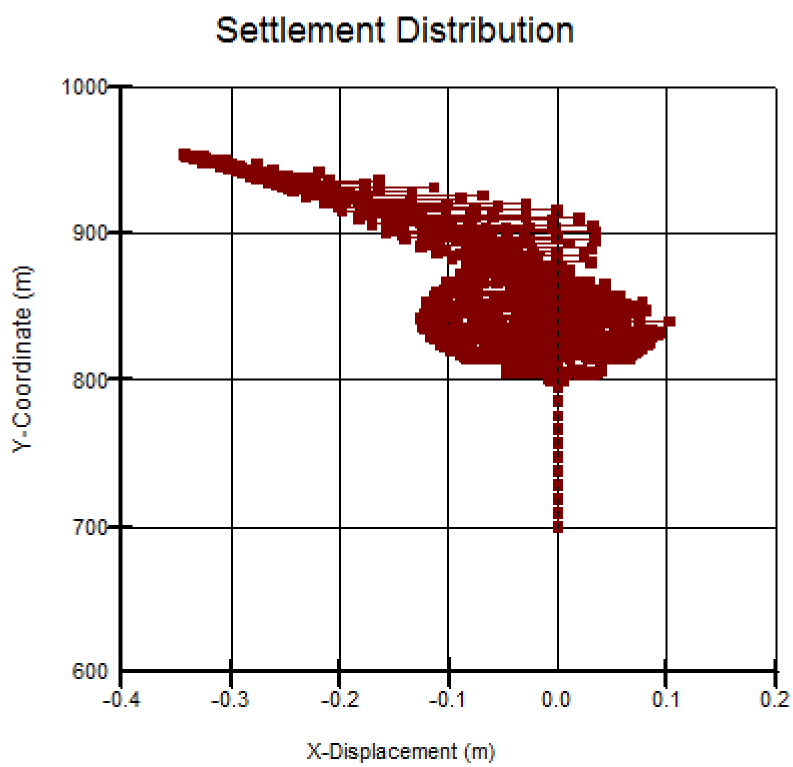

Figure 30: Settlement Distribution in the dam body and foundation (Steady state condition).

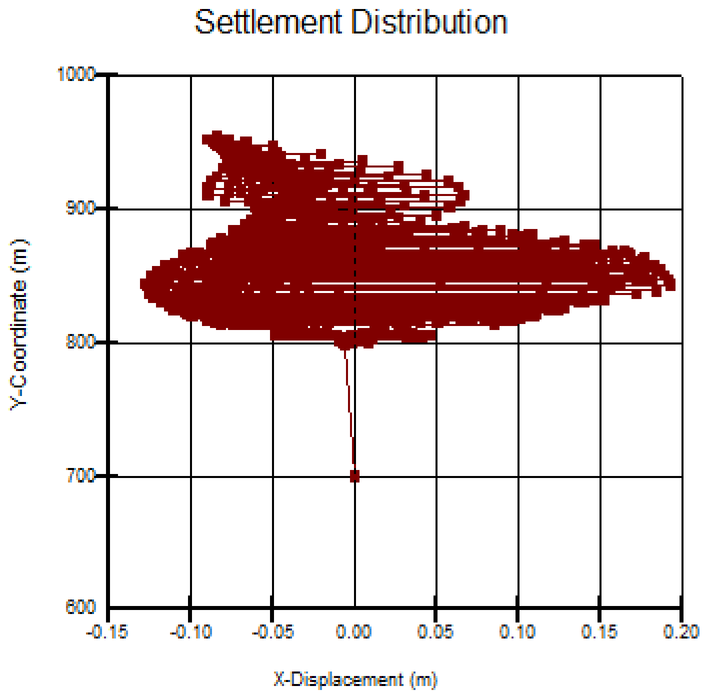

Figure 31: Settlement Distribution in the dam body and foundation (End of construction).

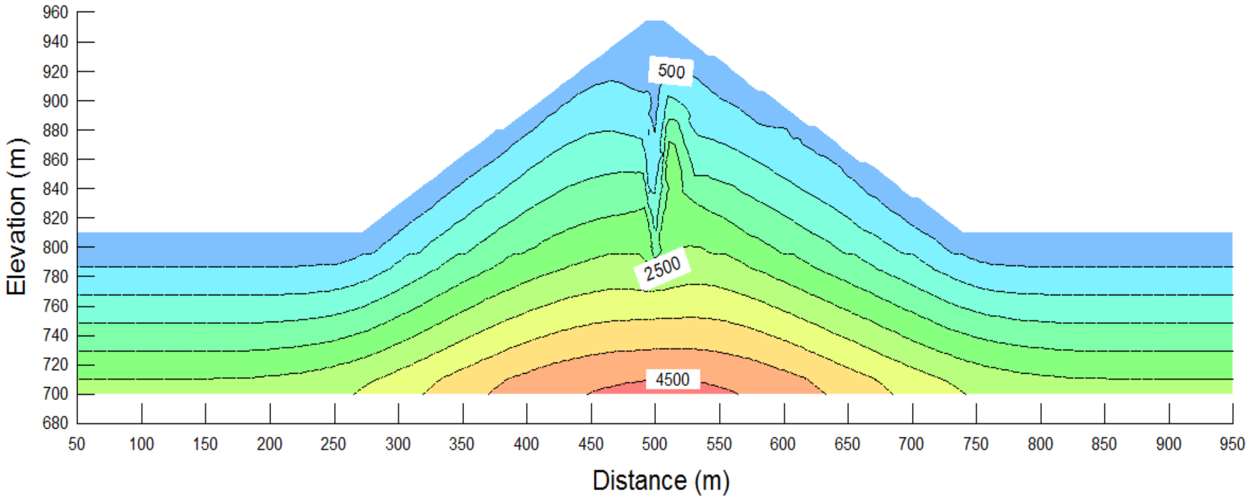

Figure 32: Y-total stress in the dam body and foundation.

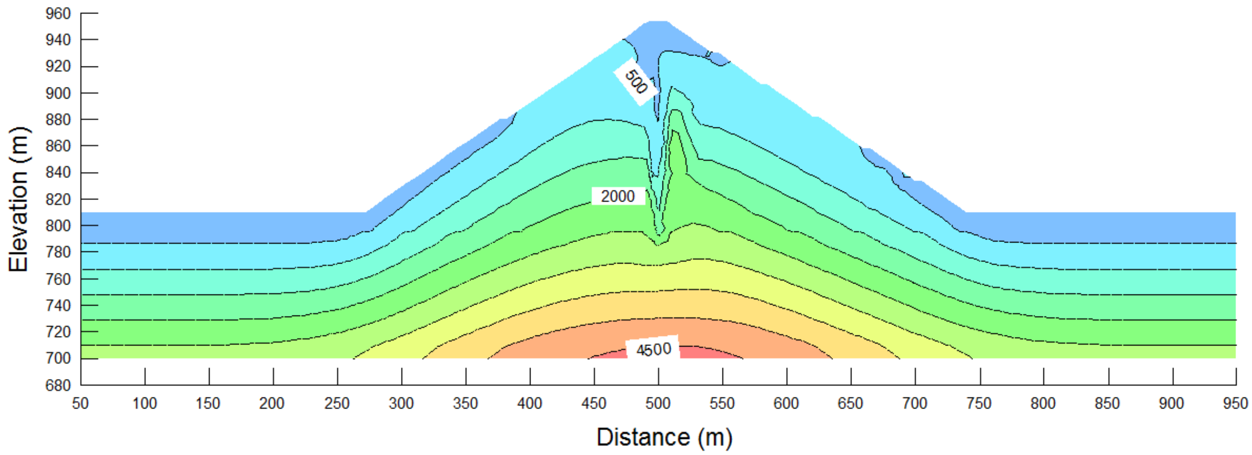

Figure 33: Maximum total stress in the dam body and foundation.

