ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

INSTITUTE OF TECHNOLOGY

DESIGN AND ANALYSIS OF HEADWORK ON A CHALLENGING FOUNDATION CONDITION

(The Case of Kuraz Project on Omo River)

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

Fekremariam Negash

Addis Ababa

Ethiopia

June 2014

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Fekremariam Negash.

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LIST OF ABBREVIATIONS AND SYMBOLS

a.m.s.l	Meter above mean sea level
AACE	Association for the advancement of cost engineering
BH	borehole
BOQ	Bill of Quantities
C	crest
D/S	Downstream
EEPCO	Ethiopian Electric Power corporation
EL	Elevation
ETB	Ethiopian birr
EWWDSE	Ethiopian Waterworks Design & Supervision Enterprise
FOS_{min}	Minimum Factor of safety
HEC	hydrologic engineering center
HFL	high flood level
HGL	Hydraulic grade line
HMS	Hydrologic Modeling System
ICOLD	International Commission on Large Dams
ITCZ	Inter-Tropical Convergence Zone
J	Jump
JHC	Jump height curve
MoWR	Ministry of water resources
MWL (m)	Maximum water level
NWL	Normal water level
PGA	probability ground acceleration
PMF	probable maximum flood
PMP	probable maximum precipitation
RCC	roller-compacted concrete
S_b	Stilling basin
SEDSE	Southern Ethiopia Design & Supervision Enterprise
SI	International unit system
SNNPR	Southern Nations, Nationalities and Peoples Region
TEL	Total energy line
TWC	Tail water curve
U/S	Upstream

USACE	United States Army Corps of Engineers
USBR	United States Bureau of reclamation
USCOLD	United States Commission on Large Dams
USSD	United States Society on Dams
USSD	United States Society on Dams
UU	Unconsolidated un drained
WBS	Work Breakdown Structure
WBS	Work Breakdown Structure
WWDSE	Water Works Design and Supervision Enterprise
n	Manning‟s roughness coefficient
D_1 (m)	Depth of water before the formation of hydraulic jump
D_2 (m)	Depth of water after the formation of hydraulic jump
H (m)	Maximum static head
d (m)	Depth of downstream cutoff wall
b (m)	Total impervious floor length
G_E	Safe exist gradient
h (m)	Unbalanced head
G	Specific gravity
t (m)	Floor thickness
Q (m^3/s)	Design flood
Le (m)	Effective length of crest for calculating discharge
L (m)	Net length of crest
N	Number of piers
Kp	Pier contraction coefficient
Ka	Abutment contraction coefficient
He (m)	Total head on crest
V_a (m/s)	Approach velocity
h_a (m)	Approach velocity head
X (m)	Horizontal coordinate
Y (m)	Vertical Coordinates
Ho (m)	Design Head
R (m)	Normal depth of scour below the highest flood level
f	Silt factor
q (m^3/s/m)	Intensity of flood discharge

D (m)	Depth of scour below the original river bed
h_f(m)	Flood surcharge
h_1 (m)	Height of chute block
W_1 (m)	Width of chute blocks
S_1 (m)	Spacing b/n chute blocks
h_2 (m)	Height of Dentainted sill
W_2 (m)	Front width of Dentainted sill
S_2 (m)	Spacing b/n Dentainted sills
P (m)	Height of the embankment
T (m)	Top crest width
$\gamma_{d\ max}$ (kN/m^3)	Maximum Dry unit weight
C" (kN/m^2)	Effective (true) cohesion
C (kN/m^2)	Apparent cohesion
Ø" (degrees)	Effective (true) angle of shearing resistance
Ø (degree)	Apparent angle of shear resistance
m_v(m^2/kN)	Coefficient of volume compressibility
k (m/s)	Coefficient of horizontal permeability
E (kN/m^2)	Young"s modulus
ν	Poisson"s ratio
C_d	Coefficient of discharge
g (m^2/s)	Acceleration due to gravity
El (m)	Elevation
E_1 (m)	Pre jump specific energy
E_2 (m)	Post jump specific energy
V_1 (m/s)	Pre jump Velocity
Fr_1	Pre jump froud number
Fr_2	Post jump froud number
SDF	Spillway Design Flood
HMRs	Hydro meteorological Reports
NWS	National Weather Service

ABSTRACT

The foundation geology at a diversion site often dictates the type of dam suitable for that site. The strength, thickness, and inclination of strata; permeability; fracturing; and faulting are all important considerations in selecting the diversion structure type. Occasionally, like the problems in Kuraz project the foundation formation show clear distinction in vertical level of appearance of a given material, two clear distinct faults within its longitudinal span axis, the structure‟s immediate foundation is resting on alternating strong and weak formations with different bearing capacity exerting differential pressure creating overstress in the structure will create differential settlement and the existence of the sandy silt formation at different levels along the longitudinal axis creates a liquefaction prone soil during an Earth quake. The major objective of this project is to study the Design and analysis of a diversion head work or small dam on a challenging foundation condition especially solving the problem on Kuraz Project on Omo River by providing technically, economically and functionally plausible alternative solutions.

In this research the following have been accomplished: important data were collected from existing documents, laboratory test results and other secondary data sources of the project and the final computation and the analysis is presented using Geo-Studio soft wares. Thus, the scope of the research will be limited to identifying or selection of appropriate headwork typology, hydraulic design and analysis of a foundation for the structures on the existing complex geology requiring very careful approach. In additions the research emphasizes to demonstrate theories, procedures and techniques through practical applications.

For this particular challenging foundation conditions, it was proposed three different possible alternatives. which wereOn the rock foundation (right side) concrete weir and on the island and left side of the structure earth fill Dam, on the rock foundation (right side) concrete structure and on the island and left side of the structure earth-rock fill structure and Entire weir structure which is proposed by WWDSE.Hence after design of all the three alternatives of the structures and analysis using Geo-Studio software the results indicate that for the embankment part of the structures, the first alternative was selected as the most economical, functional and practical plausible alternative.

1 INTRODUCTION

The primary purpose of a diversion structure may be defined as to provide for the safe retention and storage of water. As a corollary to this every structure must represent a design solution specific to its site circumstances. The design therefore also represents an optimum balance of local technical and economic considerations at the time of construction.

It is also important to recognize that many major dams are now necessarily built on less favorable and more difficult sites. For obvious reasons, the most attractive sites have generally been among the first to be exploited. A proportion of sites developed today would, in the past, have been rejected as uneconomic or even as quite unsuitable for a dam. The ability to build successfully on less desirable foundations is a reflection of advances in geo-technical understanding and of confidence in modern ground-improvement processes.

For any storage and flood control dam under design, construction or operation, the foundation of the dam is a critical structural component and requires a special attention throughout the life time of the project. The foundation has a threefold function, namely to provide stability, to provide sufficient stiffness for limiting deformations within acceptable behavior patterns under the weight of the dam and the force acting on it and to provide control of seepage both inflows through adequate water tightness and principally in uplift and erosive stresses through good filter drainage details. If one of the performance of the dam risks impairment, even to the extreme of being unsafe, or suffering failure. The foundation conditions at a site are a determining factor in the selection of the type of dam [18].

Designing each Dam is preoccupied with generating alternative technical solutions that can meet intended functional objectives. The process of choosing among potential technical alternatives requires not only a detailed analysis of technological aspects but also critical evaluation of design conformance against specifications (geological, environmental, economic, social, political criteria etc.). Moreover, assessments have to be made on whether there is conformance with safety standards and whether an optimal balance is set between safety and economy. Such an assessment is vital in design of proposed dams and in making intelligent dam upgrading and rehabilitation decisions.

1.1 Background

Dam, reservoir and the foundation on which it rests is a complex structure comprising of a rock mass which inevitably undergo changes with time. Some of these changes are slow and subtle and don‟t reveal their existence unless precisely and constantly monitored.

The suitability of the various types of rock and soil as foundation and construction materials are geologic questions that must be considered. The foundation geology at a dam site often dictates the type of dam suitable for that site. The strength, thickness, and inclination of strata; permeability; fracturing; and faulting are all important considerations in selecting the dam type. Some of the different foundations commonly encountered are Rock Foundations, Gravel Foundations, Silt or Fine Sand Foundations and Clay Foundations. Occasionally, situations occur where reasonably uniform foundations of any of the types described above cannot be found and where a non-uniform foundation of rock and soft material must be used if the dam is to be built. Nevertheless, such conditions can often be counterbalanced by special design features. Even dam sites that are not highly unusual present special problems requiring the selection of appropriate treatment by experienced engineers. The study and design of Omo Kuraz Sugar Factories Project was done at a cost of 118.3 million Br by the state owned EWDSE in collaboration with SEDSE, which will have a 40pc share in the project.

From study of this project the major findings of the geological, geotechnical and geophysical studies and the observation from the site relevant to the foundation are the following:

- There are two clear faults identified along the river flow axes.
- A pervious gravel formation with cobles and pebbles is identified at the center part (the small island).
- A sandy-silt formation which is a result of complete disintegration of rhyolite exists.
- The competent Aphanatic basalt appears at very shallow depth.
- The weathered Rhyolite is found at center and near surface at the right bank.

The research attempted to address the problem faced during diversion structure design on challenging foundation condition.

Through the use of different alternatives different design approaches and updated design procedures it attempts to conceive a more transparent, comprehensive and precise result of diversion structures design. Hence the existing situation of the project is discussed in the next paragraphs to see the gap and to fill the gap using this thesis result.

In 2011, water works design and supervision Enterprise issued 3 volumes of a set of preliminary design documents under the title “Kuraz sugar development project”, including: Headwork design, Geological Geotechnical investigations, Preliminary design of headwork and Climatology, Hydrology and Drainage study.

Earlier, in this year , a free over flow concrete weir is selected and the weir section was designed on the basis of geological cross-section presented at weir axis, which was indicating the availability of good quality rock starting from the centre point of island and extending towards the right bank of the river;The geo-technical investigations were not possible on the island as well as on the right Omo channel and on the right bank due to accessibility problems; The weir structure was shifted towards right bank on its own axis, assuming that the whole weir structure will be rested on rocky foundation. The left abutment was moved to approximately at the centre of the island so that even the left abutment should rest on rocky foundation including the whole weir structure; the hydraulic as well as structural design of the weir structure was carried out assuming that whole weir structure will be resting on rocky foundation and the relevant drawings and tender documents were prepared and the over flow length of the ogee shaped weir section was 322 m and the abutment to abutment length was 386m. There were two silt excluders under sluices, one on each flank of the river.

In late 2012 WWDSE issued a revision and integration of the previous project documents, including Design documents of the main structures together with some sectorial studies: cofferdam and temporary closure, right side spill channel, climatology and hydrology, geothecnics and seismicity, material investigation, headwork hydraulic design criteria, Structural and foundation design and Hydrology final draft report. When the approach was possible on island as well as on the right bank of the Omo River due to construction of Coffer Dam.

The geo-technical investigation was carried out on island as well as on the right bank; On the basis of recent geo-technical investigations, the new geological cross-section at weir axis was provided to the design team in August 2012, which indicates the presence of 14m deep river deposited gravel material on the island which was earlier assumed as good quality rock. The gravel deposit extends up to the right side edge of island; Therefore, the design team finally decided to get the over flow weir structure back to its original location (placing the left abutment on its original location on the left bank of Omo River) on the same axis; In such a condition, about 250 m foundation length of the left side weir structure rests on the stiff silty clay (heterogeneous type strata below river bed) and on gravel deposits and the remaining 150 m right side length of the weir structure rests on the good quality rock; From the primary geological investigation, it appears that there are two major fault locations, approximately at the perpendicular direction of the weir axis. Its actual opening width and depth has not been identified so far, it can be assessed properly during construction;

In the above circumstances, the foundation design team has decided to transfer the load of weir, divide wall, U/S and D/S wing walls on the hard rock strata through piles, particularly for the left side 250 m portion of the weir and the remaining right side length (about 150m) of the weir will be resting on the rocky foundation strata; Therefore, in the above scenarios, the design team has revised the hydraulic design, structural design as well as foundation design of the weir structure.

Between September 2011 and May 2012 the local division of WWCSE built a large cofferdam daming the Omo River upstream of the original weir axis. The impressive cofferdam is 23m wide and approximately 800m long provided downstream 2 berms. The cofferdam is being progressively jeopardized and requires urgent measures to reinforce and protect the embankment from failure.

The design of Kuraz headwork was considered requiring significant changes in view of the definitely unfavorable foundation conditions disclosed under along the headwork axis. Many of the existing engineering solutions proposed were inadequate and unfeasible. Additional investigations and a radical restructuring of the project were considered necessary to achieve acceptable design, attaining an overall safety of the works, a better costs-benefit ratio.

1.2 Problem Statement and Research Objectives

1.2.1 Problem Statement

The main problem statements of this research are the result from the beginning of main headwork site investigation of Kuraz project. The followings are the main problems:

- The foundation stratification\formation show clear distinction in vertical level of appearance of a given material due to the vertical drops created possibly by the fault movement.
- The Head work (weir) is a very long structure extending through for about 350m over different formation at shallow depth rest on erratic sub-surface formation (issue of differential treatment).
- The headwork structure has to bridge over two clear distinct faults within its longitudinal span axis.
- The structure‟s immediate foundation is resting on alternating strong and weak formations with different bearing capacity exerting differential pressure.

The existence of the sandy silt formation at different levels along the longitudinal axis creates

- A weaker bearing capacity formation
- A settlement prone soil,
- A potential for differential settlement as they are not uniform through the axis of the structure and this soil is found at different depths.
- A liquefaction prone soil during an Earthquake.
- The two distinct faults and the upper gravel could lead to substantial loss of water undermining the function of the headwork.
- The two faults may also lead to instability by exerting hydrostatic and hydrodynamic pressures as the water head increases as function starts and consequent pressure is increased.

1.2.2 Research Objectives

Major Objective

The major objective of this research is to study the design and analysis of headwork options on a challenging foundation condition especially solving the problem on Kuraz Project on Omo River by providing technically, economically and functionally plausible alternative solutions.

Specific Objectives

In order to achieve the main objective of the study, the following specific objectives are set for major milestones of the study.

- Observing the Hydrologic conditions on the headwork site.
- Selection of headwork typology by giving especial consideration to the challenging foundation condition.
- Fixing the Hydraulic Design (structural dimension)
- Application of state-of-the-art computer soft wares in the analysis,
- Creative foundation design and headwork configuration (alternatives generated).

1.3 Organization of the Thesis

Apart from the introduction in chapter 1, the fundamental theories and principles of design and Analysis of headwork on challenging foundation conditions are laid in chapter 2. Towards the end of this chapter extensive review is given.

Chapter 3, 4 and 5 focuses on the methodology used, over view of the hydrology part of Omo river basin including the identification of important data for design and analysis of kuraz project diversion structures alternatives and construction materials investigation and the identification of haulage distances of different embankment materials which are available in the diversion site respectively.

Chapter 6 Focuses on selection, Hydraulic design and Structural dimensions for different alternative structures, that is Fixing structural dimensions for different alternatives based on hydrologic, topographic and the existing challenging conditions on the diversion sites.

Chapter 7 and 8 Deals with the data used in the design and analysis of different alternative structures and the design of creative foundation and the analysis of alternate structures using Geo-Studio software.

Chapter 9, Focuses on the selection of best alternative structures for the existing challenging foundation condition after focusing on economical, technical and Functional evaluation. Finally, in chapter 10 conclusions and some suggestions for future research are presented.

The thesis has appendices giving summary of some basic design results, and engineering properties of materials. In the main body of the thesis reference to the different sections of the appendix will be made as appropriate.

2 LITERATURE REVIEW

2.1 Introduction

Although a variety of dam types may be suitable for a potential site, a thorough examination of the relevant factors and issues will reveal which type will best achieve an acceptable balance between cost and safety, while still fulfilling the intended purpose of the proposed dam. While there are not many sites where the construction of a safe and functional dam is completely unfeasible from a construction standpoint, the physical characteristics of a particular site may make a potential project too costly. Some potential dam sites may face a challenging foundation conditions which may require an extensive geotechnical investigation to properly assess local conditions. This is likely to occur in cases where previous geotechnical documentation does not exist or many unknowns are yet to be determined. Other sites, however, may only require a few borings to verify local conditions that are generally known or well documented. Moreover, different types of proposed dams will call for different levels of detail from the geotechnical investigation. For instance, when designing the foundations and abutments for large and intermediate sized and high hazard concrete dams or concrete appurtenant structures, an in depth comprehensive geotechnical investigation is vital. The foundations and abutments for embankment dams, however, do not typically require geotechnical determinations that are as extensive as those required for concrete dams.

The quality of a properly prepared geotechnical investigation should be based on the type of dam being proposed or the particular dam rehabilitation or modification being proposed, in conjunction with the geological complexity at the site. The geotechnical investigation can sometimes extend beyond the design phase of dam improvements, as the construction phase may present unforeseen geological problems. When this occurs, construction is often delayed for proposed dams; you should conduct an initial investigation to assess the general suitability of the site for a particular dam type and size.

During earth quake fault movements in the dam foundation or discontinuities in dam foundation near major faults which can be activated during strong nearby earthquakes causing structural distortions. Fault displacement in the reservoir bottom causing water waves in the reservoir or loss of freeboard.

In similar case San Andreas and Coyote Dam Dam located on a fault and has direct damage caused by the movement of the plates because of Great San Francisco Earthquake of 1906.The Cedar Springs Dam in California was built over active faults. Because of the faults, the original design of the dam was modified including a reduction in height, shifting of the axis, and changes in construction material zonation [30]. The new Saluda Dam, near Columbia, SC, USA is a traditional gravity section, making it the second largest modern RCC Dam in the USA. It is founded on folded and faulted gneiss and granite, with intense weathering in zones to depths as deep as 10 m below the dam/rock interface [28].

On 5 June 1976, the Teton dam, located on the Teton River in Idaho, US, failed, releasing a wall of water that was reported to be 22.9m high. Taking this as an indication of water seepage, the face of the dam was examined. When water enters defective zones of the fill or its foundation, some embankment zones could be subject to weathering and differential settlement at zone interfaces [29].

Liquefaction of fill in the dam may occur. Liquefaction is the large drop in stiffness and strength of soil due to seismic movements (Byrne and Seid Karbasi 2003). As a result, part of a dam may slump and slideoff the structure. During the Santa Barbara Earthquake of 1925, the Sheffield Dam in California failed due to liquefaction apparently in the area just below the dam embankment containing silty sand. Little damage and no loss of life resulted. The earthquake had a magnitude of 6.3 and the dam was constructed in 1917 [29].

The question that remains is thus: what type of structure and how to design this structure for the existing challenging foundation conditions. This chapter gives brief theoretical overview and concepts towards answering these problems. The succeeding chapters‟ present details of selection of diversion structure based on the existing conditions, design of alternatives and analysis procedures.

2.2 Processes of Designing and Analyzing Headwork on Challenging Foundation

2.2.1 Small Embankment Dams

The first step to design and analyze a small embankment dam is collecting the necessary Hydrologic and geology data of the headwork including the challenges in the foundation. For small-sized embankment dams, determining the feasibility data may only necessitate a review of maps (soil survey, topographic, geological, river survey, aerial photography, etc.),well and spring data, geological surveys or investigations, construction records of nearby structures (highway or railroad cuts, building excavations, soil pits, rock quarries, etc.).

After collecting the necessary site data the hydraulic and structural design and analyzing of this dam on challenging foundation conditions can be done using analytical methods and using soft wares by considering the effect of dispersive soils, Collapsible soils (slaking shales, gravelly materials, etc.), sands susceptible to liquefaction, differential settlement etc. And when there is geological discontinuities like faults, folds etc. it should be treated using especial techniques.

2.2.2 Concrete Dams and Intermediate and Large Embankment Dams

Before design and analysis of a selected head work type collecting the necessary hydrologic and geologic data including the challenges in the foundation are inputs for design and analysis. Unlike the design of small embankment dam, concrete dams and intermediate and large sized embankment dams they need a more extensive initial feasibility assessment, with subsurface exploration (numerous, s and comprehensive borings, fully representative of the site) to evaluate additional considerations, including: The depth to bedrock (if applicable),The bedrock profile along the axis of the proposed dam ,The classification and integrity of geological strata beneath the proposed dam and appurtenant structure including possibly during past improvements.

After collecting the necessary site data the hydraulic and structural design and analyzing of this dam on challenging foundation conditions can be done using analytical methods and using soft wares By considering the effect of challenges in the foundation.

2.2.3 Foundation Examination and Geotechnical Considerations

If geological issues or abnormalities are known to exist at a dam foundation location (valley floor and abutments), or should an initial geotechnical investigation uncover problematic concerns for a foundation, you must determine if an engineered remedy is feasible.

The foundation requirements for design of the dam will vary based upon the foundation conditions, geology, rock jointing and faulting, dam size and use of the reservoir. Here are the general foundation competence requirements and the Kuraz situation in a comparative form.

Table 2.1: Foundation general competence requirements and the Kuraz situation in a comparative form.

Foundation general competence requirements	Kuraz situation
Suitable, consistent and competency foundation under the dam.	Cannot obtain a suitable and consistent foundation material extending uniformly at shallow depth.
Free from differential settlement	Exposed to differential treatment.
Free from geological formations and discontinuities	Two clear distinct faults within its longitudinal span axis.
Good foundation Strength, free from sliding, liquefaction, stability problem and poor permeability characteristics	Alternating strong and weak formations with different bearing capacity exerting differential pressure creating overstress in the structure.
Free from liquefaction prone soils during an Earthquake.	Existence of the sandy silt formation at different levels along the longitudinal axis creates A liquefaction prone soil during an Earthquake.

If a foundation vulnerability cannot be eliminated or treated to an acceptable safety standard, either the proposed dam should not be built (at the specific location), or the existing dam should be removed or intentionally breached (if a downstream threat exists).

Geotechnical considerations extend beyond the design phase of a dam improvement project and into the construction phase. But from the geological study of Kuraz project the cross section of the foundation is shown below in figure 2-1.

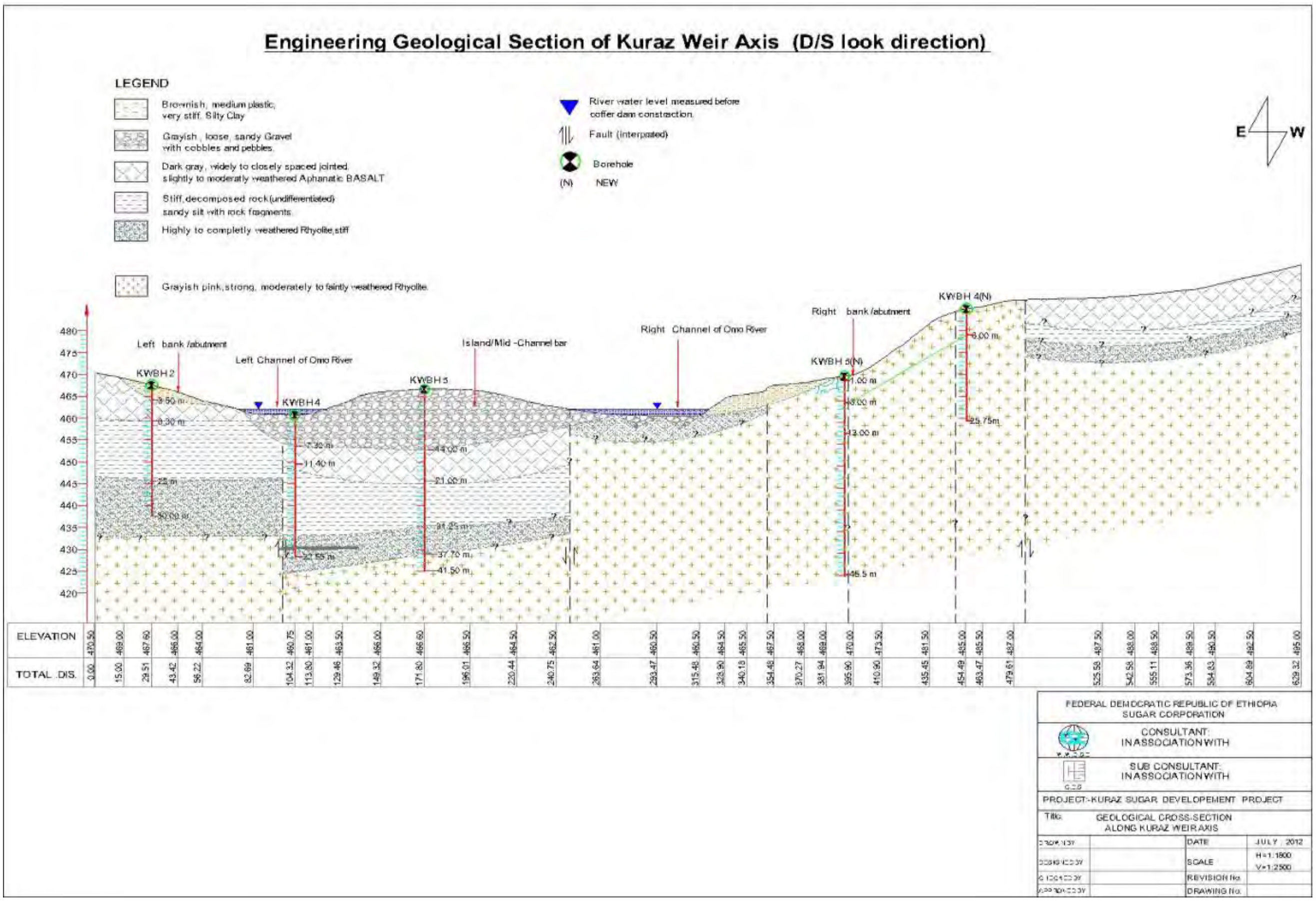

Figure 2-1. Engineering geological section of kuraz weir site

Ultimately, when a concrete dam (other than a small, low-hazard structure) is proposed, or when a construction project includes significant foundation treatment, a geotechnical professional needs to be involved during the initial phase of construction to verify site conditions and confirm the suitability of the geotechnical design recommendations previously established.

2.3 Types of Diversion Structure

A diversion structure is required for diverting the water from the river into the Canal system. This can be a Dam with storage reservoir, a Weir/Barrage type diversion structure or river intake type of diversion structure. The choice of a suitable type depends on various factors such as topographical condition of the diversion site and the command area, foundation and geological condition, hydrology, morphological condition of the river etc.

Broadly there are three types of head works:

- Storage with a dam which may be an earthen or concrete/ masonry dam;
- Weir type, which may again be a weir or a barrage and
- River intake type diversion structure

Following is a brief overview of the most common diversion option types that will use based on the site condition.

2.3.1 Embankment Dams

Embankment dam can be designed with a fair degree of theoretical accuracy, provided the properties of the soil placed in the dam, are properly controlled. This condition makes the design and construction of such dams, thoroughly interdependent [4] continuous field observations of deformations and pore water pressures have to be made during the construction of such dams. Suitable modifications in the design are then made during construction, depending upon field observations.

The embankment dam possesses many outstanding merits which combine to ensure its continued dominance as a generic type. The more important can be summarized as follows:

- The suitability of the type to sites in wide valleys and relatively steep sided gorges.
- Adaptability to a broad range of foundation conditions, ranging from competent rock to soft and compressible or relatively pervious soil formations.
- The use of natural materials, minimizing the need to import or transport large quantities of processed materials or cement to the site.
- Subject to satisfying essential design criteria, the embankment design is extremely flexible in its ability to accommodate different fill materials.

- The construction process is highly mechanized and is effectively continuous.
- Largely in consequence of 5, the unit costs of earth fill and rock fill have risen much more slowly in real terms than those for mass concrete.
- Properly designed, the embankment can safely accommodate an appreciable degree of settlement deformation without risk of serious cracking and possible failure.

The relative disadvantages of the embankment dam are few. The most important include an inherently greater susceptibility to damage or destruction by overtopping, with a consequent need to ensure adequate flood relief and a separate spillway, and vulnerability to concealed leakage and internal erosion in dam or foundation.

But from the above merits for this particular project on the island and left side (on week part of the foundation) of the structure earth fill dam or earth fill-rock fill dams are taken as an alternatives.

2.3.2 Concrete Dams

- Concrete dams include gravity, arch, buttress, masonry, and roller-compacted concrete (RCC) dams. Concrete dams:
- Are best suited for in-channel overflow structures, as well as narrow gorges.
- Are less susceptible to erosion.
- Rely on the weight of the structure and/or the strength of the bond or anchor at the abutments.
- Require solid impervious strata for an adequate foundation (extensive geotechnical investigation is critical).

Because of the above facts taking Construction of concrete dam on Kuraz project as an option is inadequate and unfeasible. And requiring significant changes in view of the definitely unfavorable geological conditions disclosed under along the dam axis.

2.3.3 Composite Dams

Combining elements of several dam types, composite dams typically utilize an earthen embankment for the non-overflow portion of the dam and concrete for overflow spillways and/or specialized structures.

This possibility gives an option for constructing a composite structures for the case of Kuraz project as shown below

- On the rock foundation (right side) concrete weir and on the island and left side of the structure earths fill Dam.
- On the rock foundation (right side) concrete structure and on the island and left side of the structure earth fill-rock fill structure.
- Using the cofferdam as a permanent structure by constructing an overflow section in either of the spill channels (left or right).

For using these options the very challenging issue here is the interface between the earth and concrete part. The followings are the options we have

Concrete cutoff walls: these walls are sometimes used to provide positive seepage cutoff through embankments and/or pervious foundations of rock or soil. The bottom of the concrete cutoff wall should be founded in an impervious stratum. Concrete cutoff walls are formed by constructing cast-in-place or precast concrete panels in a serious of slurry during excavation and prior to concrete placement. The tremie method is used to place concrete in the trench from bottom to top, displacing the slurry. the workability and flow ability of concrete is the most important property in constructing a high quality precast concrete cutoff reinforcing steel may be necessary.to be effective the cutoff walls must be fully penetrate pervious strata [8].

Geo membranes: Geo membranes are composed of low permeability materials. Such as Geotextiles are used in a variety of applications in embankment dam construction and rehabilitation. The case of geo membrane alone also applied to a geo membrane placed on a rigid material such as concrete which has an irregular surface, which may crack, or which may undergo differential settlement.

A better solution as suggested by Giround and Huol (1977): Giving a convex shape to the embankment next to the concrete structur, placing a low friction geotextile between the geomembrane and the embankment and constructing the concrete structure with a batter. Although policy varies, most practitioners in the United States limit the use of geotextiles to locations where there is easy access for repair and replacement (shallow burial), or where the geotextile function is not critical to the safety of the dam should the geotextile fail to perform.

It is the policy of the National Dam Safety Review Board that geotextiles should not be used in locations that are critical to the safety of the dam. Because geotextiles are prone to installation damage and have a potential for clogging, their reliability remains uncertain. If geotextiles are not designed and constructed correctly, embankment dams may have an increased probability of failure. The particular design requirements and site conditions of each embankment dam and geotextile installation are unique. No single publication can cover all of the requirements and conditions that can be encountered during design and construction. They must be designed and approved by engineers experienced with all aspects of the design and construction of these structures. But absorption of components of the polymeric compound, shrinkage of materials in contact, Algae contact, chemical in water and abrasion may cause surface cracking of the geomembrane [31].Based on the reasons above for this particular project it is better to adopt a concrete wall cutoff.

2.3.4 Weir/barrage structure

A weir can be defined as a barrier with crest provision to raise the water level in order to take it by gravity to an area for irrigation development. A weir structure is generally constructed across the river for diversion of water into the canals, Raising of water level to feed canals, Storing the excess flow received from upstream storage sites and Regulation of desired supplies.

2.4 Factors Affecting the Selection of a Diversion Structure Type

While some proposed dam types may be determined according to a specific intended purpose or function, the majority of proposed dams will have their type determined by the following considerations.

2.4.1 Economic Considerations

One of the factors in selecting a dam type is economics. Some of the conditions that can result in extensive costs are: Its location and accessibility. For some sites, the long distance to adequate Construction materials (soils for embankments, rock for embankments and/or riprap, concrete aggregate, etc.) and/or the lack of availability of labor equipment can result in extensive costs, Need for erosion protection. For some sites, unfavorable topography and/or highly erodible soils can result in extensive costs, Need for care and diversion of water.

For some sites, where the local subsurface Conditions are unknown, or for some dams (including most concrete dams of Considerable size and/or having a high-hazard classification), a comprehensive Geotechnical investigation may be required, and can result in extensive costs.

Hence, the final selection of the type of dam should be made only after careful analysis and comparison of possible alternatives, and after thorough economic analyses that include costs of structures and foundation treatment.

2.4.2 Geological Considerations

Another consideration in selecting a dam type is the geological characteristics of the dam site. Generally competent foundation, geological formations and discontinuities, liquefaction prone soils during an Earthquake, stability problem, permeability characteristics and differential settlement are the major factors that should be considered.

The composition of underlying or adjacent geological strata (rock, gravel, silt, sand, clay, etc.) at the site, which determines: The site"s ability to adequately support the foundation and anchor abutments of a proposed dam type. The site"s general suitability. If the geological strata are not uniform, consisting of both rock and soft materials, or if dissolvable material such as gypsum is present, you might need to consider and evaluate special improvements.

2.4.3 Existing foundation condition

The geology and foundation conditions at the dam site may dictate the type of dam suitable for that site. For the design of composite structure depending on the existing foundation conditions shown above it is better to decide which composition of the structures is best. The selection is done based on the following guides of standards of small dam design.

Rock Foundations:- Competent rock foundations, which are free of significant geologic defects, have relatively high shear strengths, and are resistant to erosion and percolation, offer few restrictions as to the type of dam that can be built upon them. The economy of materials or the overall cost should be the ruling factor. The removal of disintegrated rock together with the sealing of seams and fractures by grouting is frequently necessary.

Weaker rocks such as clay shales, some sandstones, weathered basalt, etc., may present significant problems to the design and construction of a dam and may heavily influence the type of dam selected.

Gravel Foundations: - Gravel foundations, if well compacted, are suitable for earth fill or rock fills dams. Because gravel foundations are frequently subjected to water percolation at high rates, Special precautions must be taken to provide adequate seepage control and/or effective water cutoffs or seals. Also, the liquefaction potential of gravel foundations should be investigated.

Silt or Fine Sand Foundations:-Silt or fine sand foundations can be used for low concrete gravity dams and earth fill dams if properly designed, but they are generally not suitable for rock fill dams. Design concerns include non-uniform settlement, potential soil collapse upon saturation, uplift forces, the prevention of piping, excessive percolation losses, and protection of the foundation at the downstream embankment toe from erosion.

Non dispersive clay foundations may be used for earth dams but require flat embankment slopes because of relatively low foundation shear strength. Because of the requirement for flatter slopes and the tendency for large settlements, clay foundations are generally not suitable for concrete (or roller compacted concrete) or rock-fill dams (Golze 1977, Bureau of Reclamation 1984).

Non-uniform Foundations:-Occasionally, situations occur where reasonably uniform foundations of any of the types described above cannot be found and where a non-uniform foundation of rock and soft material must be used if the dam is to be built. Nevertheless, such conditions can often be counter balanced by special design features. Even dam sites that are not highly unusual present special problems requiring the selection of appropriate treatment by experienced engineers.

2.4.4 Location, Topography, and Soil Considerations Materials available

Another consideration in selecting a dam type is the physical features of the site. The topography and soils characteristics of a site, which determine its suitability for a particular dam type.Topography, to a large measure, dictates the first choice of type of dam. A narrow V-shaped valley with sound rock in abutments would favor an arch dam.

A relatively narrow valley with high, rocky walls would suggest a rock fill or concrete dam (or roller-compacted concrete). Conversely, a wide valley with deep overburden would suggest an earth dam. Irregular valleys might suggest a composite structure, partly earth and partly concrete. Composite sections might also be used to provide a concrete spillway while the rest of the dam is constructed as an embankment section.

Hence, Site conditions that may lead to selection of an earth or a rock-fill dam rather than a concrete dam (or roller-compacted concrete dam) include a wide, stream valley, lack of firm rock abutments, considerable depths of soil overlying bedrock, poor quality bedrock from a structural point of view, availability of sufficient quantities of suitable soils or rock fill, and existence of a good site for a spillway of sufficient capacity.

The most economical type of dam will often be one for which materials can be found within a reasonable haul distance from the site, including material which must be excavated for the dam foundation, main structure, and other appurtenant structures. Materials which may be available near or on the dam site include soils for embankments; rock for embankments and riprap, and concrete aggregate (sand, gravel, and crushed stone).Materials from required excavations may be stockpiled for later use. However, greater savings will result if construction scheduling allows direct use of required excavations.

2.4.5 Environmental and The ecological impact of the proposed structure

Recently environmental considerations have become very important in the design of dams and can have a major influence on the type of dam selected. The principal influence of environmental concerns on selection of a specific type of dam is the need to consider protection of the environment, which can affect the type of dam, its dimensions, and location of the spillway and appurtenant facilities.

The specific structure type may need to accommodate the demands of local wildlife; for example, it may need to provide transit means for migratory aquatic species through or around the dam, requiring the construction of specialized components.

2.5 Spillway

The flood hydrograph leaving the reservoir that is used to design and/or modify a specific dam and its appurtenant works (also referred to as the outflow hydrograph). The SDF is estimated by routing the appropriate IDF through a dam‟s spillway, outlet works, and attendant surcharge storage.

In addition to design flood selection, the accommodation of the IDF is critical to the hydrologic safety of a dam. IDF accommodation includes the consideration of spillway type, flood routing design criteria, and freeboard criteria. Spillways and flood outlets should be designed to safely convey major floods to the water course downstream of the dam. They are selected for a specific dam and reservoir on the basis of dam safety, dam type and purpose, release requirements, topography, geology, project economics, and other possible factors.

2.5.1 Controlled versus Uncontrolled Spillways

By definition, an uncontrolled spillway releases water whenever the reservoir elevation exceeds the spillway crest level. Conversely, a controlled spillway can regulate releases over a range of water levels through the use of gates and/or valves. Each of these spillway types has specific design implications which should be considered when designing a spillway.