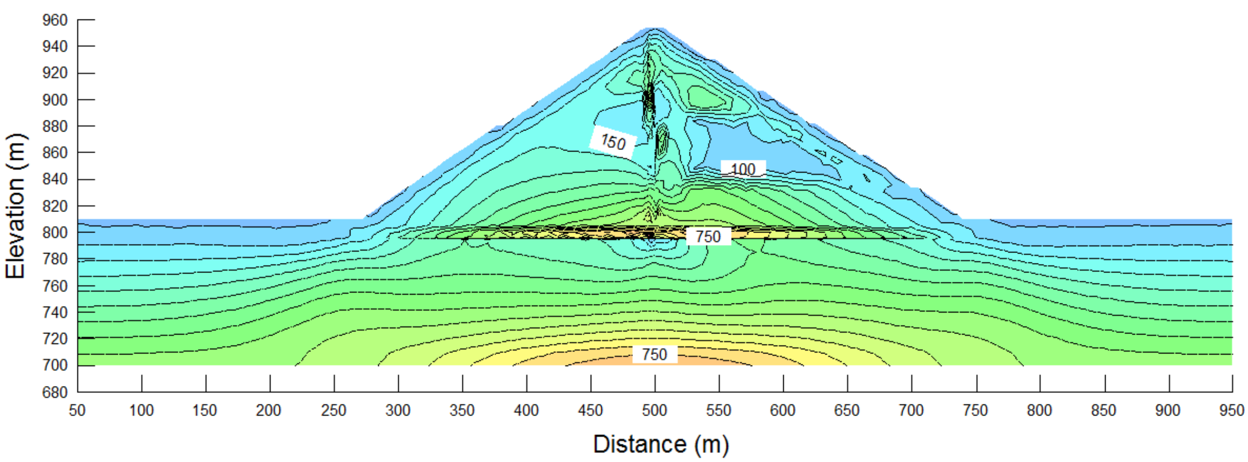

Figure 34: Minimum total stress in the dam body and foundation.

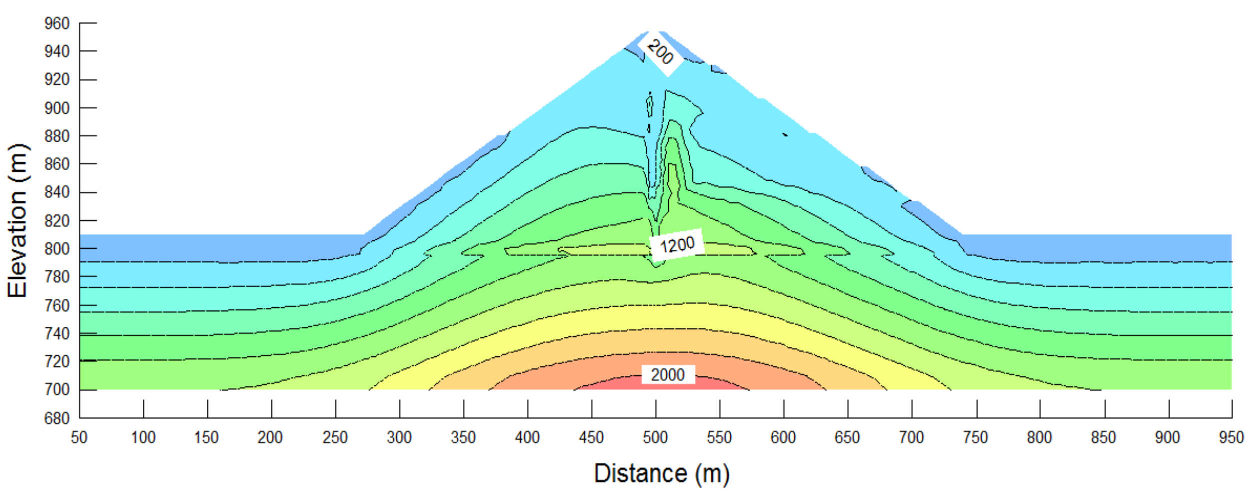

Figure 35: Mean total stress in the dam body and foundation.

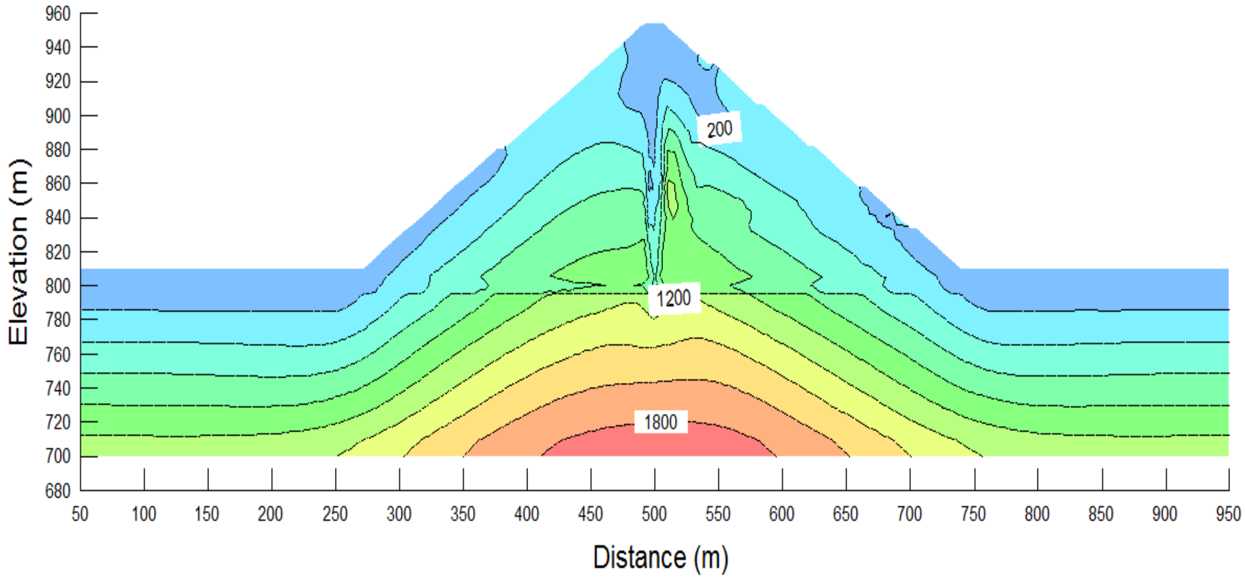

Figure 36: Maximum Shear stress in the dam body and foundation.

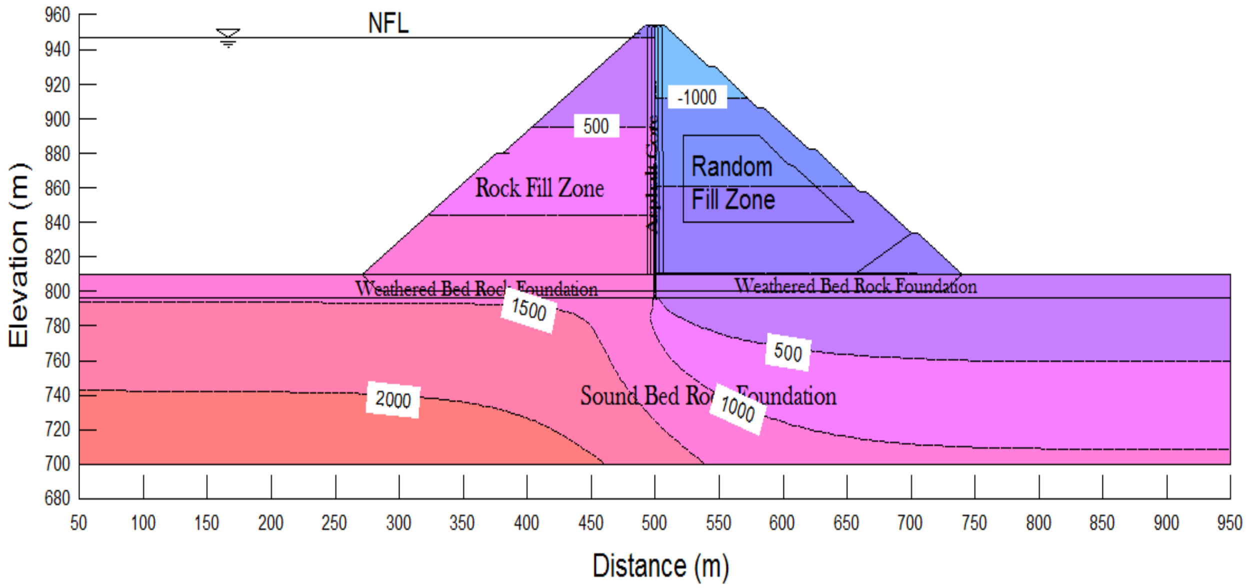

Figure 37: Pore water pressure in the dam body and foundation.