The selection of a controlled or uncontrolled type of spillway for a specific dam depends on site conditions, project purposes, the magnitude of the IDF, economic factors, costs of operation and maintenance, and other considerations. The Discharge capacity, Project objectives and flexibility, Operation and maintenance , Reliability , Data and control requirements, Emergency drawdown , Economics influence the use of either a controlled or uncontrolled spillway [32]:

Discharge capacity – For a given spillway crest length and maximum allowable water surface elevation, a controlled spillway can be designed to release higher discharges than an uncontrolled spillway if the spillway crest elevation is lower than the normal reservoir storage level. This can impact spillway design selection when there are limitations on spillway crest length or maximum water surface elevation.

Project objectives and flexibility – Controlled spillways permit a wide range of releases and have capability for pre-flood drawdown.

Operation and maintenance – Uncontrolled spillways are typically more reliable and self-maintaining than controlled spillways. Controlled spillways may experience more operational problems and are more expensive to construct and maintain than uncontrolled spillways. Constant attendance or several inspections per day by an operator during high water levels is highly desirable for reservoirs with controlled spillways, even when automatic or remote controls are provided. However, access to the dam during a major flood event might be difficult or even impossible. Controlled spillways require regular maintenance and periodic testing of gate operations.

Reliability – The nature of uncontrolled spillways reduces dam failure potential associated with improper operation and maintenance. Where forecasting capability is unreliable, or where time from the beginning of runoff to peak inflow is only a few hours, uncontrolled spillways are more reliable, particularly for high hazard potential structures. Consequences of failure of operation equipment or errors in operation can be severe for controlled spillways. Susceptibility to plugging due to debris can also impact the reliability of both controlled and uncontrolled spillways.

Data and control requirements – Operational decisions for controlled spillways should be based upon real-time hydrologic and meteorological data to make proper regulation possible.

Emergency drawdown – Typical uncontrolled spillways cannot be used to evacuate a reservoir during emergencies. The capability of controlled spillways to draw down pools from the top of the gates to the spillway crest can be an advantage when rapid reduction of load on the dam is required.

Economics – Economic considerations often influence whether controlled or uncontrolled spillways are selected. Controlled spillways are typically more expensive than uncontrolled spillways.

Final selection of the type of crest control should be based on a comprehensive analysis of all pertinent factors. The size, type, and restrictions on location of the spillway are often controlling factors in the choice of the type of dam. When a large spillway is to be constructed, it may be desirable to combine the spillway and dam into one structure, indicating a concrete overflow dam. In the case of kuraz project the Head Work Structure Draft Hydraulic Design Report in 2012 the ungated spillway structure is provided in combination with the over flow concrete weir.

2.10 Major Causes of Diversion Structures Failure

Diversion structure design principles and considerations evolved with the identification of major causes of dam failure and the progressive understanding of their mechanisms. In addition, knowledge on causes of dam failure is crucial for dam safety evaluation, dam monitoring and rehabilitation decisions. There are varying statistics on causes of dam failure, for example statistics given by ICOLD, USACE and Novak et al. (2003) are given in Table 2.2. Many attempts have been made at compiling and assessing statistics on dam failure. Main attempts on worldwide scale have been made by International Commission on Large Dams (ICOLD) in 1974, 1983 and 1995. ICOLD (1995) states that foundation problems are the most common causes of failure in concrete dams, with internal erosion and foundation shear strength each contributing for 21%. In case of earth and rock fill dams, the most common cause of failure is overtopping (31% as primary cause and 18% as secondary cause) followed by internal erosion in the body of the dam (15% as primary cause 13% as secondary cause), and in the foundation (12% as primary cause and 5% as secondary). Foundation defects includes Slope instability, differential settlement, high uplift pressure, foundation seepage, whereas internal erosion Piping and seepage.

Table 2.2: Statistics on causes of dam failure.

Over topping	Foundation defects	Internal erosion	Others	Source
34%	30%	20%	6%	[19]
30-35%	No data	30-35 %		[20]
31% primary cause 18% secondary cause	No data	27% primary cause 18 % secondary cause		[21]

The following principal defect mechanisms and failure modes for embankment dams [1]:

Overtopping leading to washout: spillway and outlet capacity must be sufficient to prevent overtopping. Also there should be sufficient freeboard to prevent overtopping by wave action. The freeboard must also include an allowance for the predicted long-term settlement of the embankment, foundation compressibility and sedimentation. Overtopping has risk of serious erosion and possible washout of embankment.

Internal erosion and piping with migration of fines from core and foundation: regression of „pipe" and formation of internal cavities, may initiate by internal cracking or by seepage along culvert perimeter. Seepage within and under the embankment must be controlled to prevent concealed internal erosion and migration of materials. Hydraulic gradients, seepage pressures and seepage velocities within and under the dams must, therefore, be contained at levels acceptable for the materials concerned. Care must be taken to ensure that outlet or other facilities constructed through the dam do not permit unobstructed passage of seepage water along their perimeters with risk of soil migration and piping.

Embankment and foundation settlement (deformation and internal cracking): care must be taken with soft compressible foundations and proper compaction has to be done during construction of dams.

Instability: the embankment, including its foundation, must be stable under construction and under all conditions of reservoir operation. Instability might occur when downstream slope too high and/or too steep in relation to shear strength of the shoulder material or when there is rapid drawdown of water level or because of failure of downstream foundation due to overstress of soft, weak horizons. Face slopes must, therefore, be sufficiently flat to ensure that internal and foundation stress remains within acceptable limits under different conditions of loading.

2.11 Theories related to Software analysis methods

Geo-Studio 2004 is a new integrated tool for running GEO-SLOPE‟s leading suite of geotechnical modeling software products: SLOPE/W, SEEP/W, SIGMA/W, QUAKE/W, CTRAN/W, TEMP/W, and VADOSE/W. Analyzing the seepage and stability of earth structure is the oldest type of numerical analysis in geotechnical engineering. Even to this day, stability analyses are the most common type of numerical analysis in geotechnical engineering this is in part stability is obviously a key issue in any project. Modern limit equilibrium soft wares like Geo Studio is used for this research and making it possible to handle ever increasing complexity within any analysis.

2.11.1 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 in the dam. In order to design the filter thickness, seepage analyses through the dam only and through the dam and the foundation have been conducted using the state of the art Finite Element Method based computer program Seep/W from Geo-slope international, 2004. In order to design the chimney and horizontal filter thicknesses, seepage analyses through the dam only and through the dam and the foundation have been conducted.The computed discharges are used to calculate the Filter thicknesses. Besides, the computed phreatic surface is used to set up the pore water pressure line in the stability analysis of the dam.

2.11.2 Methods of 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 stability of this Dam is analysed using state of the art software Slope/W from Geo-Slope.The slope stability investigation was carried out using the Slope/W computer program based on the limit equilibrium method and the Morgenstern-Price method was used to obtain the factors of safety. This particular method has been adopted because, unlike Fellenius or Bishop‟s or Janbu‟s methods, the Morgenstern-Price method satisfies 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.

Table 2.3: Factor of safety methods and satisfied equilibrium methods.

Method	Moment equilibrium	Force equilibrium
Ordinary or Fellenius	Yes	No
Bishop,s Simplified	Yes	No
Janbu,s Simplified	No	Yes
Spencer	Yes	Yes
Morgenstern-Price	Yes	Yes
Corps of Engineers-1	No	Yes
Corps of Engineers-2	No	Yes
Lowe-Karafiath	No	Yes
Janbu Generalized	Yes (by slice)	Yes
Sarma -Vertical Slices	Yes	Yes

The stability analyses had been conducted for the following loading and critical conditions in order to determine the factor of safety for various slip surfaces of:

- 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.

2.11.3 Stress analysis and settlement analysis

The initial in situ static stress conditions can come from a SIGMA/W analysis. Usually this is however, not necessary since QUAKE/W has its own procedures for establishing the initial stress conditions. The stress-deformation analysis of earth structures can be performed using SIGMA/W software which is a finite element software product. Its comprehensive formulation makes it possible to analyze both simple and highly complex problems. It can perform a simple linear elastic deformation analysis or a highly sophisticated nonlinear elastic-plastic effective stress analysis.

SIGMA/W can be used to compute stress-deformation with and without the change is in pore-water pressures that arise from stress state change. The major effects of deformations are loss of freeboard due to settlement, damage to appurtenant structures located within or upon the dam, loss of confidence in the dam due to swayback appearance, cracking of the embankment most detrimental to the impervious core, development of localized zones susceptible to hydraulic fracturing, and failure of instrumentation. The effects of deformation can usually be mitigated by designing features based on experience gained from studying historical performance of existing dams without the need for performing any elaborate analyses. Detailed attention to embankment zoning and foundation shaping can minimize differential settlements, thereby reducing the potential for cracking of the core or development of zones susceptible to hydraulic fracturing. The followings are the two major ideas that should be considered during analysis of dam using QUAKE/W.

Liquefaction

Liquefaction is a term that has been applied to different, but overlapping, phenomena that occur in loose sands and gravels subjected to cyclic loading. One phenomenon is slope failure caused by loss of shear strength during undrained shear of highly contractive, fully saturated zones. If the soil mass liquefies along a critical failure surface, and is unrestrained, the mass appears to flow when this type of catastrophic failure occurs. For this reason the phenomenon is referred to as a flow slide. The other phenomenon is cyclic deformation. Deformations caused by cyclic loading may or may not lead to failure of the dam. Once the seismic loading of the dam is determined, the general design approach is to assess if liquefaction may be triggered. Generally if liquefaction is triggered, the pore water pressures increase until the strength of the material drops to some residual value.

Loose, saturated sands are most vulnerable to liquefaction. Sands exhibiting low blow counts (from Standard Penetration Tests or Cone Penetration Tests done in borings), uniform gradations, and rounded grains are likely to be potentially liquefiable. The determination of liquefaction potential is independent of the stability analysis. The foundation design analysis shall consider an assessment of potential consequences of any liquefaction and soil strength loss including estimation of differential settlement, lateral movement or reduction in foundation soil bearing capacity, and may incorporate the potential benefits of any proposed mitigation measures.

In evaluating the potential for liquefaction, the effect of settlements induced by seismic motions and loss of soil strength shall be considered. The analysis performed shall incorporate the effects of peak ground acceleration, appropriate earthquake magnitudes and duration consistent with the design earthquake ground motions as well as uncertainty and variability of soil properties across the site. Peak ground acceleration, seismically induced cyclic stress ratios and pore pressure development may be determined from a site-specific study taking into account soil amplification effects and ground motions appropriate for the seismic hazard.

Damping ratio

The dynamic shear modulus and damping ratio of soils are two important parameters in soil property and affect earthquake ground motion greatly. The laboratory experimental results at present show a significant variation because of the soil complication. Studying the uncertainty of the dynamic shear modulus and damping ratio is quite helpful for understanding the nonlinear behavior of soils. Also, the quantitative results of the variability are one of basic elements of the reliability analysis for the ground motion and structure damage. However, the related research at present mostly concentrates on the static problem and the study on dynamics parameters especially nonlinear uncertainty problem is rare.

The dynamic modulus and damping ratio of soils are two important parameters in soil dynamics. They are indispensability for soil layer seismic response analysis and seismic safety evaluation of engineering sites. During the soil layer seismic response analysis, the difference of dynamic modulus and damping ratio will affect earthquake ground motion greatly. Over a wide range of inertia ratios the damping ratios range between 10 and 50% [2].

2.12 Economics of a project

2.12.1 Cost Estimation

Successful cost estimates use planning techniques to define the project objectives in sufficient detail to support the level of cost estimate being developed. The WBS process assists the project stakeholders in developing the elements of the project scope into hierarchical, manageable and definable work elements that balance the control needs of management with an appropriate and effective level of project data.

A well-developed WBS that presents information at the appropriate level of detail and in formats and structures meaningful to those estimating the work is an invaluable tool in overall project management.

Effective cost estimating involves the use of data derived from the most current pricing for materials, appropriate wages and salaries, accepted productivity standards, and customary construction practices, procurement methods, equipment needs, and site conditions. Cost estimates are by definition prepared with less than complete information and have inherent levels of risk and uncertainties.

The costs estimated for this project shown are estimates only. These estimates were developed from current cost in WWDSE, Earthen embankment Dam Preliminary opinion of cost for alternative assessment, Rib dam planning report and discussions with consulting engineers on the normal range of costs for dams that they have worked on and relate to this hypothetical scenario only. Actual costs may vary greatly from these estimates based on actual conditions.

2.12.2 Developing the cost estimate

Typically, a cost estimator will use one of the following approaches (or a combination of both) to develop the cost estimate:

Unit price (historical) estimates : are generally prepared using current unit prices. They are prepared using relevant previous bid abstracts, cost curves, catalogs, detailed analyses, vendor quotations, and regression analyses.

Detailed estimates : are built-up estimates representing hypothetical offeror‟s bid prices, including all direct costs and indirect costs (i.e., project overheads, business overheads, profit, and bonds) to perform the work required by the solicitation.

Both approaches can result in the same level of confidence in the cost estimate when used appropriately; however, each has its strengths and limitations depending on the level of estimate, the complexity of the project, and relative amount of labor costs versus material costs. Typically the built-up approach is used on major items that are variable and cannot be confidently quantified by unit prices.

An Indirect Project Costs are those not addressed in specific Work Breakdown Structure items as Direct Costs or recovered as General and Administrative Costs. They are generally associated with project-specific overhead costs that occur on the project. Indirect costs can generally run in the range of 8 to 20 percent of overall project costs. It is also important to understand the project schedule.

2.12.3 Quantities, Unit Prices, Bill of Quantities and Cost estimates

Quantities

When estimates are required before drawings have been prepared, shortcut methods of determining quantities are often used, such as methods where in the quantities are roughly estimated on the basis of unit quantities for similar structures. Except for appraisal estimates, the quantities for the major items should be obtained from actual layouts developed in sufficient detail for their computation.

The quantities of the different items composing the all the alternative structures and all other components necessary for comparing all the feasible alternative structures for the Project site conditions, such as the proposed embankment dams and concrete structures, etc, were calculated from the three design alternative dimensions, feasibility study plans, draft hydraulic design report etc.

Unit Prices

When estimating the unit price for labor, equipment, and materials required for construction, consideration must be given to numerous factors that influence the cost, such as: geographical location of work; seasonal weather conditions; unusual or special physical conditions; accessibility of the work; accommodations for housing and transportation; materials handling and storage facilities; sources of construction power; availability of labor and materials; types of labor required; wage rates; construction plant and equipment requirements; associated production rates for the anticipated crews; construction schedule; and economic trends.

The unit prices prepared for the project were derived from several sources to meet the definitions of the specific work in each of the sub clauses in Appendix 11.3.1 to 11.3.3, Bill of Quantities (BOQ):

Unit prices received from the Contracts Department of WWDSE

Unit prices from other projects where similar works were carried out

Unit prices generally practiced in Ethiopia

Unit prices from projects, not necessarily in Ethiopia, were similar specific works or parts needed for the project have already been priced.

2.12.4 Contingency

The total amount of contingency in an Engineer‟s Estimate should shrink as the project design increases in detail and level of understanding. As noted previously, uncertainty and, therefore, risk contingency dollars flow from contingency into existing and additional line items, quantity changes, hedging, insurance, and other definable items. Contingency in an Engineer‟s Estimate is a very real cost that represents the risk cost of uncertainty in a line item, subtotal, or total. This contingency evolves as the level of project detail and understanding increases as shown below in figure 2-2.

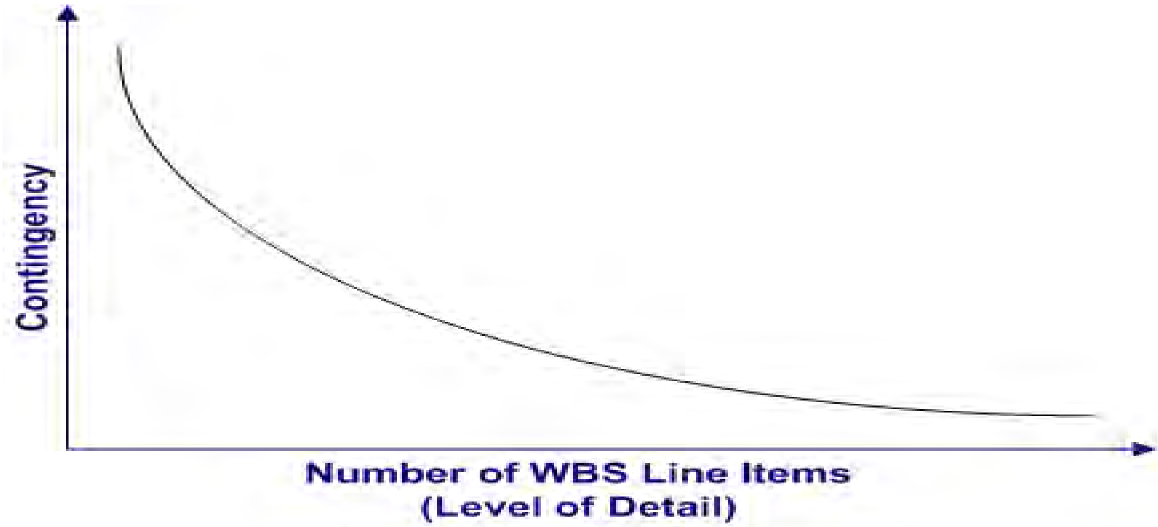

Figure 2-2. WBS Line Items vs. Contingency

According to AACE Civil works at this level of development should have a minimum contingency of 30 percent, and a minimum contingency of 10 percent should be applied to preliminary budgetary quotes from vendors.

2.12.5 Taxes

Tax codes can be very complex and vary with jurisdiction. Taxes can become a significant cost to the project and must be addressed, usually as a line item in the estimate. The cost estimator will need to research and provide information about what is taxed and what is exempt on the subject project, given its nature and location. Materials and supplies are generally subject to sales taxes. Under some state laws, permanent materials for the project are not subject to sales tax.

2.12.6 Bill of Quantities and Cost Estimate

The embankment dam possesses many outstanding merits which combine to ensure its continued dominance as a generic type. The use of natural materials, minimizing the need to import or transport large quantities of processed materials or cement to the site, subject to satisfying essential design criteria, the embankment design is extremely flexible in its ability to accommodate different fill materials, the construction process is highly mechanized and is effectively continuous, the unit costs of earth fill and rock fill have risen much more slowly in real terms than those for mass concrete. Completed unit costs for mass concrete, i.e. cost per cubic meter, and are very much higher than for embankment fills, typically by an order of magnitude or more. This is seldom counterbalanced by the much lower volumes of concrete required in a dam of given height.

The results of the quantifying and the pricing, conforming to the different work items of the implementation of the project, are presented in Appendix 11.3.1, 11.3.2 and 11.3.3.

3 METHODS, MATERIALS AND PROCEDURES

The methods that used to solve the problems in this thesis are selection of criteria‟s in order to generate possible alternatives. Based on the precondition criteria alternative engineering solutions are generated for the existing challenging foundation conditions. Basically, important data and parameters will describe from existing documents and laboratory test results of the project. But the seepage data through the dam and foundation was obtained from simulation result using SEEP/W software.

For each generated alternatives fixing the dimensions of the structure are done based on different design standards and codes. Then the design and analysis of each alternative structure was done using analytical calculation methods and Geo-Studio software techniques, which is an equilibrium finite element method. It conceives an approach for classical design equations, which are originally set for concrete weir design applications, so that they suit the intended safety of structures. After fixing all the dimensions, designing and analysis of each alternative the cost estimation for each alternative solution is done by taking the major quantities of work and the current unit rate cost. The functional plausibility of each alternative is also analyzed by considering the practicality of the proposed alternative on the challenging foundation it rests during construction, after construction and on the operational time.

The final step to solve the problem of this project for this thesis is selection of best alternative from generated alternatives using functional plausibility, economic consideration and practicability of the structure on the site condition.

The scope of the research will be limited to identifying or selection of appropriate headwork typology, design and analysis of a foundation for the headwork (weir or dam of different types) on the existing complex geology. Based on the above methods and procedures used the research will emphasize to demonstrate theories, procedures and techniques through practical application of approaches in Design and analysis of a diversion headwork and it will technically focus on concerns of providing engineering solution for enabling headwork construction under extreme foundation strength and sealing constrained condition.

4 HYDROLOGY OF KURAZ PROJECT

4.1 The Omo Gibe Basin

The Omo-Gibe basin is one of the major river basins in Ethiopia which is situated in the southern part of the country. It has a total catchment area of about 79,000 km^2 and it has a catchment area of 48,520 km^2 at the diversion site. The Gibe River rises just north of latitude 9°N and longitude 37°E of the Ethiopian Plateau which is geologically an area comprised of Tertiary volcanic. The Gibe headwaters are at an elevation of about 2200m.asl. Although there are some important tributaries from different directions, the general direction of flow of the Gibe River is southwards, towards Lake Turkana trough, a fault feature filled with alluvial and lacustrine sediments of recent origin associated with the Great Rift Valley.

The northern part of the catchment has a number of tributaries from the North East, the largest ones being the Walga and the Wabi River. These drain largely cultivated land, much of it with rather impeded drainage. This is an area where erosion processes are important. The Tunjo and Gilgel Gibe Rivers are important tributaries, also draining mainly cultivated lands from the South West. Many of the rivers rise in plateau areas at an elevation above 2000m.asl and parts of the watershed are higher than 3000m.asl. The Gibe itself rises at an elevation of 2000m.asl and crosses the 1000m.asl contour between the Megecha and the Gojeb tributaries. To the west of the river basin the watershed reaches an elevation of 3000m.asl between the Gojeb and the Gilgel Gibe River.

The lower Omo River has experienced a lot of flood problems in recent years, but this will be regulated from now on ward due to the large hydropower dam of Gibe–III hydropower. The width of the river in the lower reach (below Weir station) during flood time may vary from 800m to 3km. The river depth is around 4-30m. Thick densely grown brushwood and bushes dominate the over flooded area.

4.2 Description of Kuraz Project

The project area is located in South Omo Zone, Selamago & Gnang atom woredas and Bench Maji zone, Maji & Menit Shasha woredas, of SNNPR. The large part of the area is found in south omo zone. Hydrologically the area is part of the large Omo-Ghibe basin, at the lower part of Omo river.Kuraz headwork site is situated on the Omo river at a location of 6^0 17" 03"N and 36^0 2" 35"E with an elevation of 475 m.a.s.l.

The site is located about 240 km far from Gibe Ⅲ dam. This diversion site was selected at the outlet of Omo River to flood plain and it is intended to divert irrigation water to150, 000 ha of land on both sides of the river for Sugar cane development. The following section discuss about catchment flood, climate, low flow, sediment yield analysis of the project and all the hydrologic data used for this research is based on [5]. The following figure shows the weir site location.

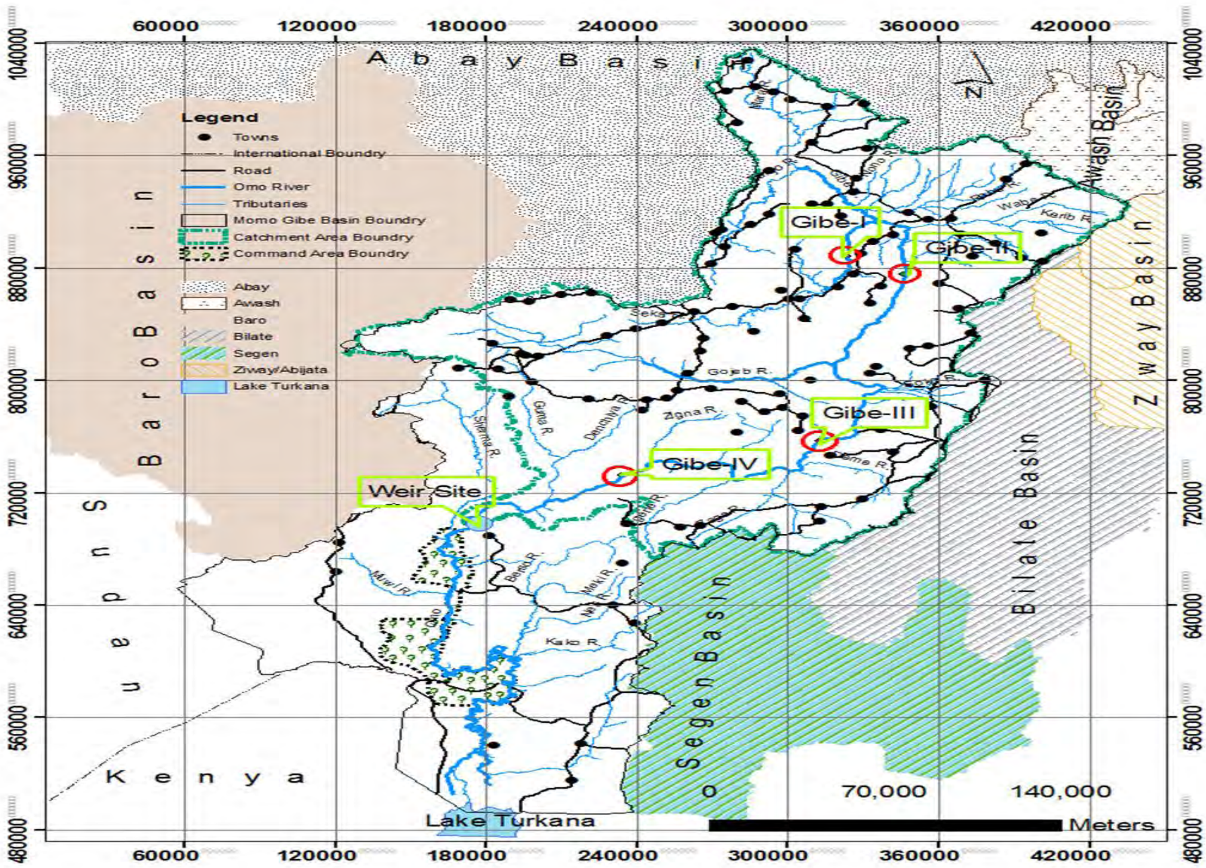

Figure 4-1. Location map and drainage network of the study area

4.3 Climate

The climate of Omo river valley varies from tropical humid in the highlands that include the extreme north near Bako to the hot arid climate in the southern parts of the flood plain.

The climate is classifiable as tropical humid in the highlands that include the areas surrounding Jima and around the headwaters of the Gojeb River. For the rest, and greatest part, of the watershed the climate is classifiable as a tropical sub-humid, intermediate between the tropical humid and the hot arid climate characteristic of the southernmost part of the floodplain toward Lake Turkana. The rainfall ranges from over 1900 mm/year in the northern and western part of the basin, and rainfall rates decrease strongly southwards, to less than 300 mm/year near Lake Turkana.

4.4 Design Flood for the Diversion Weir and embankment option

Gibe River is not gauged for most of its lower portion which is inconvenient for proper quantitative estimation of flow. The river is originating from the upper escarpments located in the north-west bordering with Abbay Basin. There are 7 or more major gauging stations in the upper area that are useful in directly assessing the yield of lower catchment at the selected site. For the incoming flow to Kuraz diversion site two scenarios have been seen:

- Flow estimation not taking into consideration the Gibe Ⅲ dam and
- Flow estimation by taking into consideration the effect of Gibe Ⅲ dam.

The HEC HMS hydrological model result shows the hydrographs for the range of return periods i.e. 5, 10, 25, 50, 100 and 500 years return periods. The design flood peaks for the above return periods at the diversion site are tabulated below in table 4.1 and figure 4.2 shows the design hydrographs at the weir site for 100 and 500 years return period [5].

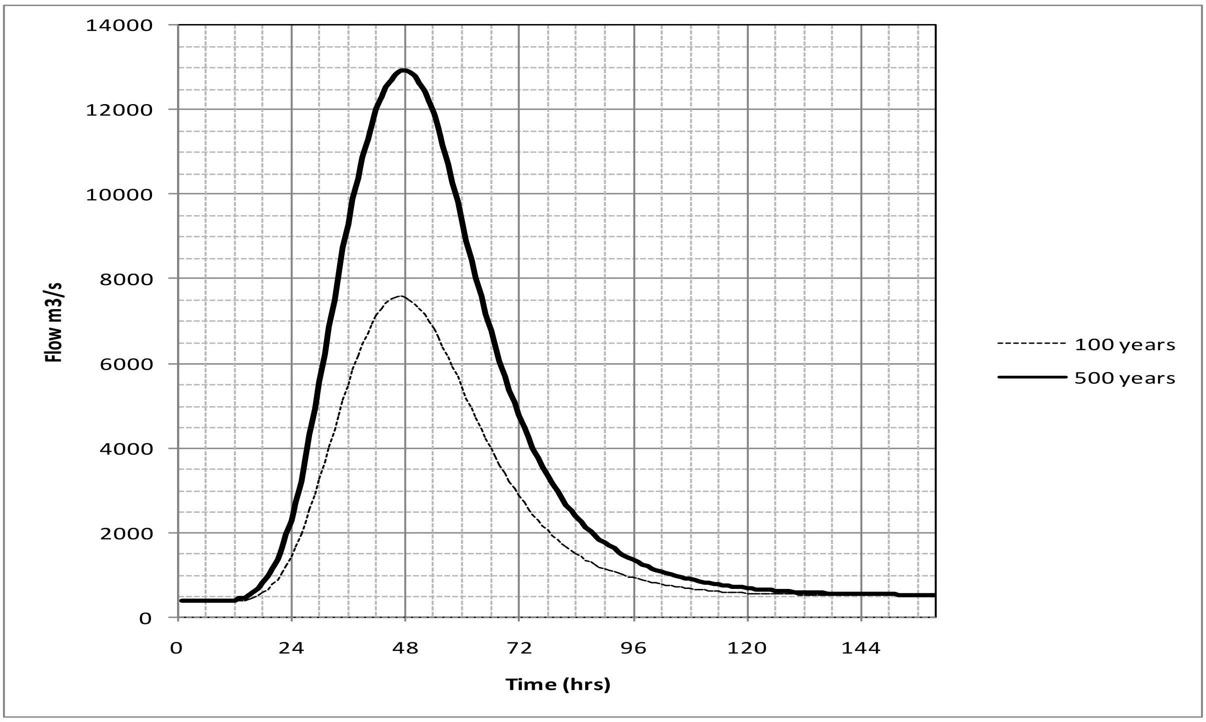

Figure 4-2. Flood hydrograph HEC-HMS output

Table 4.1: Design Peak Flow at Diversion Weir.

Return Period, yrs	Design Flood, m^3/s
5	3591
10	4245
25	4858
50	5048
100	7585
500	9500
1000	10300
10000	12900

In addition to the flood from Gibe III dam the diversion weir will face up the discharge from the residual area. The peak discharge from residual area have been estimated and augmented to the flood incoming from Gibe III dam to find the peak discharge at the proposed weir site. The flood magnitude at the Kuraz head work will be reduced after Gibe III HP is operational (Table 4.2).

Table 4.2: Peak Discharge With and / or Without Gibe III Dam Site.

T-yr	5	10	25	50	100	500
Peak Q (m3/s) at Weir site if Gibe III is not Operational	4,083	4,866	5,607	5,847	8,812	15,134
Peak Q (m3/s) at Weir site if Gibe III is Operational	3,678	4,376	5,042	5,261	7,931	10,286

For instance the 100 year peak design flood of Omo River at the proposed weir site is 8,812 m^3/s when Gibe-III becomes operational the 100 year design flood will reduce to 7,931m^3/s.

Dam overtopping failure is defined as the case where the water level or individual waves exceed the highest water tight level of a dam. There are several standards that are in use for design of dams against flood and wave overtopping. Much of these standards are mostly the selection of „safe" design flood is based on estimation of annual exceedence probability that corresponds to an acceptable level of flood risk with respect to human and economic consequences of failure. For example, where an area is heavily populated and/or developed industrially and the failure of the dam will result in loss of life and great property damage, a design flood of very low probability calculated by different methods (such as flood frequency analysis on historical records of maximum observed floods, flood envelop curves, empirical and regional formulae, modern methods of rainfall-runoff analysis) will usually be justified. The highest standard uses the probable maximum flood (PMF), i.e. the extreme flood that is physically possible, which result from severe most combinations of meteorological, like the probable maximum precipitation (PMP) and hydrological factors.

The estimation of very rare floods, or of the Probable Maximum Flood (PMF), can raise questions that encompass the practical, the scientific and the philosophical. In a study of modest proportions (3.5 man-years in the context of more than 2000 major UK dams) it is inevitable that coverage has been uneven and adoption of PMF for the design of a new dam [26].

In order to apply consistent, safe hydrologic design standards across the nation, it is recommended that the practice of prescribing an IDF using arbitrarily selected composite criteria or percentages of hydrologic events be discontinued.

Additional variation is present in methodologies related to the transformation of a PMP to a PMF as well as in required freeboard and other IDF accommodation criteria. Of those agencies that have incorporated some form of risk-informed analysis in the selection of the IDF, the accepted risk tolerances and risk analysis methodologies also differ.

The prescriptive approach relies upon determination of a PMF for high hazard dams which requires assessment of the PMP. The most common sources of the PMP information are the regional HMRs published by the NWS. These reports provide generalized rainfall values that are not basin-specific and tend to represent the largest PMP values across broad regions. Most of these reports have not been updated to reflect current state-of-the-art knowledge and technology. A site specific study of the PMP/PMF using current techniques can result in a more appropriate estimate of the PMF for consideration as the IDF.

However, in agricultural areas where failure would result only in flooding of crops, a design for a much smaller degree of protection could be reasonable. When conditions lie between these two extremes, varying design flood of certain probability (return period) (e.g. *Q150*, *Q1000*, *Q10,000*) will apply for the relevant level of dam safety.

In practice, the selection of the presumably „safe" design flood is done following recommendations of exceedence probability for design floods given in standards. Most of the standards base for their recommendations on subjective judgment of potential effects of dam breach (no ridged monetary or quantified otherwise limit is given).ICOLD (1992) gives summary on this standards and approaches. ICOLD (1992), whilst not providing fixed rules for selection of „safe" design floods, it reports criteria, which can be used to guide selection of suitable design floods that are developed for selecting a spillway design flood. This guideline is reproduced in Table 4.3 below.

Table 4.3: US Army Corps of Engineers recommended spillway design flood (ICOLD 1992).

S.No	Hazard Category	Hazard Definition	Reservoir design flood
1	Low	No loss life Minimal economic loss	Small Dam1: 50-100 year flood Medium Dam2: 100 year flood - 0.5 PMF Large Dam3: 0.5-1.0 PMF
2	Significant	Few lives lost small number of habitable structure appreciable economic loss	Small Dam: 100 year flood - 0.5 PMF Medium Dam: 0.5-1.0 PMF Large Dam: PMF
3	High	More than a few lives lost Extensive economic loss (community,industry, agriculture)	Small Dam: 0.5-1.0 PMF Medium Dam: PMF Large Dam: PMF

For their recommendation these standards generally capitalize on dam size, risk involved to life and property in case of failure and/or type of dam. Based on subjective consideration of these factors, a „safe" design flood is recommended, As a normal practice, a weir is generally designed for 50 years flood (Q_{50}) and its free board is checked for next higher flood (Q_{100}).But, realizing the importance of this weir structure as it is planned to serve more than 150,000 ha net command area with the establishment of 6 sugar crushing factories, the designed flood has been considered as 100 years flood (Q_{100}) and for the embankment dam section $Q_{10,000}$ adopted reasons specified above.

4.4.1 Rating Curves

The stage discharge graph also called rating curves have been developed for the river cross section at the weir axis and at 100m downstream of the weir axis for the hydraulic design of the weir structure by using FLOW MASTER software [5].

The rating curves have been developed by using the software with the assumption of average river bed slope near the weir site (0.0025) and the value of Manning‟s roughness coefficient, n = 0.035 in the well-known Manning‟s formula to calculate the average velocity.

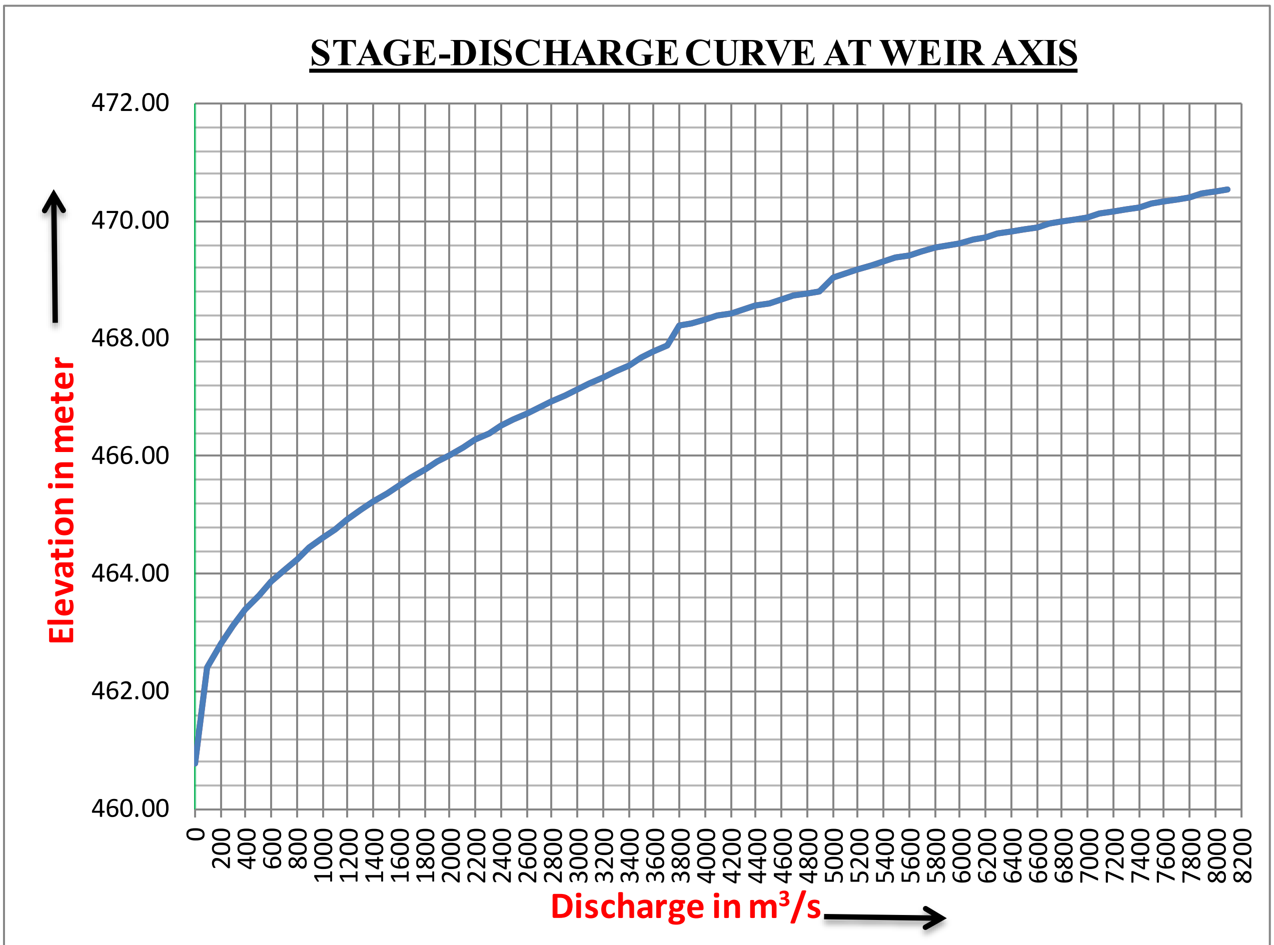

Figure 4-3. Rating curve

The Rating curve developed at weir axis has been used to determine the corresponding water levels for the design flood discharge (Q_{100}). Whereas, the rating curve developed at 100m downstream of the weir axis has been used to determine the tail water depths and levels corresponding to the design flood discharge (Q_{100}) and minimum environmental release discharge. The stage discharge curves/ rating curves plotted by using the software "FLOW MASTER" at weir axis and at 100m downstream of weir axis are shown in Figure 4-3 and Figure 4-4.