6.3.2 Using Plaxis V8.5

Deformation analysis carried out using this application represents the real embankment construction approaches. The magnitude and distribution of the vertical and horizontal displacements at different phases are shown in Figures below for the maximum cross section.

The total displacements in the final stage gave a maximum of 1.56 m vertical, figure 40, and 0.4137 m, figure 41, horizontal settlements nearly central towards the right side of the embankment. The results displayed in both cases as vertical settlement coincides with USBR recommendation for rockfill dams.

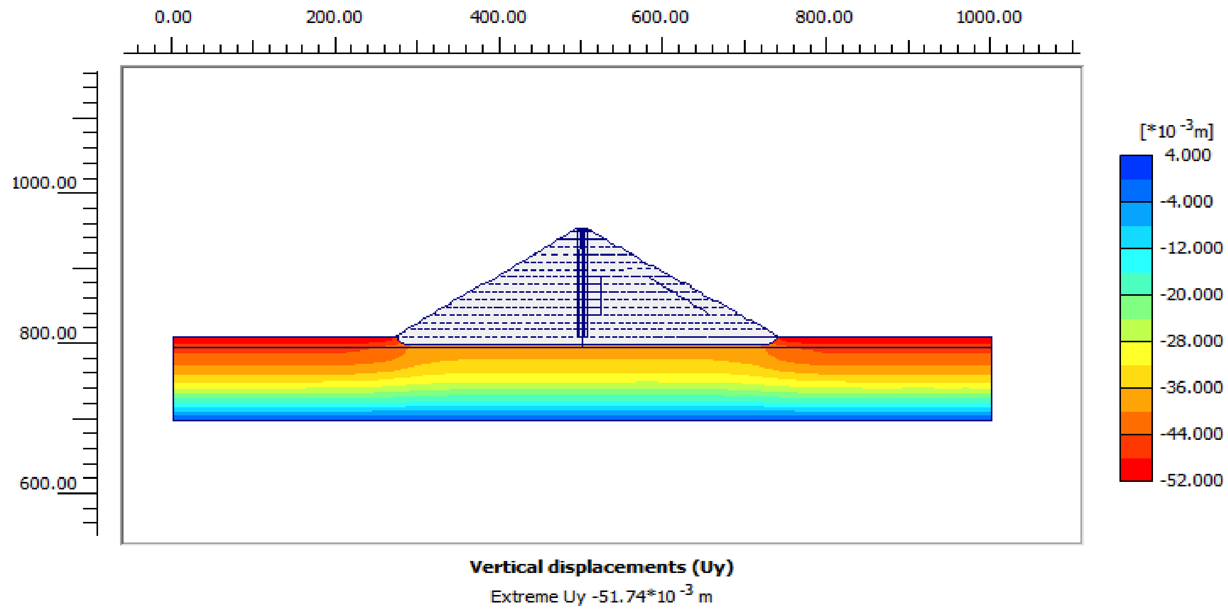

Figure 38: Plot of vertical displacements (shadings) - (Phase: 1, First phase)

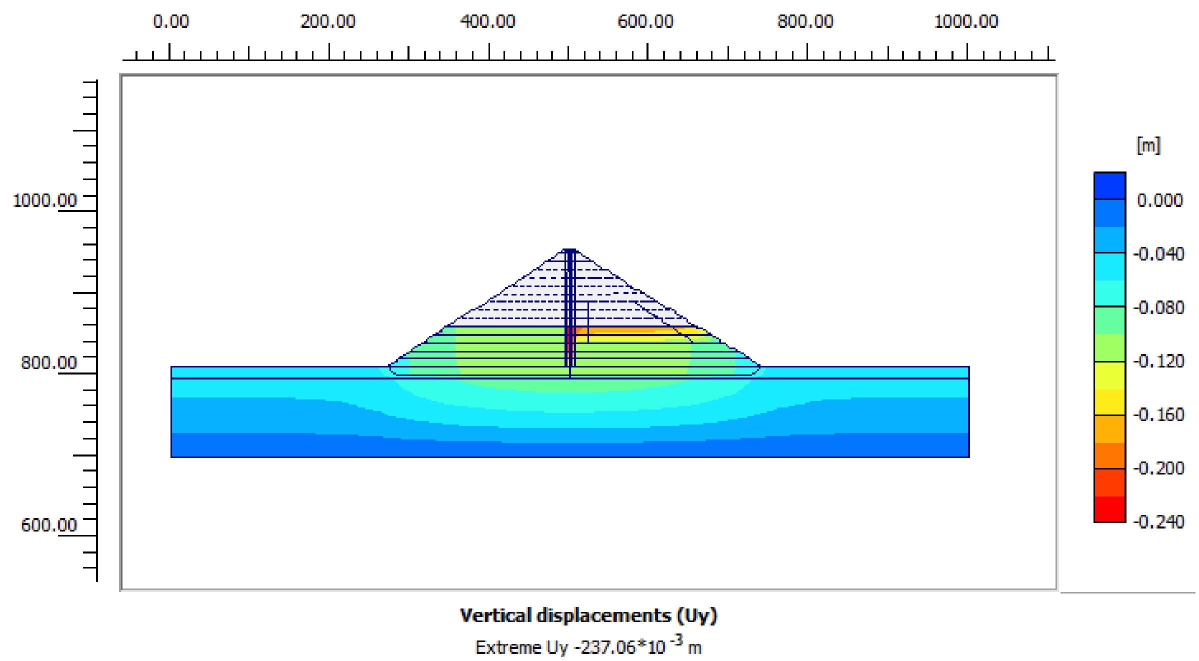

Figure 39: Plot of vertical displacements (shadings) - (Phase: 7, Intermediate phase)