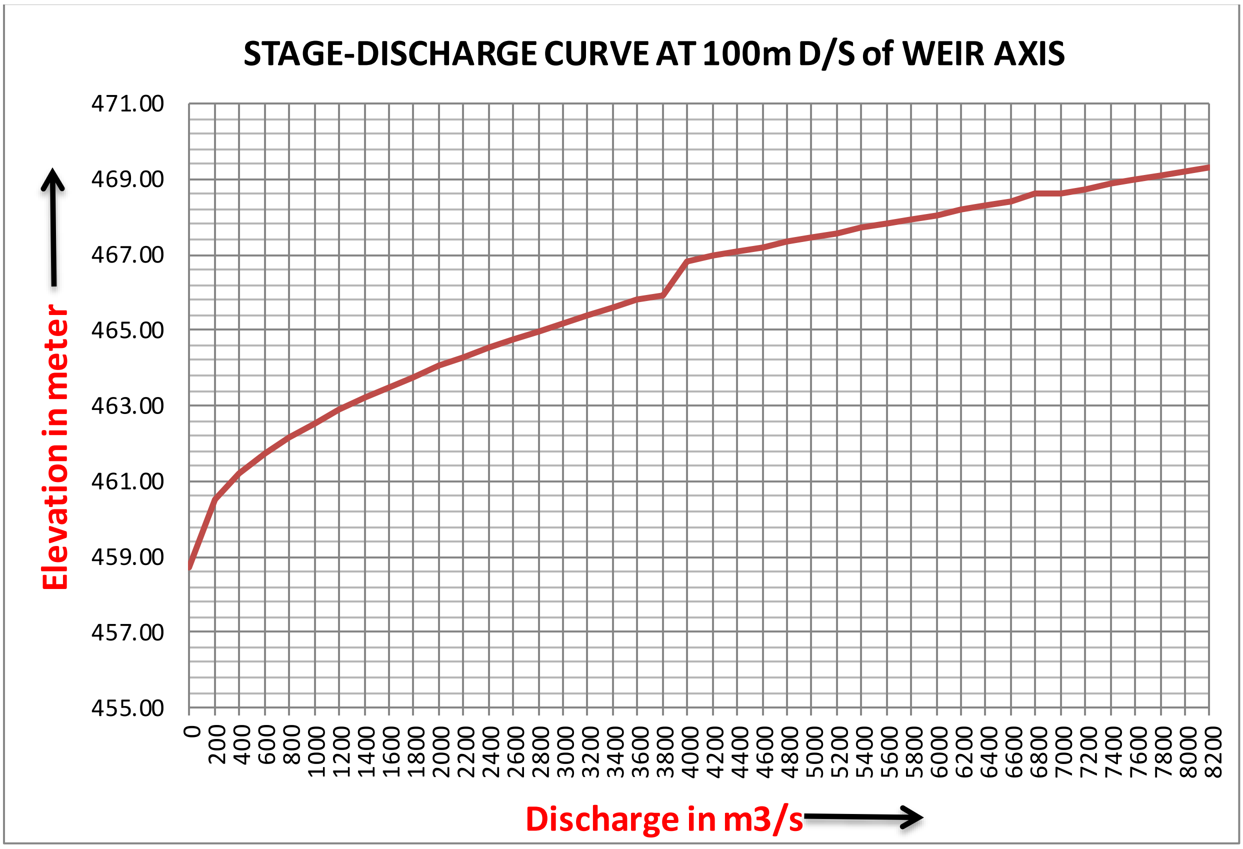

Figure 4-4. Rating curve at 100m downstream of weir axis

4.5 Irrigation Water Requirements

The irrigation water requirements have been planned for an irrigated area of 150,000 hectares. These requirements to be diverted from the head works. These are the gross requirements after accounting for all losses and on the basis of 24 hours surface irrigation.

4.6 Ecological Release from Headwork

For ecological reasons it is mandatory to release a minimum flow downstream round the year. This has been estimated to be about 25 m^3/sec (i.e. minimum flows at Omo River). The above rate basically corresponds to the lowest monthly average dry season flows encountered during the period 1964-2001.This rate of 25 m^3/sec gives a mean monthly flow of 65.7million cubic meters, which is to be released downstream of the head Works after meeting irrigation demands.

5 CONSTRUCTION MATERIAL INVESTIGATION

Construction material investigation is vital for the most economical type of diversion structures design. This will often be the one for which materials can be found within a reasonable haul distance from the site, including material which must be excavated for the dam foundation, main structure, and other appurtenant structures. The construction materials assessed and investigated on the site include:

- Rock quarry for masonry, Riprap and Coarse Aggregate;
- Embankment fills materials for coffer dam and Sand for fine aggregate.

5.1 Rock Quarry

Sound rock appropriate for masonry, riprap and for coarse aggregate (after crushing) was assessed and located at closer proximity from the weir site shown on Figure below. Rock source for different purposes were assessed in the site with the following three main criteria.

- A source which can produce rock fragments in suitable size (fresh to slightly weathered),
- A source which can produce hard, dense and durable to withstand destructive forces during placing, wave action, weathering, servicing, etc.
- Nearby site to reduce haulage distance.

The criteria for rock material generally require the rock to be sound, well graded, and free draining without specifying gradation. The soundness of the rock quarry depends on the resistance to water absorption and abrasion.

Kuraz Diversion Head Works Geotechnical, Seismic Hazard and Construction Material Investigation Report(Draft) accordingly for the Kuraz weir site; two rock quarry sites 1 km on the left side of Omo River was assessed and investigated. The rock is fresh and strong Aphanitic Basalt. The following figure shows a rock quarry site.

6 ALTERNATIVE DIVERSION STRUCTURES

6.1 Selection of Diversion Structure Alternatives

The basic consideration in the design of this diversion structure is to achieve the required functionality and safety consistent with economy. Based on the above criteria and practical points, material we have and assessing different design standards basically Design Standards Embankment Dams No 13.

Selections of different options are mainly based on;

- Purpose (diversion or storage) in relation to demands and hydrology;
- over flow structure or dam with separate spillway
- Foundation competence, Morphology and topography
- Material availability
- In this particular case the existing situation is also very critical (how to decommission the cofferdam? Is there a way for using it permanently?)

Based on the above criteria the possible technical headwork options are:-

- On the rock foundation (right side) concrete weir and on the island and left side of the structure earthfill Dam.
- On the rock foundation (right side) concrete structure and on the island and left side of the structure earth fill-rock fill structure.
- Entire weir structure which is proposed by WWDSE, for this research all the dimensions are taken from the design document by WWDSE.
- For the current progress at the site can include the possibility of using the cofferdam as a permanent structure
- Avoiding head work and replacing it with an idle canal all the way from Gibe IV.

For the fourth option above the existing temporary cofferdam dimensions are 800m length, 28m height and width 23 m. Then for the current progress of the project the existing cofferdam will be used as part of the main embankment dam after rehabilitated with a number of provisions: an impervious plastic concrete cut off along the axis, intercepting the river alluvium and partly extending into the abutment, a draining layer and additional rock fill buttress on the downstream slope, a line of deep relief wells at the embankment toe reshaping and armouring of the upstream. The coffer dam will therefore be upgraded to become an earth-and-rock dam across the Omo River. But this long length and high height of the embankment may not be feasible for the selected site and even the existing foundation of the temporary cofferdam may not be well treated as it will done in the main headwork .

Finally I proposed three alternatives.

- On the rock foundation (right side) concrete weir and on the island and left side of the structure earth fills Dam.
- On the rock foundation (right side) concrete structure and on the island and left side of the structure earth-rock fills structure.
- Entire weir structure which is proposed by WWDSE, for this research all the dimensions are taken from the design document by WWDSE.

From these alternatives the one that gives the lesser cost, ease of construction, lesser time of implementation, as well as the functionally possible is selected as best.

6.2 Design Criteria for Diversion Weir

A weir can be defined as a barrier with crest provision to raise the water level in order to take it by gravity to an area for irrigation development. A weir structure is generally constructed across the river for the following purposes:

- Diversion of water into the canals;
- Raising of water level to feed canals;
- Storing the excess flow received from upstream storage sites and
- Regulation of desired supplies.

In general the hydraulic design and planning of the various components of the weir structure have been carried out following the guidelines given in the Indian Standard with particular reference to [4],[6] and [16].

6.2.1 Topography at the Weir Axis

The alignment of the weir is selected in such a way as to ensure normal and uniform flow through all the weir bays as far as possible.

A weir is required to be aligned at right angles to the river course having the minimum length and normal flow thereby minimizing the chances of shoal formation and shrouding of a portion of the weir specially the under sluice pocket. This aspect was given full attention by studying the topography of the area and the nature of the meanders in the various diversion sites that were candidate for the Ratte Irrigation project. The information obtained is of general validity for all diversion structures to be studied in the Lower Omo Valley.

At the diversion site the following features of the river section is considered for the general alignment of the diversion weir.

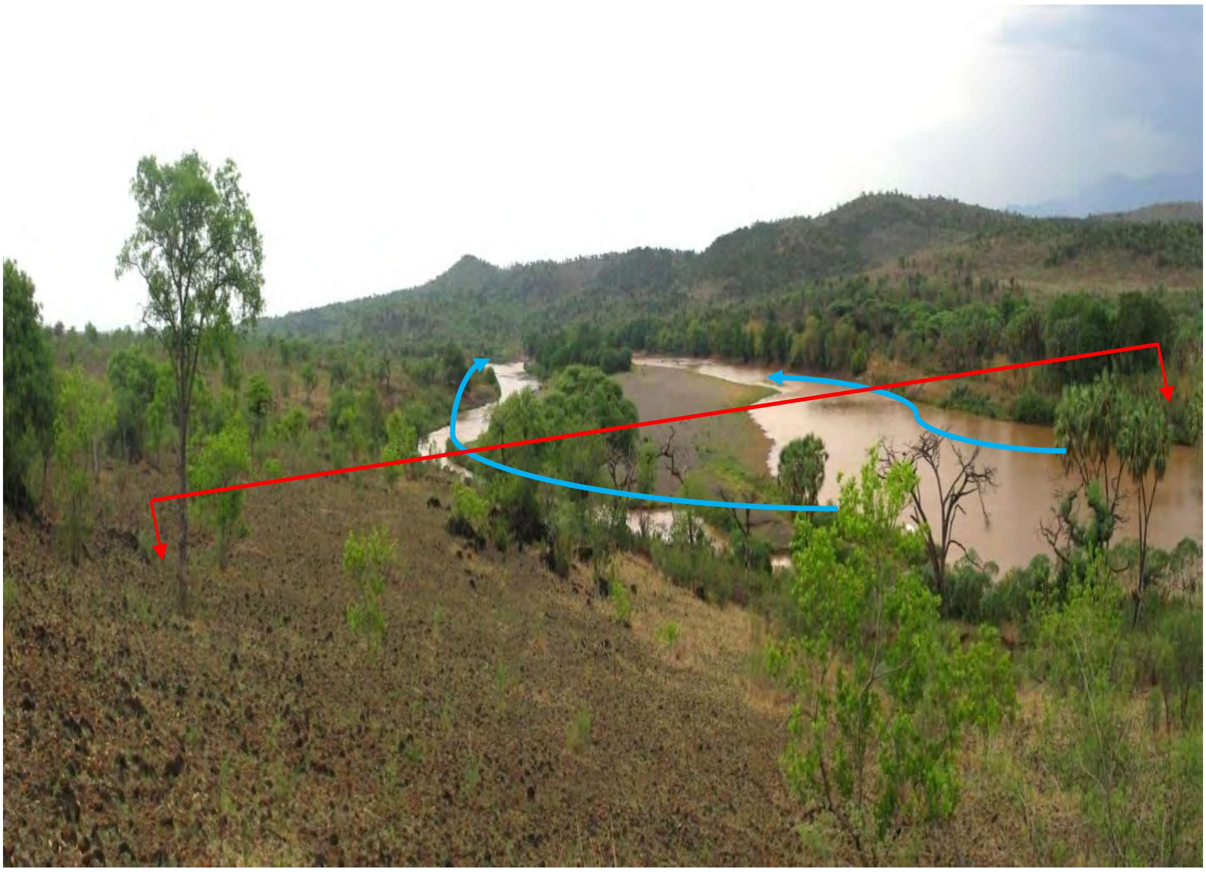

Figure 6-1. Final weir site Location on Omo River showing left and right channels and island

The figure shown below gives information about the river cross section at selected weir site.

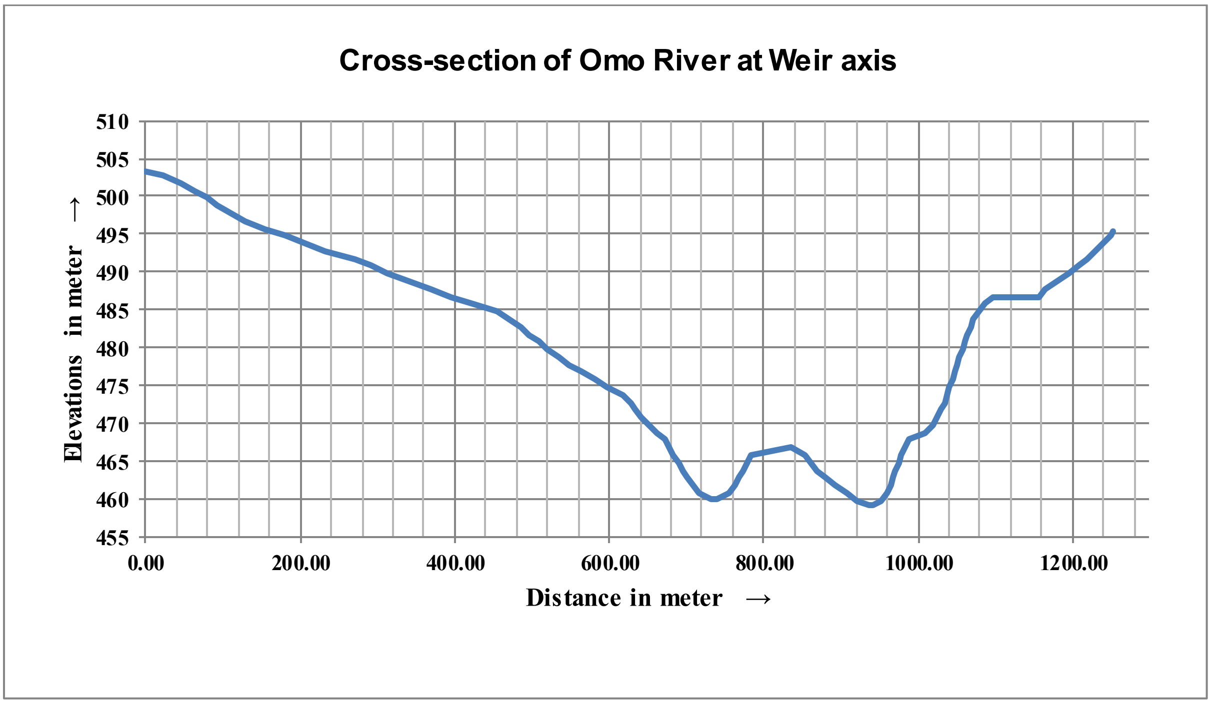

Figure 6-2. Omo river cross section at the diversion weir site

6.2.2 <u>Water Ways</u>

In deep and confined rivers with stable banks, the overall waterway (between abutments including thickness of piers) should be approximately equal to the actual width of the river at the design flood.

The design flood (Q_{100} = 7,931 cumecs) of 1 in 100 years return period flood routed through reservoir of Gibe III hydro power dam is taken for designing the weir and for obtaining the required water way for the proposed weir. The Lacey‟s waterway is given by the formula (in meter) $W = 4.83 * Q^{0.5}$.

For plains and meandering alluvial rivers where silt factor is less than 1.0 the waterway is generally provided 1.0 to 1.2 times of Lacey‟s waterway and where, the silt factor is between 1.0 to1.5 the waterway is generally provided 0.6 to 1.0 times of Lacey‟s waterway for shoal formations.

As per selected site condition and the requirement for keeping the stage of the river at a comparatively low level the water way has been recommended to be kept 397 m width with a looseness factor of approximately 1.0.

6.2.3 Bifurcation

The bifurcation of the river at this location with left and right channel and island at center was acceptable and considered as an advantage for diversion during construction. The left branch of the river was planned to be closed by an upstream and downstream cofferdam to enable dewatering for foundation investigation in the first phase and for construction undertaking in the later stage.

6.2.4 Alignment

As the flow direction of the river (morphology) at the island site is not straight an alignment adjustment of the weir axis was required. The weir axis is rotated to make it perpendicular to the flow direction.

6.3 Methodology for Hydraulic Design

The design of the weir was carried out in two parts i.e., hydraulic design and structural design. In this research only the hydraulic design overall dimensions and profiles of the main structures are worked out so that satisfactory hydraulic performance of the structure will be ensured. In this case the diversion structure has to be properly designed for both the surface and subsurface flow conditions.

The surface design includes the fixing up of waterway, top profile of various structures, energy dissipation arrangements, scour depths, length of divide walls, and afflux etc. Whereas, the sub-surface design includes the fixing up of the depth and section of cut-offs, uplift pressure calculations, exit gradient etc. The following basic parameters which were considered and selected before taking up detailed hydraulic designs.

6.3.1 Design Flood Discharge

The occurrence of a certain peak flood of given return period during the life time of a project is a probability of a given flood that is required to be safely discharged from the project structure.

For the purpose of design of diversion weir other than its free board, a design flood of 100 years frequency (Q_{100}) is considered. The 100 years return period flood have been worked out by various hydrological methods and by using updated nearest meteorological data and found 7,931cumecs at the proposed diversion weir site on Omo River after routing through the Gilgel Gibe-III hydropower dam site. This is called design flood discharge for the structure [5].

6.3.2 Environmental Release

For the purpose of survival of wild life animals, aquatic life or human beings on the downstream of the diversion weir structure, a minimum environmental release of about 35 m^3/sec has been proposed in the design [5]. This minimum release shall take place through the scouring sluices gates throughout a year.

6.3.3 Full Supply Level in Main Canals

The Full Supply Level in the left and right main canals (476.723 m) have been decided on the basis of elevations on irrigable surveyed topography on the both sides command area and having intension to feed these irrigable command area by gravity flow water diverted from the upstream of diversion weir structure.

6.3.4 Pond Level

The pond level is the level of water which is maintained, immediately upstream of the weir to facilitate withdrawal into the main canals for irrigation water withdrawal or for any other purpose. The pond level, in the under-sluice pocket, upstream of the canal head regulators is generally obtained by adding the working head (in this study, it is considered 1.0m) to the designed full supply level in the canal. The working head includes the head required to pass the design discharge into the canal and the head losses in the regulator structure too.

The maximum required water level including the losses in canal system at the upstream top periphery of the right bank and left bank proposed development command area is 476.723 m and Assuming 1.0 m maximum head loss in main canal head regulator structure and driving head required for the design discharge to pass in the proposed main canal. Adding the two the pond level becomes 477.723.

6.3.5 Afflux

Afflux is an important factor for the design of downstream cistern, flood protection and river training works, upstream and downstream loose protections and upstream and downstream cut-off depths. It predicts the extent of back-water effect and the area being submerged on the upstream of the weir structure.

The difference in water level at any point upstream of diversion weir before and after the construction of a diversion weir, maximum afflux occurs just upstream of the diversion weir.

6.3.6 Crest Levels and Profiles

The crest levels and profiles of the diversion weir structure are as detailed below.

- The crest level/ floor level of the scouring sluice has been fixed at the lowest average river bed level with the elevation of 461.423 m so that any deposited silt in front of head regulator structure shall be disposed off easily;
- The downstream profile of the scouring sluice has been kept at a slope of 3H to 1V;
- The crest level of the weir (477.723 m) in the weir proper bays has been fixed at 16.30 m above the sill level of the scouring sluice;
- The profile of the overflow type weir has been fixed as ogee shape profile which represents the under nappe of a jet flowing over a sharp crested weir for the rocky foundation.

6.3.7 Concentration Factor

The Concentration factor is factor by which the discharge per unit length of a weir and under sluice structure assuming uniform distribution is created for designing its various elements. In the design of weir and under sluice structures of the concentration factor has been considered 20 %.

6.3.8 Downstream Retrogression

Due to construction of a weir or under sluice structure across a river, progressive retrogression or degradation of the downstream river bed levels causes lowering of the downstream river stages, which has to be suitably provided for in the design of downstream cisterns. The lowering of river water level due to retrogression on the downstream causes increased exit gradients. Normally, the reduction of gauges due to retrogression of alluvial river bed may be considered varying from 0.3 to 0.5 m depending upon whether the river is shallow or confined during floods.

The Omo River bed is found stiff Silty clay/ jointed rocks/weathered rocks on the central and left part of the weir structure which has very less permeability and less erosion tendency. A 0.50 m downstream retrogression in the river bed has been considered in the design. As the right side river bed at this location is having a good quality rocky profile no river bed retrogression is expected and therefore no downstream retrogression has been considered in the design.

6.3.9 Hydraulic Design

As about 114 m length out of total 397 m length of the proposed diversion weir (height 16.30m above river bed) is planned to be founded on Rocky foundation and 238m embankment dam on stiff silty clay and alluvial deposits foundation, its design needs more concern. The USBR does not prohibit the construction of a concrete dam on pervious foundations but puts up a caution-sign considering the complexity of the foundation materials depending on the type, stratification, permeability, homogeneity and other properties of the foundation materials, as well as the size and physical requirements of the structure itself. USBR also advised that the control of erosion, seepage and uplift forces under dams constructed on pervious foundation requires some additional devices such as:

- Upstream apron with cutoffs at the upstream end;
- Downstream apron with scour cutoffs at the downstream end and
- Cutoffs at the upstream or downstream end or at both ends of the overflow section.

Therefore provisions of upstream and downstream protection works and aprons with cutoff walls at both ends and cutoffs at upstream and downstream end of overflow weir section has been made in the design.

In general, the hydraulic design of any structure requires the fixation and checking of the followings:

- Fixation of sill level, width of sill and shape of sill;
- Fixation of waterway, number and width of spans and height of gate openings, requirements of breast wall, etc;
- Safety of structure -from surface flow consideration;
- Safety of structure from sub-surface flow consideration and
- Energy dissipation arrangements.

6.3.10 Safety against Sub-surface Flow and Uplift

In our case some part of the foundation is resting on silty or alluvium bed the safety against piping shall be attained by providing sufficient flow length in order to provide safe hydraulic gradient. The following measures shall be taken and the values given below for safe hydraulic gradient were used as a guide line.

- Control of sub-surface flow shall be established by provision for uplift pressures and adhering to acceptable limits of exit gradient;
- Provision of upstream and downstream cutoff walls/ diaphragm walls in addition to provision of intermediate cutoff walls/ diaphragm walls, if required, shall be made for limiting the exit gradient;
- Appropriate foundation grouting measures to control the passage of water below the weir structure.

The exit gradient at the end of the impervious floor is determined using accepted procedures which make use of equations and curves plotted for the purpose. The factors of safety for exit gradient for Shingle 4 to 5, Coarse Sand: 5 to 6 and fine Sand: 6 to 7.

For the design of diversion weir structure the portion of the structure which is going to be resting on silt / alluvial deposit foundation, the safe exit gradient has been considered 1/6 (0.17) and for the portion of structure which is going be founded on rocky foundation, the value of safe exit gradient has been considered 1/4 (0.25).

The thickness of downstream floor on sloping glacis with reference to hydraulic jump has been determined on the basis of the hydraulic jump profile has been plotted under different conditions of surface flow, the average height of the jump trough has been obtained by deducting the levels of the jump profile from corresponding hydraulic gradient line. This has been taken as the unbalanced head for which safety of glacis floor has been ensured and the safety against uplift requires the balancing of the uplift pressure by the weight of the floor with a margin of safety factor. The following equation has been considered to determine the required floor thickness.

6.3.11 Design Criteria for Floor and Foundation

The following criteria are established by consideration of surface and subsurface flow to design the sub-structure components.

The downstream horizontal floor level has been provided at such a level so that within the range of discharges the jump always occurs on the slopping glacis for the case where a stilling basin is to be provided. The downstream stilling basin of ogee shaped weir structure has been provided with dented sill and end sill as per USBR stilling basin type-II. For the case where a sloping glacis is provided, the main disturbance of the hydraulic jump normally dies out at a distance equal to five times the length of the hydraulic jump. In that case, the length of horizontal floor should be nearly 5 * (D2- $D_{1)}$.

6.3.12 Floor Length

The total length of impervious floor which consists of upstream floor, crest length, downstream glacis and downstream stilling basin and end sill required from consideration of safety against sliding and piping shall be fixed in conjunction with the depth of downstream cutoff to satisfy the requirements of safe exit gradient, scours and economy.For calculating the total floor length of the structure, Khosla‟s theory has been applied.

6.3.13 Thickness of Floors

For determining the thickness, uplift pressures have been worked out by using Khosla‟s theory at all points of the toe of the ogee profile or downstream glacis of scouring sluice and in the cistern are determined based on maximum pond level in upstream side and minimum flow in downstream side (except minimum environmental release). In this situation, the uplift pressures has been worked out at all the key points of downstream slope of the ogee profile or glacis of scouring sluice and cistern and Net uplift pressures at locations of hydraulic jump at variable discharges at different points during the design discharge flow.

For determining the floor thickness the unbalanced head is worked out by considering the pre-jump and post jump profile in three conditions which are:

- High flood flow condition;
- Pond level flow condition and
- Maximum static head condition i.e. no downstream flow condition except minimum environmental release.

A maximum of the 2/3rd of the maximum unbalanced head of the above two jump formation conditions and one third in the static head condition is chosen for determining the thickness of the floor.

After determining the unbalance head at various key locations the floor thickness is worked out by dividing the unbalanced head with the submerged unit weight of floor material (1.4 considered for submerged concrete).

The uplift pressures are assumed 100 % for the structure portion resting on silty foundation and 50% for the structure portion resting on moderately jointed rocky foundation.

6.3.14 Shape of the Weir Overflow Profile

The weir profile shall be of ogee shape which represents the under nappe of a jet flowing over a sharp crested weir. The recommendations of the United States Bureau of Reclamation hydraulic laboratories have been followed to represent its shape.

The crest level is kept at elevation 477.723 m. The upstream profile has vertical face up to the level of 469.573 m from top and after this it has sloped upstream surface up to the river bed level.

Design of downstream ogee profile

The Equation of downstream quadrant of the Crest for all Spillways can be expressed as [7]

$\frac{y}{Ho}$ = -k($\frac{X}{Ho}$)^{n} Where: n=1.865 and P/H_o= 3.212, hence K=0.502

The detail design of downstream profile of the proposed weir on rocky foundation is presented in appendix 11.1.1 separately and figure 6-3 shows clearly the profile at the downstream face.

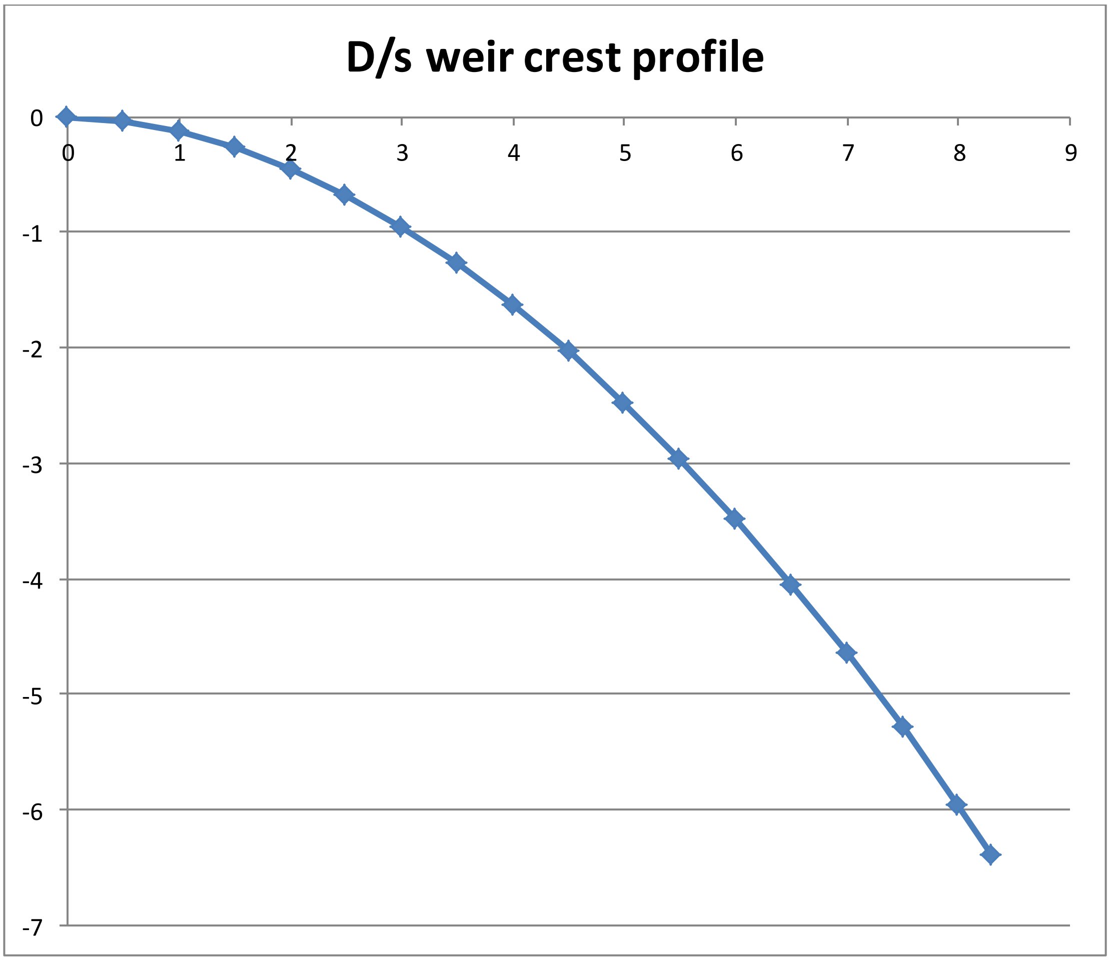

Figure 6-3. Downstream weir crest profile

Design of Upstream crest

The Equation of U/S quadrant of the Crest for all Spillways can be expressed as from design of small dam taking upstream vertical weir section

$X_c/H_d = 0.276$, $X_c = 1.4m$

$Y_c/H_d = 0.122$, $Y_c = 0.62m$

$R_1/H_d = 0.525$, $R_1 = 2.66m$

$R_2/Hd = 0.228$, $R_2 = 1.16m$

6.3.15 Weir floor thickness

Floor thickness calculation Taking floor material density 2.4. Hence the thickness at the end,10, 30 and 45m of downstream end are 2.1,2.5,3 and 3.5m respectively.

Gate of spillway

In the previous design of WWDSE the proposed weir is over flow concrete weir extending in the entire cross section of the river. But for this research for the proposed composite dam a radial controlled or gated spillway is adopted because of the following advantages.

- The operation of radial gate defines the water level behind the dam as the discharge rate can be easily controlled by partially opening the gates, adopting the height of the orifice.
- The sediment transport by means of radial gates and the low ogee crest will be greatly enhanced.
- Controlling downstream flooding or maximization of conservation storage is possible.
- Since the site is restricted it possible to construct easily rather than the required long length of ungated spillway.

6.3.16 Design of stilling basin

Steps in design of spillway stilling basin

I. Use the tail water curve
II. Preparation of the hydraulic jump curve

The discharge versus conjugate depth elevation/ jump elevation curves and discharge versus tail water level curve are important to get the safe level of stilling basin and calculated from detail design of stilling basin and drawn as shown below in figure 6-4 and 6-5.

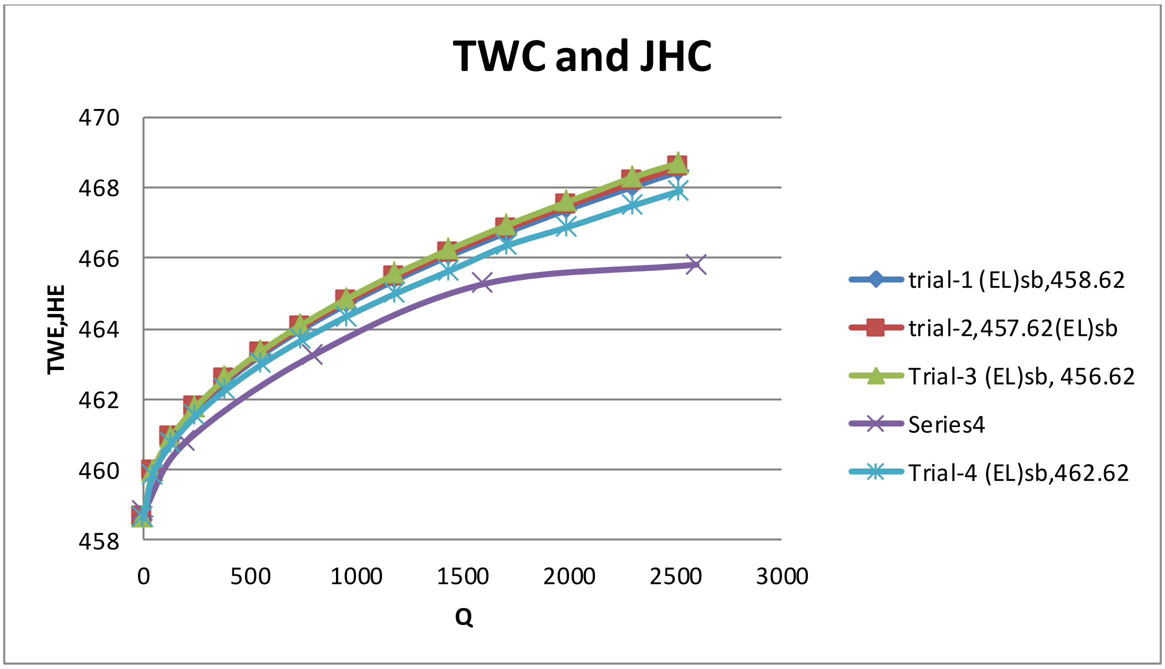

Figure 6-4. Tail water curve and jump height curve

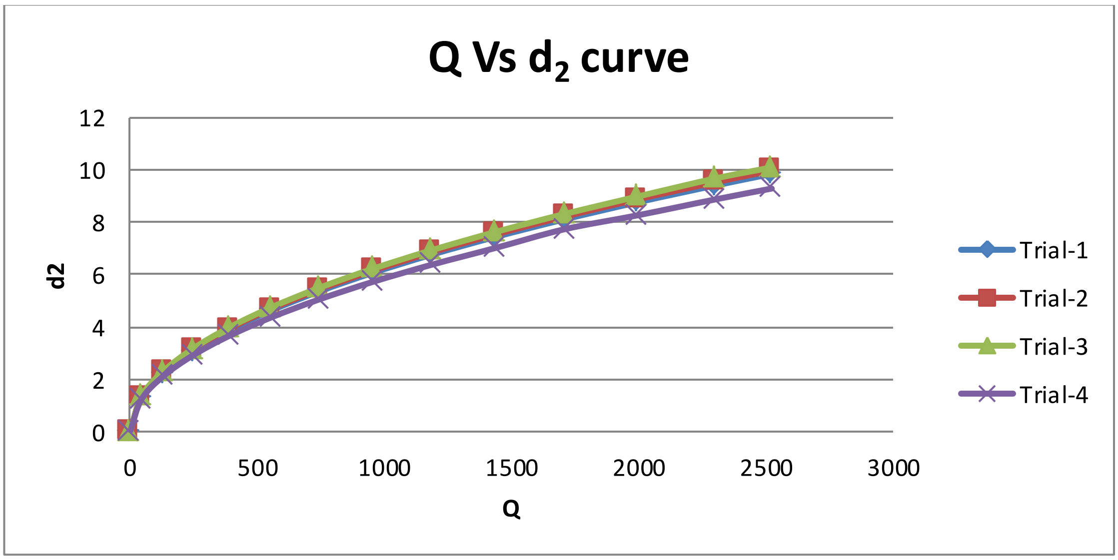

Figure 6-5. The discharge versus depth after jump (d_2) curves

The stilling basin level is 462.62 m and the length of stilling basin by USBR chart type-II is 56.02 m i.e., taken as 57 m. since the F_r>4.5 and velocity>18m/s the adopted stilling basin is Type-II [7].The detail design of the proposed stilling basin is presented in Appendix 11.1.6 to Appendix 11.1.9 separately.