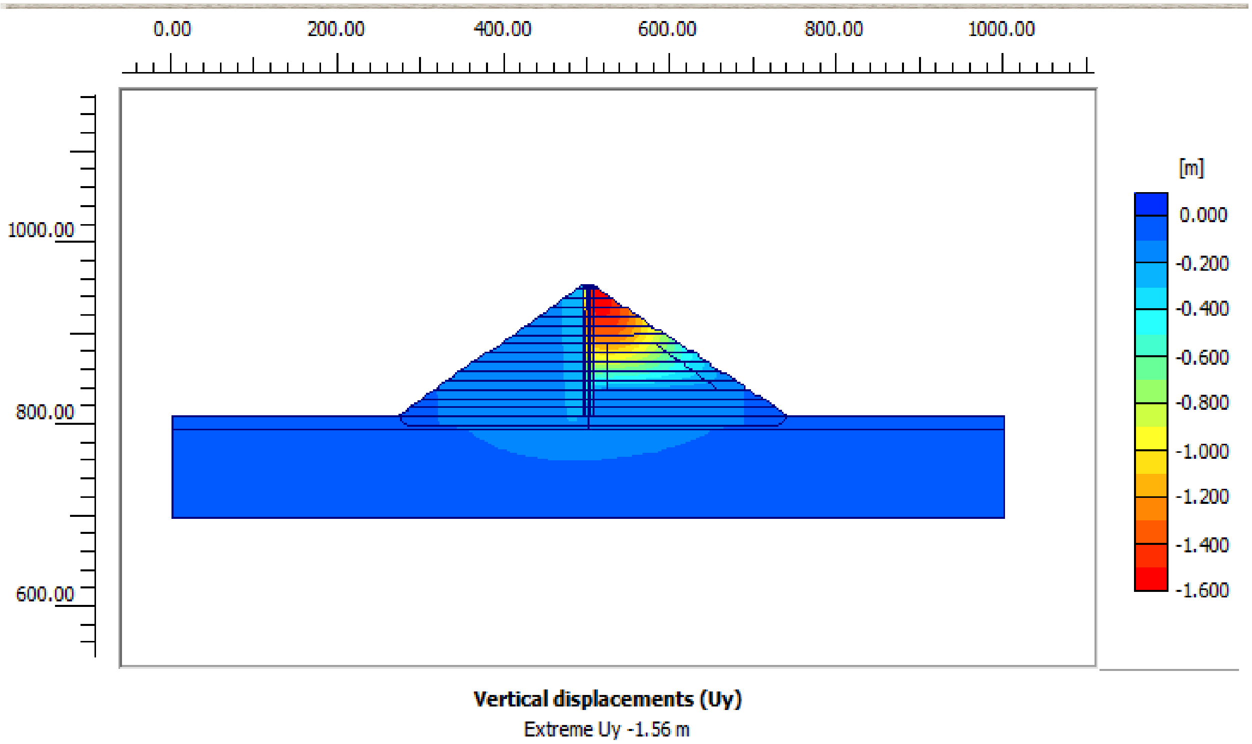

Figure 40: Plot of vertical displacements (shadings) - (Phase: 17, Last phase of Plastic Calculation)

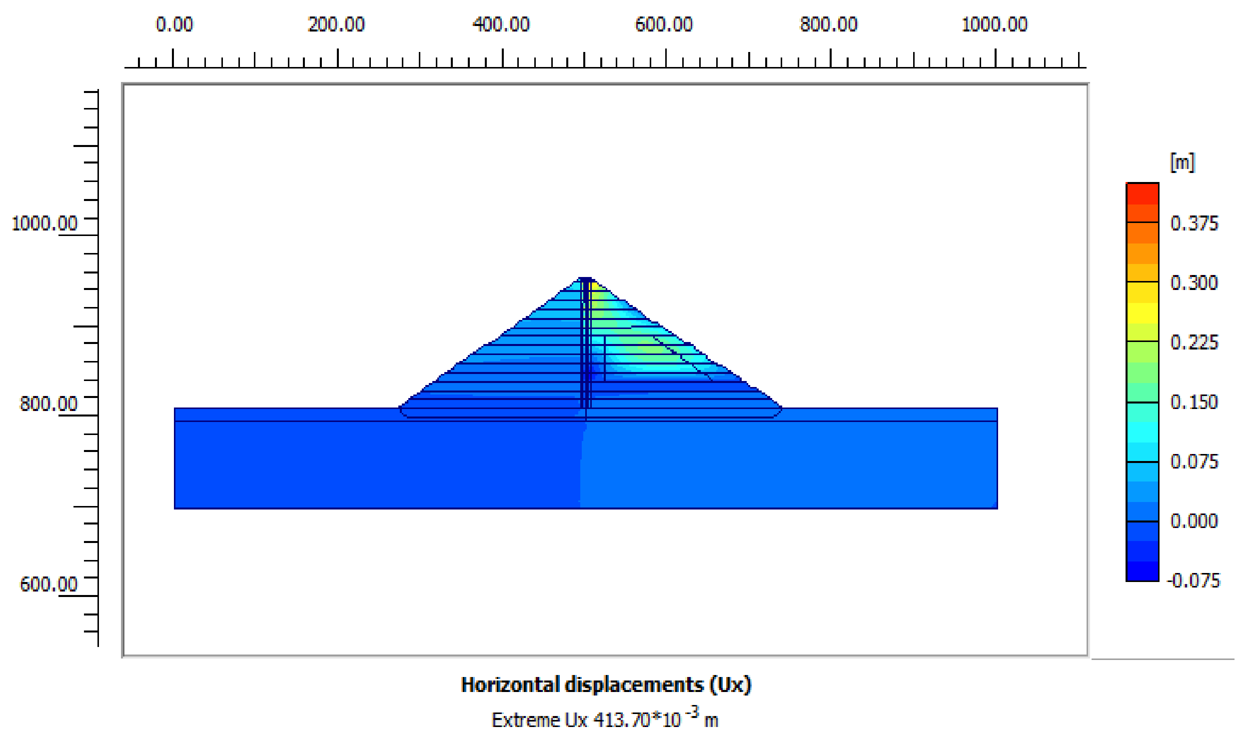

Figure 41: Plot of horizontal displacements (shadings) - (Phase: 17, Last phase of Plastic Calculation)

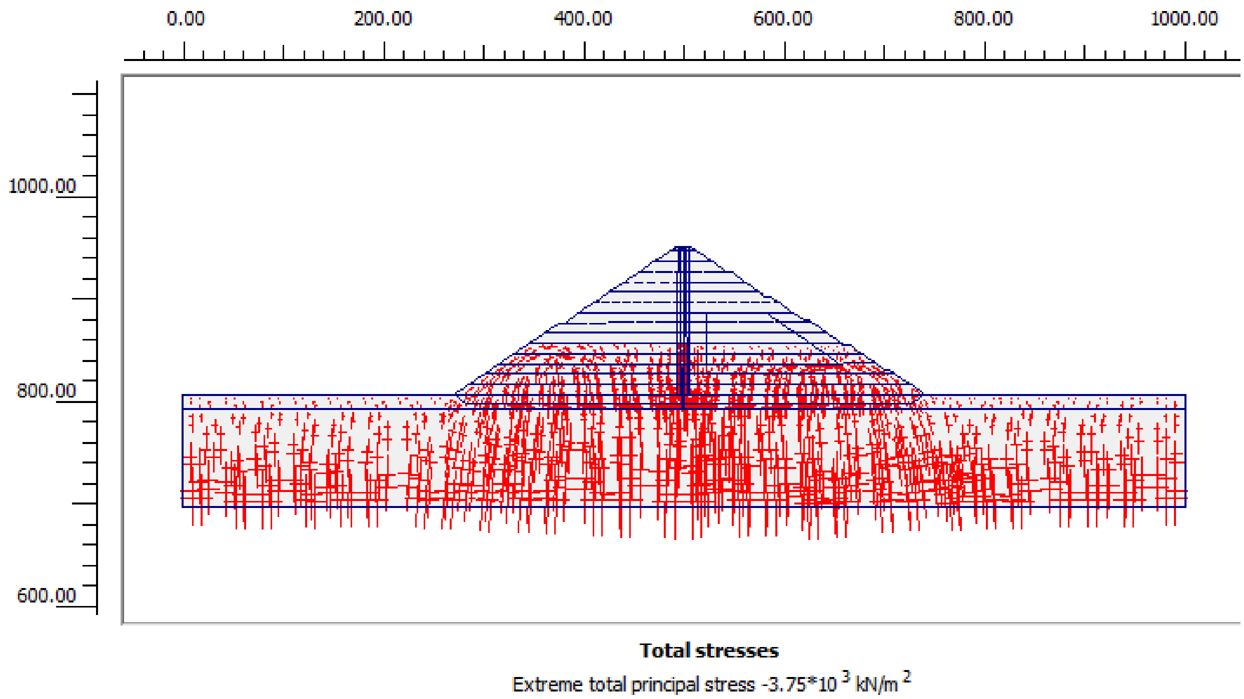

Figure 42: Plot of Total stresses (shadings) - (Phase: 7, Intermediate phase)