6.3.17 Type II Basin Dimensions

Chute block

Height of chute block = h_1= d_1 = 1.3m

Width of chute blocks =W_1= d_1 = 1.3m

Spacing b/n chute blocks = S_1 = d_1 = 1.3m

Space of end chute from wall = $d_1/2$ = 0.65m

Stilling basin

Length of Stilling basin = 57m

width of stilling basin = 114m

Dentainted sill

Height of Dentainted sill = h_2 = $0.2d_2$ = 1.86m

Front width of Dentainted sill = W_2= $0.15d_2$ = 1.39m

Top side width of Dentainted sill = $0.02d_2$ = 0.19m

Spacing b/n Dentainted sills = S_2 = $0.15d_2$ =1.39m

6.3.18 Diaphragm Cutoff Walls

To reduce the loss of water by seepage through sub-surface flows to an amount compatible with the purpose of the project and To eliminate the possibility of structural failure by piping. The deep cutoff walls can be provided as a diaphragm wall [15]. Because the diaphragm walls have excessive depth of the impervious stratum overlain by pervious or heterogeneous strata; Construction difficulties, like heavy dewatering; Instability of side slopes of excavations; lack of construction materials and Time constraints and etc.

Where the foundation strata consists of boulder deposits, open gravel pockets, zones of talus, slide areas, contacts of formation of different geological age, fractured zones or similar features, it is obvious that voids/ openings may be larger and contact with diaphragm wall face may not be continuous. It was imperative and important carefully consider that bending stresses between contact free lengths are taken care of. Any flexible type of diaphragm wall which undergoes large deformations at low stress levels is not suitable. So we proposed the rigid type of diaphragm wall is preferable under such conditions.

In general the diaphragm walls are of thicknesses 500 mm, 600 mm, 750 mm and 900 mm. These thicknesses have been found to be satisfactory up to a depth of 50 m. In this project a wall thickness of 1000 mm has been used to ensure trench stability and resistance to hydro fracturing.

6.3.19 Vertical Cut offs

Cutoff walls can be constructed under aprons or under the dam itself to prevent or reduce under seepage and to increase the length of percolation to reduce the uplift under the main portion of the structures. A concrete cutoff is probably the best type of cutoff for preventing under seepage. In addition to acting as a cutoff it also adds substantially to the stability (sliding resistance) of the dam when placed under the dam structure.

The new proposed overflow weir is 16.3 m high resting on rocky foundation the provision of cutoff walls has been made at the upstream and downstream end and for the under sluice part on silty foundation intermediate cutoff is added.

The vertical cut offs are required under the structure to reduce the loss of stored water through foundation and abutments; to prevent sub-surface erosion by piping and to reduce the exit gradient to bring it within the safe limit at exit end.

The upstream and downstream cut off walls are generally be provided to cater for scours up to 1.25 * R and 1.50 * R respectively where R is the normal depth of scour below the respective high flood levels. The concentration factor of 20 % has also been taken into account in fixing the depth of cut offs.

River scour is likely to occur in erodible soils such as clay, silt and sand. In non-cohesive soils, the depth of scour may be calculated from the Lacey"s formula which is as follows.

$R = 0.473 * (Q/f)^{(1/3)}$ when looseness factor is more than 1 or;

$R = 1.35 * (q^2/f)^{(1/3)}$ when looseness factor is less than 1;

For the part of structure resting on silty foundation where the soil is mixed with pebbles and cobbles, the silt factor has been assumed to be 2 and for the foundation resting on moderately jointed rock the silt factor has been assumed to be 3. The cutoffs have been provided on the basis of provisions contained in USBR recommendations. These are:

- Cutoff at the upstream end of the apron;
- Scour cutoff at the end of the downstream apron and
- Cutoffs at upstream and downstream, both ends of the overflow section.

6.3.20 Design of Protection Works

Design of protection works for the Weir on Rocky foundation

Protection works are required on the upstream as well as on the downstream in order to obviate the possibility of scour hole travelling close to the pucca floor of the weir and to relieve any uplift pressure through the filter. The arrangement consists of inverted filter and launching apron.

Downstream Loose Protections

Inverted Filter

An inverted filter invariably reduces the possibility of piping, as it allows free flow of seepage water through itself without allowing the foundation soils to be lifted upward. The river bed is scoured during flood flows and large scour holes may develop progressively adjacent to the concrete apron which may cause undermining of the structure. Such flood scour depth below HFL corresponding to a regime width is called regime scour depth.

Estimated by the following (Lacey‟s) formula

R = 0.475 (Q/f) 1/3

R = 0.475*(7931/3)1/3 = 6.57m

The total scour below HFL is taken as xR, where R is Lacey‟s normal scour depth and the values of x for different classes of scour are tabulated below in table 6.1.

Table 6.1 : The value of x for different classes of scour.

Class of scour	Reach	Mean value of x	D = XR-water depth above bed
A	Straight	1.25	1.25R-Y
B	Moderate bend	1.5	1.50R-Y
C	Sever bend	1.75	1.75R-Y
D	Right angled bend	2.00	2R-Y

Value of x is generally taken as 1.5 for design of d/s protection works and 1.25 for design of up steam protection works. Just at the end of the concrete floor, an inverted filter 1.5 to 2D long is generally provided, where D is the depth of scour below the original river bed.

D = xR - Water depth above bed

Downstream HFL = 470.473

Downstream River bed level =461.423

Water depth above bed = 9.05m

D = 1.5*6.57 – 9.05 = 0.81m

Hence length of inverted filter is 2D = 2*0.81 = 1.62m, take 2m

The depth of inverted filter is kept equal to the depth of downstream launching apron. It generally consists of 1.0 to 1.2m deep concrete blocks with open joints laid over 0.6m thick graded filter material.

The filter therefore, consists of layers of materials of increasing permeability from bottom to top. The gradation should be such that while it allows free flow of seepage water, the foundation material does not penetrate to clog the filter. To prevent filter form dislocation under surface flow, concrete or masonry blocks are laid over the filter material.

Launching Apron

After the inverted filter, the loose apron called launching apron is provided for a length, generally equal to 1.5D, where D has the same meaning as given above in inverted filter calculations.

Length of launched apron = $\sqrt{5}$ D = $\sqrt{5}$ *0.81 = 1.22m, Take 2m

The apron generally launches to a slope of 2:1 and if t is the thickness of the apron in the lounched position, length being $\sqrt{5}$ D; the volume of stone per meter width will then be $\sqrt{5}$.D.t

Since the volume of stone should be the same in launched and un launched apron, and if the un launched apron is laid in a length equal to 1.5D, the thickness of the launched apron a thickness t that its quantity is approximately 2.25D cu.m/m, therefore t = $\frac{2.25*0.81}{2}$ = 0.91, take 1.5m

Provide c.c blocks of size 1.5mx1.5mx0.9m over an inverted filter of 0.6m thickness for a length equal to 2m.2 rows of c.c blocks of size 1.5mx1.5mx0.9m having 10cm gaps filled with blinding layer shall be provided in length equal to 3.1m.

Upstream loose protection

Just upstream of the concrete floor of the weir, block protection is provided. It generally consists of concrete blocks laid over package stone, for a length equal to D (D= xR -Y, where x=1 to 1.5, generally taken as 1.25)

D = 1.25R –Y = 1.25*6.57 – 9.05 = -0.84m, hence no need of inverted filter and launching apron at the upstream part of weir.

The figure shown below is the section drawing of proposed weir on the right side of the cross section or rock foundation.

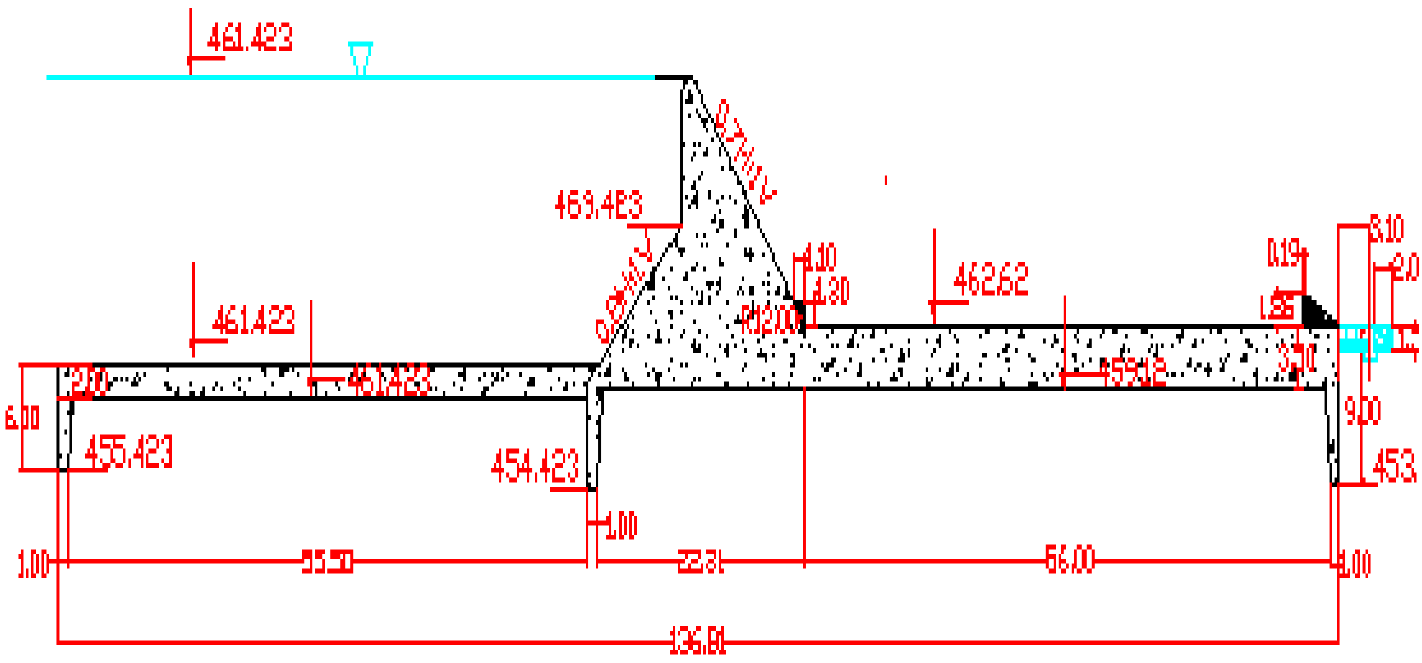

Figure 6-6. Concrete weir section

The detail design of the proposed weir on rocky foundation is presented in Appendix 11.1.2 to Appendix 11.1.5 separately.

6.4 Under Sluices

A weir type diversion structure normally requires deep pockets of under sluice portions in front of the head regulator of off taking canals and long divide walls to separate the remaining river bay portion called weir proper from the under sluices portion. The arrangement is aimed at keeping the approach channel to the canal head regulator comparatively clear of the silt and to minimize the effect of main river current on the flow conditions in the regulator.

The width of the under sluice portion has been determined on the basis of the following considerations.

- It should be capable of passing at least double the canal discharge to ensure good scouring capacity;
- It should be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods and
- It should be wide enough to keep the approach velocities sufficiently lower than critical velocities to ensure maximum settling of suspended silt load in the pocket.

The under sluice section resting on silty foundation has been designed in consideration of maximum design flood (Q_{100}), without concentration and retrogression and with 20% concentration and 0.5 m river bed retrogression and pond level flow condition, without concentration and retrogression and with 20% concentration and 0.5 m river bed retrogression.

6.4.1 Under sluice on silty foundation

The detail design of left side under sluices, the foundation of which are resting on silty strata are presented in Appendix 11.1.10 to Appendix 11.1.22 separately. The lowest points the jump will occur is 455.285m and hence provide the downstream cistern level at 455.1m a.m.s.l and length equal to 61.01m take 65m.

Total floor length and exit gradient

Safe exit Gradient, G (as stiff silty clay foundation) = 0.17

D/S cutoff depth provided below the river bed, d = 12m

Exit gradient, G = H/d1/Π√λ <= 0.17

λ = 1+√1+α2/2

α = b/d = b/12

Substituting the save value of G, gives b = 142.9m, provide total floor length to reduce uplift b = 150m

Providing 3:1 for d/s glacis at d/s level of 455.1m

d/s glacis = 3*(crest of under sluice - head regulator bed) = 19m

d/s apron length = 65m

Floor length required to join the d/s retrograded bed level = 12m

Total d/s floor length required = 96m

Therefore, the balance length will be provided in u/s floor length = 54m

Provide 12m downstream, 6m upstream and 6m intermediate cutoff walls.

Thickness of under sluice on silty foundation

From the calculation the maximum unbalanced head in the jump through is equal to 4.459m.The floor thickness. Therefore, is designed for 2/3rd of this head = 2.973m or for the static condition head, whichever is greater from these three cases, it is clear that the static condition governs the floor thickness. Hence, 100% of the head = 9.316m.Taking unit weight of concrete as 2.40. Hence provides a thickness of 6.7 at the start of glacis and 5m at the end.

Design of protection works for under sluice on Silty foundation

Protection works are required on the upstream as well as on the downstream in order to obviate the possibility of scour hole travelling close to the pucca floor of under sluice and to relieve any uplift pressure through the filter.

Downstream Loose Protections

Inverted Filter

Flood scour depth below HFL corresponding to a regime width is called regime scour depth. Estimated by the following (Lacey"s) formula

$$R = 0.475\ (Q/f)^{1/3} = 4m$$

Downstream HFL = 470.473

Downstream River bed level = 461.423

Water depth above bed = 9.05m

D = xR - Water depth above bed = -3.06m, D is the depth of scour below the original river bed. Hence provide an inverted filter and launching apron only for safety purpose. The depth of inverted filter is kept equal to the depth of downstream launching apron. It generally consists of 1.0 to 1.2m deep concrete blocks with open joints laid over 0.6m thick graded filter material.

Provide c.c blocks of size 1.5mx1.5mx0.9m over an inverted filter of 0.6m thickness for a length equal to 6m.4 rows of c.c blocks of size 1.5mx1.5mx0.9m having 10cm gaps filled with blinding layer shall be provided in length equal to 6.3m and 6m launching apron.

Upstream loose protection

Just upstream of the concrete floor of under sluice, block protection is provided. It generally consists of concrete blocks laid over package stone, for a length equal to D (D= xR -Y, where x=1 to 1.5, generally taken as 1.25)

D = 1.25R –Y = 1.25*4 – 9.05 = -4.05m , No need of protection work but for safety Upstream of the blocks, a thickness of 1.5m launching apron is provided in the same way as described for the downstream portion

Provide c.c blocks of size 1.5mx1.5mx0.9m over an inverted filter of 0.6m thickness for a length equal to 4m.3 rows of c.c blocks of size 1.5mx1.5mx0.9m having 10cm gaps filled with blinding layer shall be provided in length equal to 4.8m and 4m launching apron The walls are always generally constructed in between the filter and the apron. The figure shown below is the section drawing of proposed under sluice on the left side of the cross section or which is constructed on silly foundation.

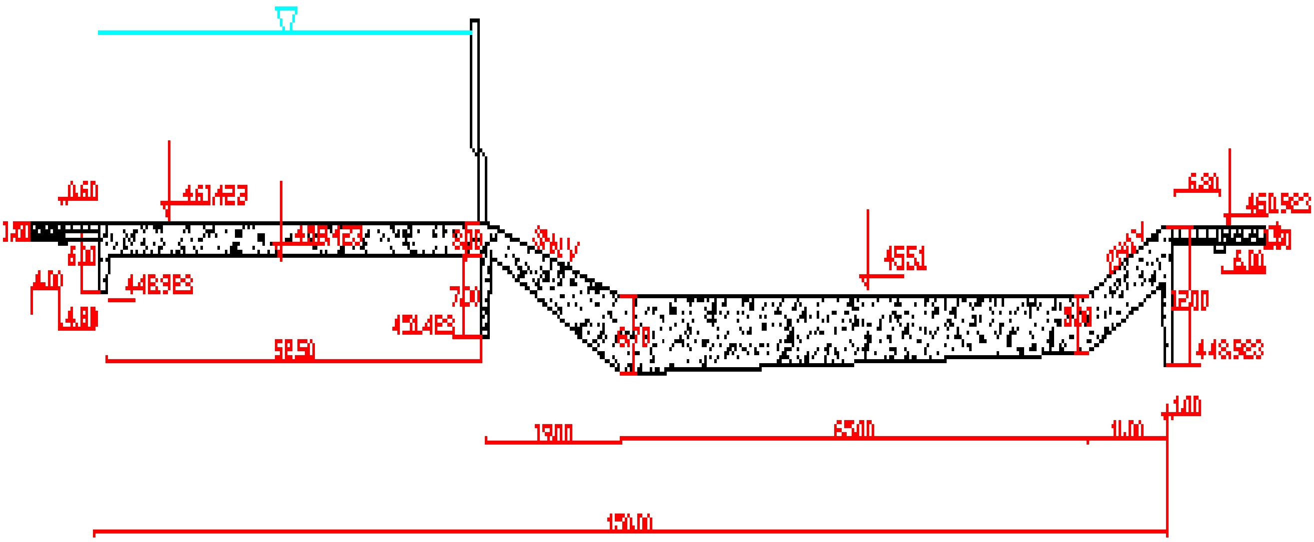

Figure 6-7. Under sluice on silty foundation

6.4.2 Under sluice on Rocky foundation

The detail design of right side under sluices, the foundation of which are resting on rocky strata are presented in Appendix 11.1.23 to Appendix 11.1.35 separately. The lowest points the jump will occur is 455.989m and hence provide the downstream cistern level at 455.9m a.m.s.l and length equal to 61.385m take 62m.

Total floor length and exit gradient

The worst condition is when the flow is at maximum and empty d/s, H = 16.3m

Safe exit Gradient, G (as stiff silty clay foundation) = 0.25

D/S cutoff depth provided below the river bed, d = 9m

Exit gradient, G = H/d1/Π√λ <= 0.25

λ = 1+√1+α2/2, α = b/d = b/9

Substituting the save value of G, gives b = 86.25m, provide total floor length to reduce uplift b = 150m

Providing 3:1 for d/s glacis at d/s cistern level of 455.9m

d/s glacis = 3*(crest of under sluice - head regulator bed) = 17m

d/s apron length = 62m

Floor length required to join the d/s retrograded bed level = 10m

Total d/s floor length required = 89m

Therefore, the balance length will be provided in u/s floor length = 61m

Provide 9m downstream and 6m upstream cutoff walls.

Thickness of under sluice on Rocky foundation

From the calculation the maximum unbalanced head in the jump through is equal to 3.387m.The floor thickness therefore is designed for 2/3rd of this head 2.258m or for the static condition head, whichever is greater. From these three cases, it is clear that the static condition governs the floor thickness. Hence, 100% of the head =7.67m

Taking unit weight of concrete as 2.40.Hence provides a thickness of 4.2 at the start of glacis and 2m at the end.

Design of protection works for Under sluice on Rocky foundation

Downstream loose protection:- Provide c.c blocks of size 1.5mx1.5mx0.9m over an inverted filter of 0.6m thickness. 2 rows of c.c blocks of size 1.5mx1.5mx0.9m having 10cm gaps filled with blinding layer shall be provided in length equal to 3.1m.and 2m launching apron. And no need of protection work at the upstream.

The figure shown below is the section drawing of proposed under sluice on the right side of the cross section or which is constructed on rocky foundation.