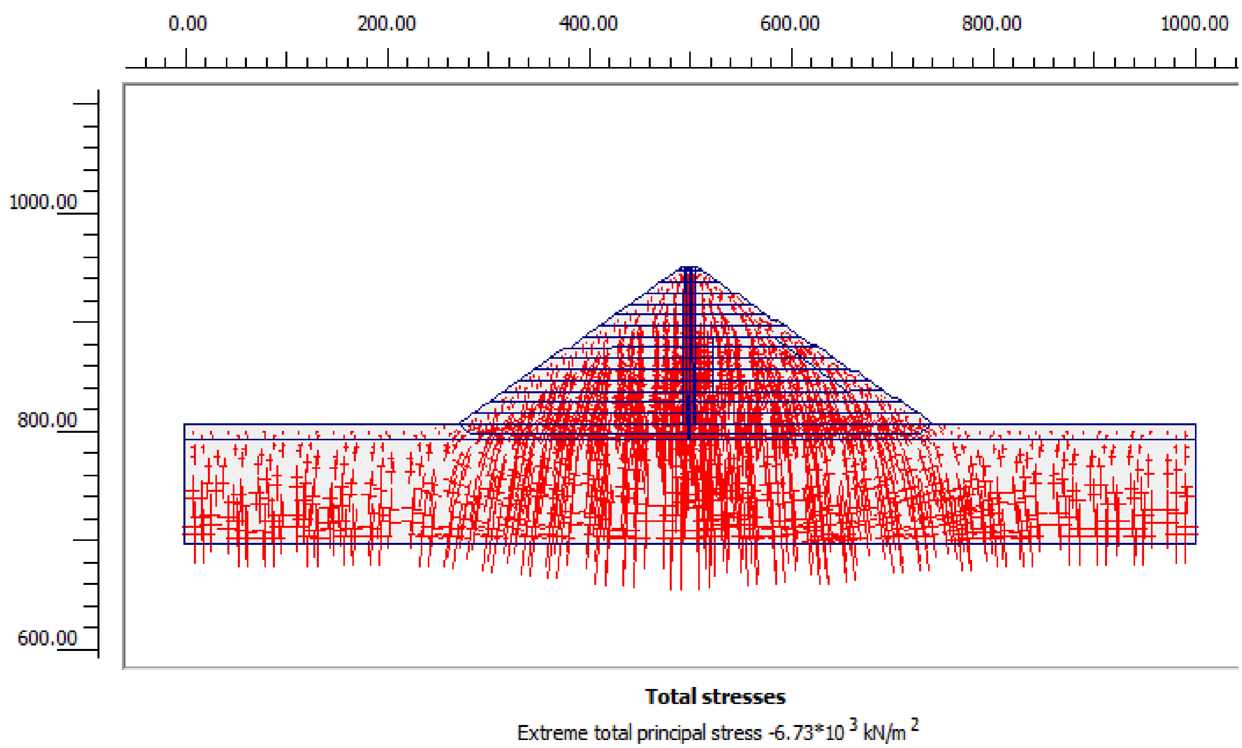

Figure 43: Plot of Total stresses (shadings) - (Phase: 17, Last phase of Plastic Calculation)

6.4 Design of Filters

In order to design the filter thickness, seepage analyses through the dam is conducted using the state of the art Finite Element Method based computer program – Seep/W from Geo-slope international, 2004.The computed discharges as shown on the Figures are used to calculate the Filter thickness. Besides, the computed phreatic surface, as shown by the blue line on the figures, is used to set up the pore water pressure line in the stability analysis of the dam.

6.4.1 Design of Vertical Filter

Discharge, q = 2.9146x$10^{-5}$$m^3$/s/m length (from seepage analysis, SEEP/W result).
Permeability of filter, k = 1.0 x 10^{-4} m/s (assumed).
Angle of discharging face with horizontal, α = 90°.
Thickness of inclined filter, t, is given by Darcy's law:

$$q = kiA$$

Where, A = t × 1 = t; i = sin α = 1.
2.9146*10^{-5} = 1.0 × 10^{-4} ×1 × t
$\Rightarrow$ t = 291.48 mm
Considering a factor of safety of 2, t = 582.96 mm.

However, according to USBR, 1987 and Fell et al., filter design from construction practical considerations for using small construction equipment's and to account for leakage through cracks in core of the dam, it is proposed to provide a 2.4 to 3m thick vertical coarse filter. For this case, 2.5m thick filter is selected. In addition to the filter a 3m transition is provided as transition between the filter and the shell. Transition is used to protect erosion of fines from the filter to the shell in downstream side of the embankment. Moreover, in case of temporary cracking of the asphalt core it will drain the water leakage acting as a filter and protects the filter-transition zone against segregation problem. Filters are taken up to the dam crest level, to be effective in the event of large crest settlements, which are likely to be associated with transverse cracking. This would give greater security in the event of large crest deformations.

Chapter 7

7.0 Conclusion

This paper describes design and analyses of high embankment dams with special consideration of material constraint conditions, case of Wolkayite high dam that is currently under construction.

From the compiled information on earth-rockfill dams with asphalt concrete core and the performed studies, it can be concluded that earth-rockfill dam solution is adequate from the technical point of view and with remarkable structural performance results, thus constitutes a valid alternative dam solution. In spite of the few economic data, this type of solution seems competitive, especially for locations where fine materials for the construction of a traditional core are scarce.

The alternative materials that form the body of the dam were the utilization of random fill material at the downstream shoulder surrounded by alluvial gravel and laid on rockfill. The search of impervious material for making the embankment watertight was finally reached in using asphalt concrete as a core. Due to its highly plastic nature, clay materials that were present around the dam site were discarded.

A series of models were performed to investigate the 2D effect on seepage phenomenon of Wolkayite embankment dam. It can be observed that due to the presence of asphaltic concrete core and adequate drainage provisions in the dam, seepage problem does not affect the free sloping surface of the dam. From the results of static and seismic slope stability analyses, it can be concluded that the proposed earth-rockfill dam section is quite safe against slope failure.

Stability analysis during rapid drawdown is an important consideration in the design of embankment dams. During rapid drawdown, the stabilizing effect of the water on the upstream face is lost, but the pore-water pressures within the embankment may remain high. As a result, the stability of the upstream face of the dam can be much reduced. The dissipation of pore-water pressure in the embankment is largely influenced by the permeability and the storage characteristic of the embankment materials. Highly permeable materials drain quickly during rapid drawdown, but low permeability materials take a long time to drain. In this study since the dam is almost 90% kept as dead storage, the upstream zone of the embankment is designed as free draining to tackle such problem by rapid drawdown.

Deformations of an embankment dam start occurring during the construction of the dam. These deformations are caused by the increase of effective stresses during the construction by the consecutive layers of fill material and also by effects of creep of material. The results found in this study were in line with USBR recommendations.

Analysis made using Plaxis were quite outstanding in simulating real working conditions of embankment dam construction. The limitation with this application was in stage construction in line with water loading while increasing water depths almost greater than thirty meters.

The analysis results that were found from these two Geostudio and Plaxis applications displays almost the same. The following table represents some of them.

Table 7: Comparison of the Analysis Made with Geostudio & Plaxis

Characteristics	Analysis Results at End of Construction	
	Geostudio	Plaxis
Deformation		
Vertical Settlement	1.608	1.56
Horizontal Settlement	0.3205	0.4137
Stability		
Factors of Safety	1.714	1.559

The major significance of this alternative than that of the dam entirely built in rockfill were not only reduces volume of the dam while reducing total cost of the dam but also shortens the time required to accomplish the project. It is quite interesting to mention the amount of cost that was proposed to be invested on this project was ***3,063,938,760.48*** **(ETB)*,*** but now the total amount of cost was reduced to ***2,378,293,314.65*** **(ETB)*.*** The amount of money saved by this alternative work was **685,645,445.83 (ETB).**

Finally, in the preparation of this study the author got enormous knowledge in the design and analysis of embankment dams. Especially, those sites that favor embankment dams but have shortage of constituent materials in making the body of the dam were the major lessons learned in this thesis. This work may also help to get insight for students and as an input for researchers of similar field in future studies.

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Appendix

Table 8: Elevation Area and Volume of the Reservoir with Sediment Accumulation

Elevation (m)	AREA (m^2)	Total Volume (m^3)	AREA (km^2)	VOLUME (MCM)	Dam crest Level (m)	50 Years Sediment Level (m)	MFL (m)	NFL (m)
820	70.9857	0	0.000	0.000	954	855	949.1	946.4
825	12730.2672	32003.13225	0.013	0.032	954	855	949.1	946.4
830	255183.4271	701787.368	0.255	0.702	954	855	949.1	946.4
835	522229.8013	2645320.439	0.522	2.645	954	855	949.1	946.4
840	836538.5951	6042241.43	0.837	6.042	954	855	949.1	946.4
845	1201326.07	11136903.09	1.201	11.137	954	855	949.1	946.4
850	1870440.594	18816319.75	1.870	18.816	954	855	949.1	946.4
855	2403739.651	29501770.36	2.404	29.502	954	855	949.1	946.4
860	3112015.403	43291158	3.112	43.291	954	855	949.1	946.4
865	3967968.601	60991118.01	3.968	60.991	954	855	949.1	946.4
870	6010885.141	85938252.36	6.011	85.938	954	855	949.1	946.4
875	11735883.35	130305173.6	11.736	130.305	954	855	949.1	946.4
880	14585722.07	196109187.1	14.586	196.109	954	855	949.1	946.4
885	18078331.25	277769320.4	18.078	277.769	954	855	949.1	946.4
890	21744308.74	377325920.4	21.744	377.326	954	855	949.1	946.4
895	28175575.82	502125631.8	28.176	502.126	954	855	949.1	946.4
900	34249713.46	658188855	34.250	658.189	954	855	949.1	946.4
905	39995311.43	843801417.2	39.995	843.801	954	855	949.1	946.4
910	46543988.57	1060149667	46.544	1060.150	954	855	949.1	946.4
915	52079924.38	1306709450	52.080	1306.709	954	855	949.1	946.4
920	58865644.34	1584073371	58.866	1584.073	954	855	949.1	946.4
925	65583583.63	1895196441	65.584	1895.196	954	855	949.1	946.4
930	72602779.68	2240662350	72.603	2240.662	954	855	949.1	946.4
935	80582534.73	2623625636	80.583	2623.626	954	855	949.1	946.4
940	87688125.02	3044302285	87.688	3044.302	954	855	949.1	946.4
945	96123629.04	3503831670	96.124	3503.832	954	855	949.1	946.4
950	103914434.1	4003926828	103.914	4003.927	954	855	949.1	946.4
955	112641638.2	4545317009	112.642	4545.317	954	855	949.1	946.4
960	121226644.1	5129987714	121.227	5129.988	954	855	949.1	946.4
965	130022220	5758109875	130.022	5758.110	954	855	949.1	946.4
970	139470161.2	6431840827	139.470	6431.841	954	855	949.1	946.4

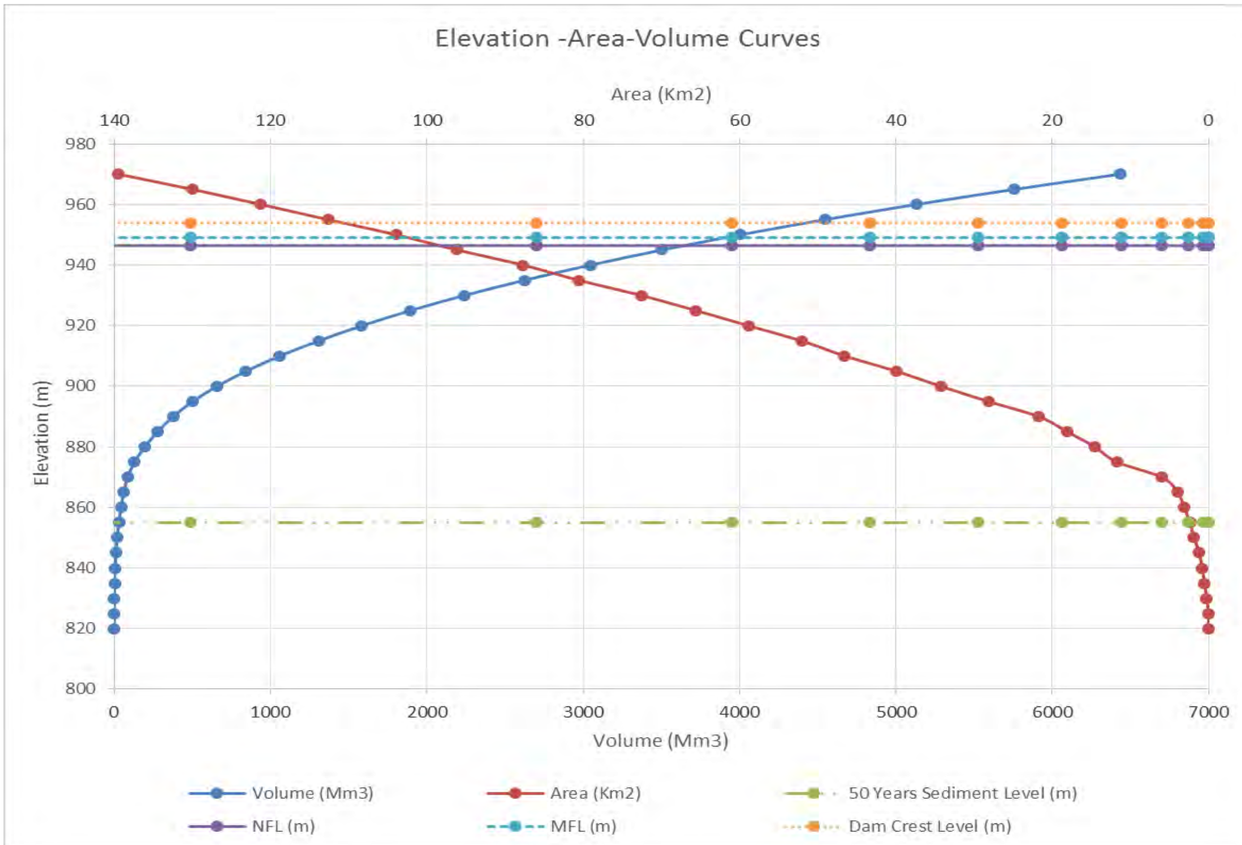

Figure 44: Elevation-Area-Volume Curves

Table 9: Location and Available Volume of Metvolcanic Shell Material

No	Type of material	Aerial distance from the dam	Thickness	Aerial extent (width*length)	Overburden (thickness and type)	Volume estimate	Remark
1	Weathered metavolcanic (SH2)	0.5-10Km	5-10m	80m* 10km	Up to 2m (average); completely weathered metavolcanic.	In excess of 16Mm^3.	This material is extensive in amount but the shear strength and permeability is low due to the high percentage of fines.
2	Fresh to slightly weathered Metavolcanic	0.5 -10Km	20m (in both sides of the river)	80m* 10km	10 to 20m is colluvial and completely to moderately weathered. metavolcanic.	In excess of 16Mm^3.	In most cases the fresh metavolcanic is found at a depth of 10 to 20m. It requires removal of top weathered rock and systematic blasting to obtain the right size. In some cases, excavation of the fresh metavolcanic could create hanging of the sedimentary rocks.
3	Fresh to sl. weathered metavolcanics (at UTM coordinates: 370949mE, 1520871mN)	3-3.5Km	50m (left side of Zarema River)	100m*250m	0-5m overburden is found in limited areas with convex terrains	About 1.25mM^3	In some areas along the Zarema River fresh to slightly weathered metavolcanic rocks are exposed on convex terrains. The cost for overburden excavation could be minimized by selecting quarry sites with less overburden thickness.

Table 10: Location and Available Volume of Basalt Shell Material

No	Type of material	Aerial distance from the dam	Thickness	Aerial extent (width*length)	Overburden (thickness and type)	Volume estimate	Remark
1	Weathered basalt (SH3)	5.5Km (average)	10m	1.5Km*2Km	Up to 0.5m (average); completely weathered basalt.	In excess of 30Mm³.	This material is extensive in amount but the shear strength and permeability is low due to high percentage of fines.
2	Fresh basalt	6-8Km	15--25m		The weathered basalt ranges from 15m to 25m thickness.	In excess of 20Mm³.	The fresh basalt is found at a depth varying from 15m to 25m (average 20m depth). It requires removal of top weathered basalt and systematic blasting of the fresh rock to obtain the right size.

Table 11: Location and Available volume of Alluvial/River Deposit

No	Type of material	Aerial distance from the dam	Thickness	Aerial extent (width*length)	Overburden (thickness and type)	Volume estimate	Remark
1	Zone 1	About 11 -12km	2m	$0.79Km^2$	0.3m to nil	$1.58Mm^3$	Surveyed, trench excavated and laboratory testing.
2	Zone 2	About 13-14 Km	2m	$0.58Km^2$	0.3m to nil	$1.18Mm^3$	Surveyed, partially trench excavated and laboratory testing.
3	Zone 3	About14-15km	2m	150m*1200m	0.3m to nil	$360,000m^3$	
4	Zone 4	0.4 to 8Km radius (both upstream and downstream).	4m	Upstream: $0.4Km^2$	nil	$1.6Mm^3$	Ground Survey, trench excavated and laboratory tested on collected samples
				Downstream: $0.44Km^2$	nil	$1.76Mm^3$	
5	Zone 5	25km from dam site, at Kalema River	1.5m	250m*1500m	0.3m to nil	$562,500m^3$.	
6	Zone 6	15-25Km; along the Tekeze River.	Over 15m but the exploitable thickness is estimated to be 5-10m.	$3.9Km^2$	nil	In excess of $20Mm^3$ is estimated.	Different access routes shall be sought. Percolating water will be difficult and appropriate method should be devised
					Total Estimated Volume	***Over 27Mm³***	

Table 12: Location and Available Volume of Sedimentary Rock

No	Type of material	Aerial distance from the dam	Thickness	Aerial extent (width*length)	Overburden (thickness and type)	Volume estimate	Remark
1	Weathered sedimentary	0-5Km	0.3m	500m*5Km	None to 0.2m.	Not more than $2Mm^3$.	This material has limited depth (not more than 0.3m) and the material is highly heterogeneous with mixtures of fragmented mudstone, siltstone, sandstone and conglomerate.
2	Sedimentary Rocks	0-5Km	20m in both sides of the river	500m* 5km	None to 0.3m, weathered sedimentary.	In excess of $20Mm^3$.	It has alternating layers of sandstone, siltstone, mudstone and conglomerate, which requires careful blasting and processing.

Table 13: Location and Available volume of Dolerite

No	Type of material	Aerial distance from the dam	Thickness	Aerial extent (width*length)	Overburden (thickness and type)	Volume estimate	Remark
1	Dolerite (RQ1)	7Km-10Km	20m	50m*200m	None	In excess of 200,000m³	Being crushed for aggregate. production
2	Dolerite (RQ2) (at UTM coordinate 358861mE, 1535834mN).	10Km	Over 20m (estimated)	200m*200m	None	In excess of 800,000m³	Under production for crushed aggregate. The depth is expected to be greater
3	Dolerite (RQ3) (UTM coordinate: 358814mE, 1521434mN)	11-12KM	Over 20m (estimated)	50m*100m	None	Over 100,000m³	Under production for crushed aggregate. The depth is expected to be greater.
4	Dolerite (RQ4) (UTM coordinate: 358177mE, 1519356mN).	12-13Km	Over 20m (estimated)	100m*2500m	None	About 5Mm³	The depth is expected to be greater
					Total volume estimate	**In excess of 6Mm³**	

Table 14: Elevation and Area Calculation of Different Parts of the Dam

Elevation (m)	Length (m)	Area (m^2)					
		Rockfill	Alluvial River Gravel fill	Random fill	Filter	Transition	Asphalt Core
805	110	4463.67	-	-	-	-	21.78
815	110	4417.44	-	-	50.92	60	13.73
825	140	4122.31	-	-	50.86	60	13.09
835	195	3797.17	-	-	50.79	60	12.45
845	241	1701.23	520.55	1260.26	50.73	60	11.82
855	296	1551.58	509.80	1115.53	50.66	60	11.18
865	340	1401.93	469.06	970.79	50.60	60	10.54
875	380	1252.28	485.31	809.05	50.53	60	9.90
885	420	1052.63	445.87	663.02	50.47	60	9.26
895	460	902.98	426.89	526.51	50.40	60	8.63
905	393	753.33	788.58	-	50.34	60	7.99
915	536	603.68	619.93	-	50.28	60	7.36
925	567	454.04	482.79	-	50.21	60	6.72
935	616	304.39	291.47	-	50.15	60	6.08
945	660	154.74	145.99	-	50.08	60	5.45
952	730	21.86	19.53	-	16.27	24	2.01
Total		**26955.25**	**5205.76**	**5345.15**	**723.29**	**864**	**157.99**

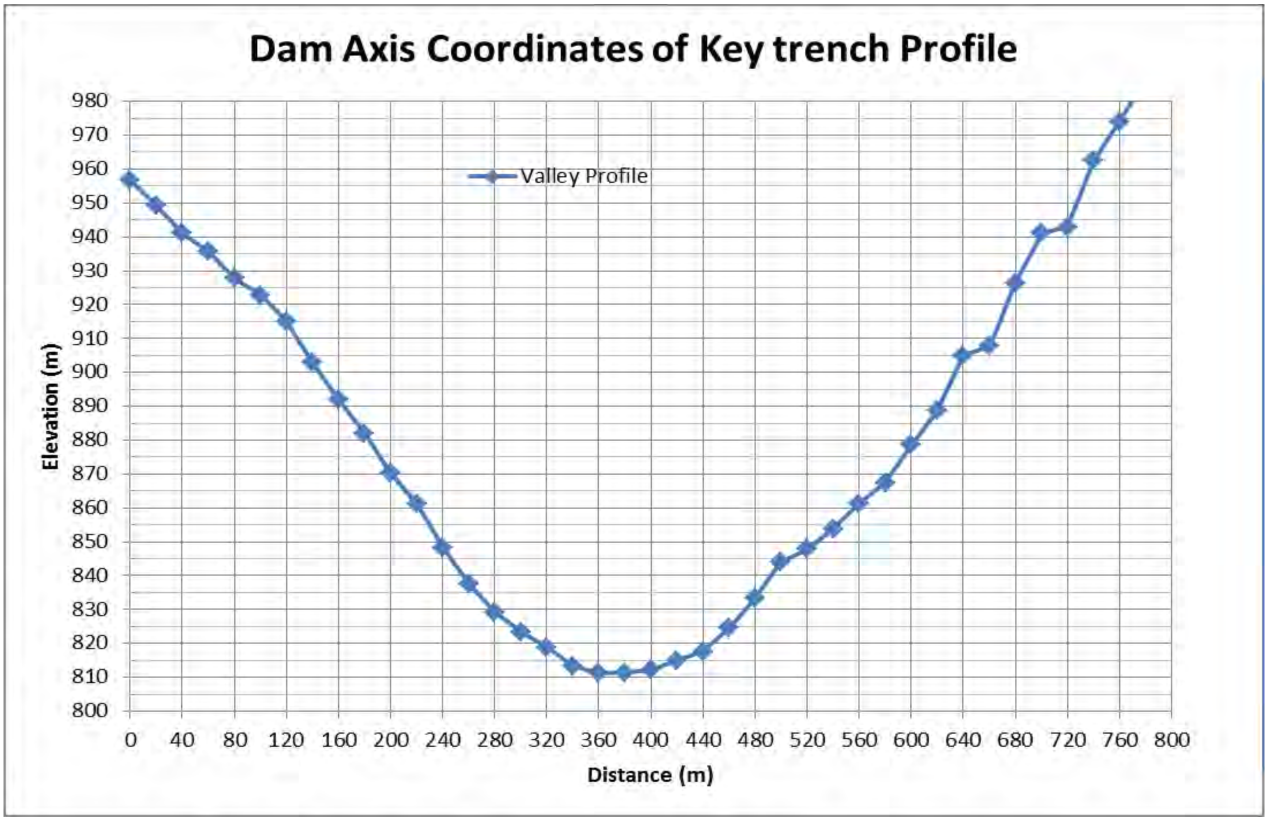

Figure 45: Dam Axis Coordinates of Key trench Profile

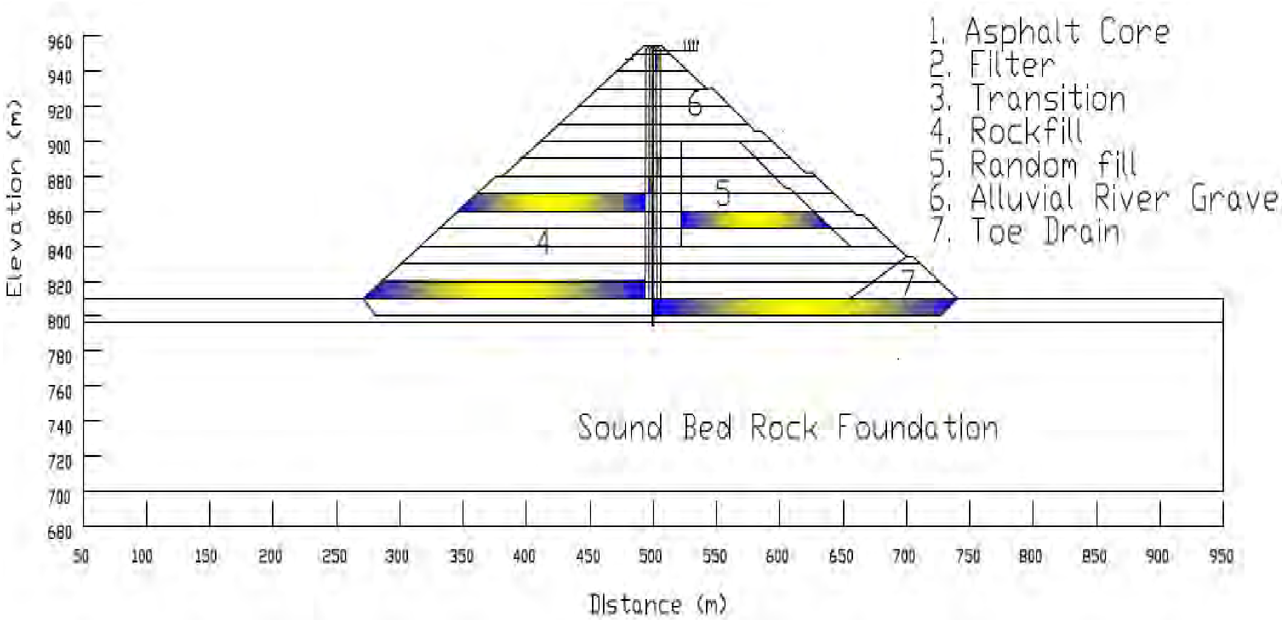

Figure 46: Dam profile with cross-sectional area computation at different stages

Table 15: Volume Calculation of Different Parts of the Dam

Elevation (m)	Length (m)	Volume (m^3)					
		Rockfill	Alluvial Gravel fill	Random fill	Filter	Transition	Asphalt Core
805		491004.10					2395.80
815	110	485917.89	-	-	5601.20	6600	1510.30
825	140	577122.88	-	-	7120.40	8400	1832.60
835	195	740447.60	-	-	9904.05	11700	2427.75
845	241	409995.97	125452.14	303723.43	12225.93	14460	2848.62
855	296	459267.35	150901.57	330195.78	14995.36	17760	3309.28
865	340	476656.13	159479.38	330068.43	17204.00	20400	3583.60
875	380	475866.67	184419.02	307439.34	19201.40	22800	3762.00
885	420	442105.27	187263.59	278466.68	21197.40	25200	3889.20
895	460	415371.95	196369.95	242193.63	23184.00	27600	3969.80
905	393	296059.99	309911.94	-	19783.62	23580	3140.07
915	536	323574.73	332284.73	-	26950.08	32160	3944.96
925	567	257437.90	273739.15	-	28469.07	34020	3810.24
935	616	187501.78	179545.09	-	30892.40	36960	3745.28
945	660	102126.29	96351.29	-	33052.80	39600	3597.00
952	730	15960.72	14257.41	-	11874.91	17520	1467.30
	Total	**6156417.23**	**2209975.26**	**1792087.30**	**281656.62**	**338760.00**	**49233.80**
	Cumulative Volume (m^3)			**10828130.21**			
	Cumulative Vol.+ (10% for settlement)			**11910943.23**			

Table 16: Bill of Quantity of Earth-Rockfill Dam with Central Asphalt Concrete Core (Note: Green shade shows current results)

ITEM NO	DESCRIPTION	UNIT	QUANTITY (Previous)	QUANTITY (Current)	RATE (ETB)	AMOUNT (ETB) (Previous)	AMOUNT (ETB) (Current)
1.0	Clearing of bushes and trees (light forest (average 100 trees/ha with a diameter of less than 30 cm)	ha	40.00	40.00	22,291.20	891,648.00	891648
2.0	Clearing, grubbing and strip topsoil to a depth of 0.5m	m3	307,500.00	307,500.00	88.91	27,339,825.00	27339825
3.0	Excavation for embankment foundation to max depth of one meter depth	m3	410,000.00	410,000.00	98.62	40,434,200.00	40434200
4.0	Excavation for cutoff trench up to 5m depth (weathered material)	m3	182,500.00	182,500.00	162.07	29,577,775.00	29577775
5.0	Ditto up to 10 m depth	m3	182,500.00	182,500.00	202.58	36,970,850.00	36970850
6.0	Excavation in rock for cutoff trench to 0.5m depth	m3	3,700.00	3,700.00	338.96	1,254,152.00	1254152
7.0	Clean up rock surface in cutoff trench	m2	3,700.00	3,700.00	42.55	157,435.00	157435
8.0	Cement slurry treatment of rock surface in cutoff trench	m2	2,000.00	2,000.00	196.95	393,900.00	393900
9.0	Concrete plinth under asphalt core (C-25)	m3	650.00	650.00	3,657.20	2,377,180.00	2377180
10.0	Asphalt concrete core	m3	42,756.00	49,233.80	7,785.89	332,893,512.84	383328951.1
11.0	Alluvial River Gravel fill of compacted shell material from borrow area within 3 km	m3	7,739,770.50	2,209,975.26	153.95	1,191,537,668.48	340225691.7
12.0	Rock fill of compacted shell material from borrow area within 5.5 km	m3	7,739,770.50	6,156,417.23	168.13	1,301,287,614.17	1035078428
13.0	Transition chimney filter obtained from the river bed screened	m3	150,000.00	338,760.00	329.41	49,411,500.00	111590931.6
14.0	Chimney filter obtained from the river bed screened	m3	150,000.00	281,656.62	329.41	49,411,500.00	92780507.19
15.0	Random fill of compacted shell material from borrow area within 5.5 km	m3	-	1,792,087.30	153.95	-	275,891,839.84
Sub-Total Comparison			***15,822,297***	***10,332,341.82***		***2,924,541,795.48***	***1,963,004,509.81***
Total Cost Comparison						***3,063,938,760.48***	***2,378,293,314.65***