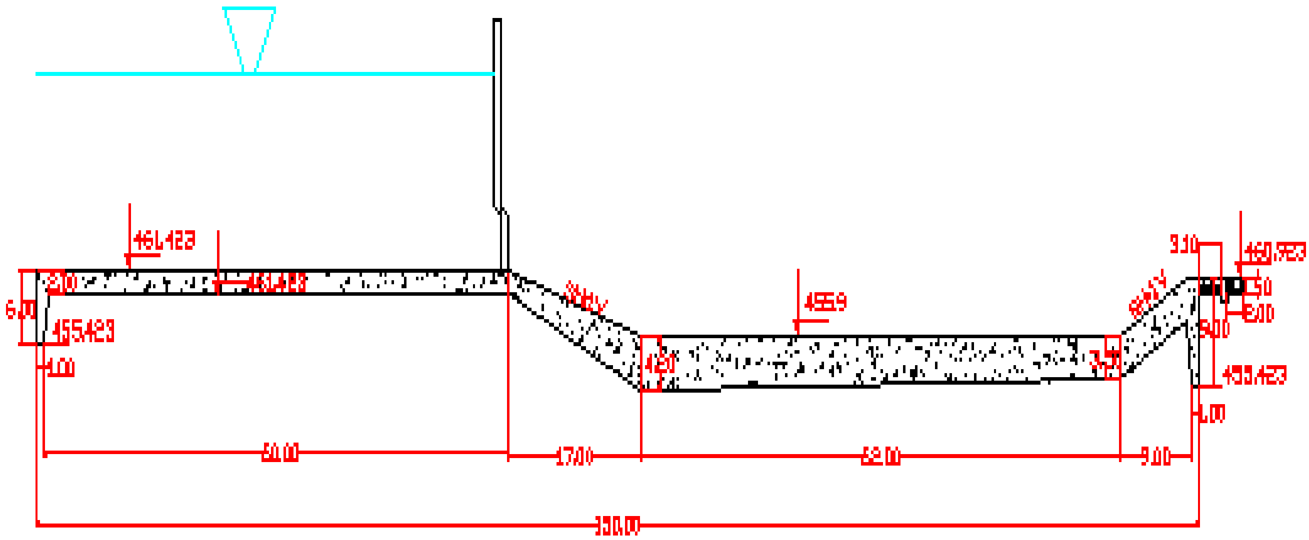

Figure 6-8. Under sluice on rocky foundation

6.5 Embankment Dam Design Criteria

- Embankment and foundation stability under all reasonably postulated conditions, including flood and earthquake;
- Control of seepage quantities and pressures in the embankment and its foundation;
- Sufficient freeboard and Safeguard against overtopping must be provided for wind set up, wave action, frost action and earth quake motions.
- Control of surface erosion
- A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost.
- Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.

The seepage line or phreatic line should remain well within the downstream face of the dam, so that no sloughing off the face occurs. The pattern of seepage through an embankment and its foundation will be influenced by the reservoir operating regime and the vertical and horizontal permeability"s of materials in the dam and in the underlying foundation. There should be no possibility of free flow of water from the upstream to the downstream face. The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain etc. The upstream face should be properly protected against wave action, and the downstream face against waves up to tail water.

The stability of the embankment and foundation is very critical during construction or even after construction(i.e. during the period of construction),due to development of excessive pore pressures and consequent reduction in shear strength of soil, the embankment slopes must remain safe under this critical condition also. The upstream and downstream slopes should be so designed as to be under worst conditions of loading. These critical conditions occur for the upstream slope during sudden drawdown of the reservoir, and for the downstream slope during steady seepage under full reservoir.

The upstream and downstream slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength of the soil, so as to provide a suitable factor of safety. All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of embankment dams.

6.5.1 Dam Zoning

The two selected options of embankment dams are:

- The first alternative is a zoned earth fill dam with an impervious central "Core" composed of clay core. This zoning is selected because of the material availability, foundation condition and dam height.
- The second alternative is a zoned rock-earth fill dam. This dam has the advantage of decreasing the volume of the fill, foundation conditions and the fill materials are easily available.

But the 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.

6.5.2 Dam Height and Crest Elevation

One of the basic requirements for design of an embankment dam is to ensure safety against overtopping caused by wind induced tides and waves by providing adequate freeboard. Additional allowances for settlement of the foundation and embankment (known as camber) also considered.The side slopes of the different dam alternatives were determined by Stability Analysis Program, accounting for each of the different construction materials and considering all design criteria conditions.

Under all conditions of construction, Dam operation, and seismic activity, the embankment and foundation must remain stable; Seepage through the embankment and foundation must be properly controlled and collected to prevent excessive uplift pressures, piping, sloughing and removal of material by solution or erosion of material by loss into cracks, joints, and cavities are considered in this Dam design.

The following criteria have been considered in the Dam height design:

- Storage reservoir elevation for $Q_{10,000}$ = 477.655m
- Pond level = 477.723 m depend on the canal full supply level
- Flood surcharge safety = h_f = 15.57m

Freeboard design

Freeboard is the vertical distance from the top of the embankment to the level of the spillway. It is most important than adequate depth be provided. If the depth is insufficient, floods will overtop the dam and the embankment material will be carried away at a progressively greater rate and extensive damage will be done. Many cases of complete dam failure have resulted from insufficient freeboard.

Freeboard must not be less than the dimensions specified on the dam works permit. The absolute minimum is 0.50 meters, usually with an additional 0.25 meters to take into account potential wave action.Hence, Settlement of soil banks is common and an allowance must be made for settlement of the dam embankment. The embankment may settle to a level where it is overtopped by water and failure will result. Or overtime settlement may result in the height of the embankment becoming lower than the spillway. Clay soil can settle in excess of 10% of the dam‟s height, but well-constructed and compacted clay dam embankments are not likely to settle more than about 5%. An allowance of 5% of the height of the embankment (along its length) to cater for settlement is necessary [17].

Taking settlement effect 5% of Dam height = 5%*32.12 = 1.6m

Top of Dam crest elevation = 477.723 + 15.57 + 1.6+0.25 = 495.143m a.m.s.l

From the elevation capacity curve bed level/bottom of the dam site = 461.423

Dam height = crest elevation – bottom elevation

= 33.72

Take 34 m at top of dam elevation 495.423m a.m.s.l

6.5.3 Top width (crest) design

The top width of an embankment dam may be governed by requirements for passage of vehicles, both during construction and in service. Generally the top width of the dam can be selected as per the following recommendations.

The width of the dam at the crest should be fixed according to the working space required at the top. No dam should have crest width of less than 4.5 m [8].
Other recommendation [4] is :

According to Guidelines for Design of Dams (1989) the minimum allowable top width (T) of the embankment shall be the greater dimension of 10 feet or T, as calculated by the following formula:

$T = 0.2p + 7$; where p is the height of the embankment (in feet)

$T = 0.2*72.178ft + 7 = 21.44ft = 6.535m$

Thus dam crest width is 10m, to accommodate a two lane road, with safety barriers on either side. A dam is 34m high above the river bed level.

6.5.4 Upstream and downstream slopes

The side slopes of the different dam alternatives were determined by Stability Analysis Program, 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. the recommended values of side slopes as given by Terzaghi are given below [4].

Based on the recommended values of after different trial slopes by checking the stability and economic consideration the shell is flanked by a 1V:3Hupstream slope and 1V:2.5H downstream slope free draining earth-fill for the first alternative and 1V:2H upstream slope and 1V:1.8H downstream for the second alternative.

Table 6.2 : Recommendations of side slopes of Embankment dams.

Type of material	u/s slope(H:V)	d/s slope (H:V)
Homogeneous well graded	2.5:1	2:1
Homogeneous course silt	3:1	2.5:1
Homogeneous silty clay i. Height less than 15m ii. Height more than 15m	 2.5:1 3:1	 2:1 2.5:1
Sand or sand and gravel with 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

6.5.5 Provision of Berms

The berm should slope towards the inner edge to prevent rain-water from flowing over the outer edge and down the slope of dam. A slope of 1 in 50 is recommended for this purpose. A minimum berm width of 3.0 m is recommended. However, 5 to 6 m width is desirable. One berm for every vertical elevation of about 10 to 15 m is recommended [12].

However, this Project dam has considerable height and in order to operation and maintenance of the dam, a total of 2 berm at 11 m and 12m vertical interval is provided on the downstream slope and 5 m wide and will be inclined at a slope of 1 in 50.

6.5.6 Core design

Core width

The core width for a central impervious core-type embankment should be established using seepage and piping considerations, types of material available for the core and shells, the filter design, and seismic considerations.

In general, earth and rock fill dam the width of the core at the base or cutoff should be equal to or greater than 25 percent of the difference between the maximum reservoir and minimum tail water elevations [11].

= 0.4*(493.293-462.033) = 12.504 take 13m

The greater the width of the contact area between the impervious fill and rock, the less likely that a leak will develop along this contact surface. Where a thin embankment core is selected, it is good engineering to increase the width of the core at the rock juncture, to produce a wider core contact area. Where the contact between the impervious core and rock is relatively narrow, the downstream filter zone becomes more important.

A core top width of 10 ft (3m) is considered to be the minimum for construction equipment. The maximum core width will usually be controlled by stability and availability of impervious materials. A core top width of 6m is considered for safety of construction equipment and material availability.

Its thickness at any section shall not be lesser than 30 percent (preferably not lesser, than 50 percent) of maximum head of water acting at that section [22]. The top level of the core should be fixed at 0-5 m above MWL. Take 1m

In zoned dams, it is common practice to limit the height of the core material to a few feet below the crest because impervious zones extending to the top of the Dam are subjected to damage by desiccation and frost action, which causes loosening and cracking of the soil. Zoning around the top of the impervious core should be provided or additional core height above the maximum water surface provided to control seepage through the embankment, the first solution being preferred. The plasticity index of the core material should be sufficient to allow the core to deform without cracking.

6.5.7 Cut - off

The cutoff material should be placed in layers to a maximum 50-75mm thick and to a minimum width of 1m for small dams (i.e. hand laid cores) and layers 75-150mm thick and 2-3m wide for larger dams i.e. material laid by scoop or scraper and compacted by machinery [3].for this particular case 3m is adopted.

The figure shown below is the cross sections of the first alternative earth fill with clay core dam which will be constructed on left part of the site on gravel and sandy silt foundation.

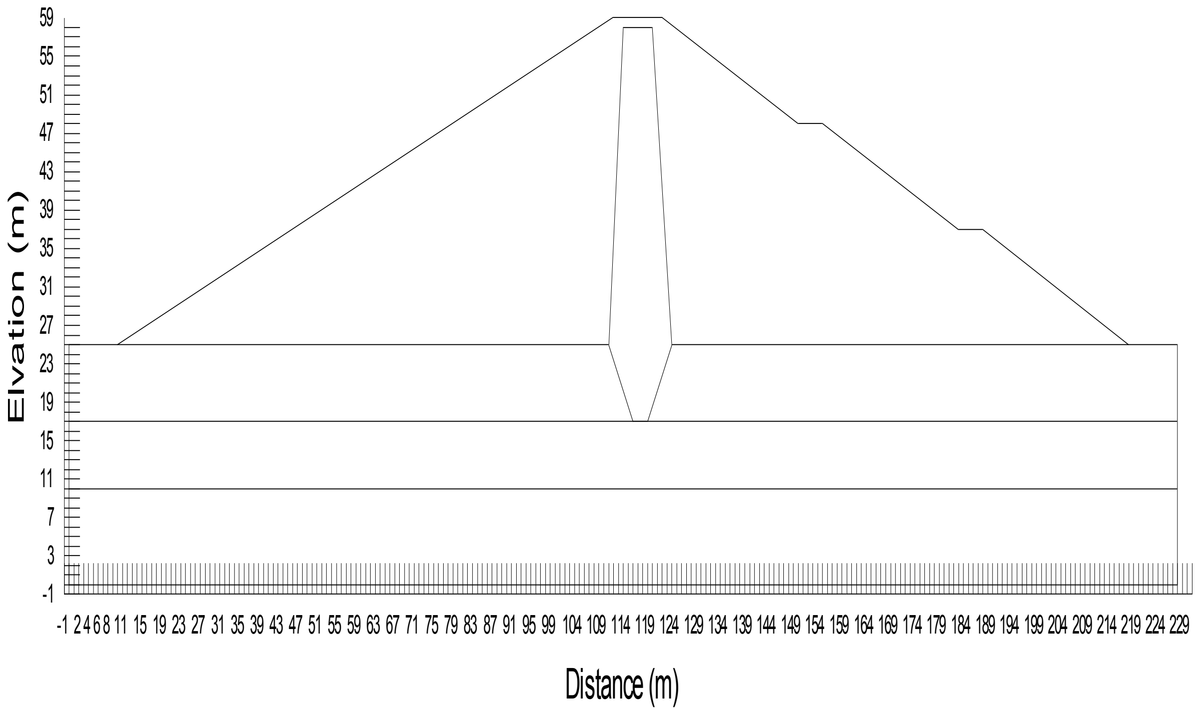

Figure 6-9. Typical sections of earth fill Dam

The figure shown below is the cross section of the second alternative earth fill-rock fill with clay core dam which will be constructed on left part of the site on gravel and sandy silt foundation.

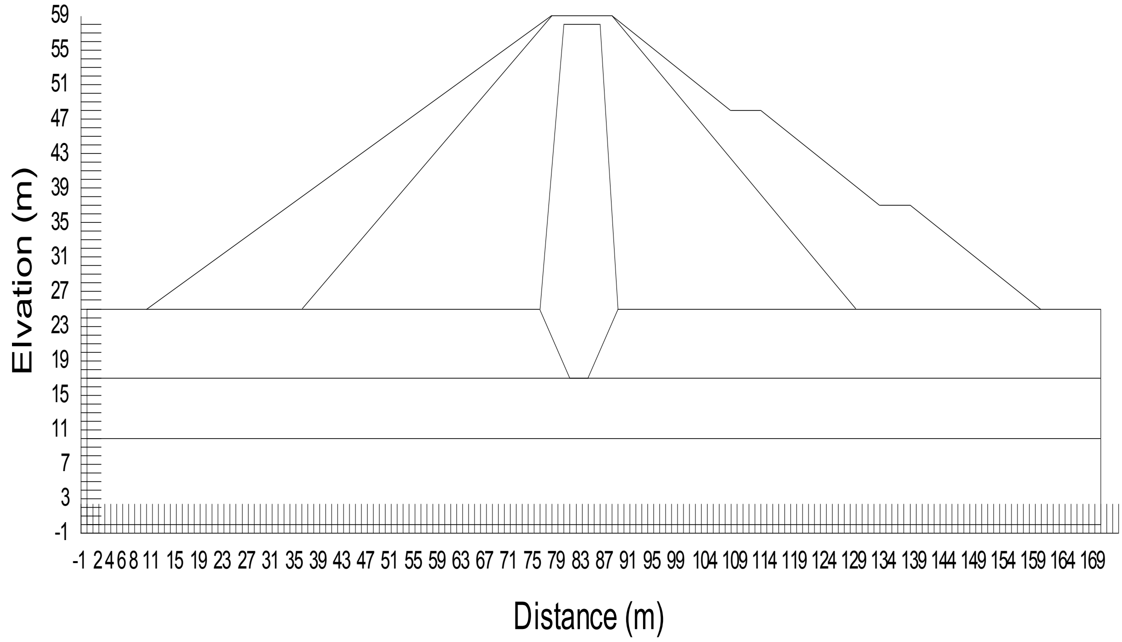

Figure 6-10. Typical sections of earth-rock fill Dam

7 DATA USED

The basis for the diversion structures analysis is the availability of accurate and adequate data on hydrological, meteorological, geophysical, geotechnical, material characteristic, Construction practice, geometry, economy, damage prevalence etc. For the case study diversion structures alternatives in this research secondary data has been collected from various sources: previous studies, design documents, different books and site visits. The data collection process for this research has been difficult because the proposed structure is under construction. This arrangement gave the researcher easy access to study documents, field engineers‟ consultations.

7.1 Resistance Related Data

7.1.1 Data on shell and core material characteristics for different alternatives

As mentioned above, considerable amount of secondary data on core and shell materials characteristic of Kuraz project has been collected. However, it was short of providing sufficient data to do the analysis of different alternatives.

Table 7.1: Proposed alternative diversion structures load and resistance data sources and methods acquiring.

Data Type	Method of data acquisition
Resistance related parameters	
shear strength parameter,c, φ, c' and φ ' , for core and shell materials	secondary data
permeability, k, for core and shell materials	secondary data
specific gravity for core and shell materials	secondary data
elevation of Pond, topography, geometry and dam dimensions	secondary data
Load related parameters	
Hydrology data	secondary data
Pore water pressure	Simulation
Sediment loading	secondary data

7.1.2 Permeability Data

The permeability data were taken from the previous study documents and different standard books for each type of core, shell and foundation layers. All the permeability data and their sources are described in Appendix 11.2.1.

7.1.3 Shear strength Data

The Shear strength Data were taken from the previous study documents and different standard books for each type of core, shell and foundation layers. From these documents to represent different drainage conditions, the samples were tested in the triaxial unconsolidated undrained and direct shear test. For the previous document the test was done only for the required soil type which is the concrete weir. Therefore the remaining important data and their sources are described in Appendix 11.2.1.

7.2 Structures Dimensions, Topography and Pond Elevation

Data on different alternatives of the diversion structures and appurtenant structures dimensions is taken from different design standards and books. Whereas topography and elevation of pondage is taken from Kuraz project design documents.

7.3 Load Related Data

7.3.1 Discharge data

There are several standards that are in use for design of dams against flood and wave overtopping. Much of these standards are mostly the selection of „safe‟ design flood is based on estimation of annual exceedence probability that corresponds to an acceptable level of flood risk with respect to human and economic consequences of failure. The highest standard uses the probable maximum flood (PMF), i.e. the extreme flood that is physically possible, which result from severe most combinations of meteorological, like the probable maximum precipitation (PMP) and hydrological factors. However, in agricultural areas where failure would result only in flooding of crops, a design for a much smaller degree of protection could be reasonable. When conditions lie between these two extremes, varying design flood of certain probability (return period) (e.g. Q_{150}, Q_{1000}, $Q_{10,000}$) will apply for the relevant level of dam safety.

.As it was done on the draft design document of kuraz project the design discharge estimation were done using two different cases. These two cases used were design flood estimation With Gibe Ⅲ Dam and without Gibe Ⅲ dam. Therefore, the earlier designed weir structure has been re-designed recently in September 2012 with 100 years flood magnitude of Q_{100} = 7,931 m^3/sec [5]. And for the embankment dam section $Q_{10,000}$ magnitude 12900 m^3/sec is adopted because of the reasons specified above in chapter 4.

7.3.2 Sediment data

Sediment transport rate data is useful in determining the useful life of dam, i.e. fixing the dead storage level at the end of the design period for deciding the location of sluices, and it is also required for fixing reservoir operation rules. The total amount of sediment volume inflowing to Kuraz head work estimated about 26 Mm^3/year. However, this much volume is in the pre Gibe Ⅲ dam scenario. When Gibe Ⅲ dam becomes functional, it will detain 30% of the sediment volume originating from its own catchment i.e. 5.49Mm^3/year. As per the study of Agri-consulting SpA-Mid-Day International Consulting Engineers the incoming sediment load composed of fine particles (colloids, clay, fine silt), that remains in suspension and will therefore continue to be carried downstream. Thus the net sediment volume coming to our headwork will be 21 Mm^3 [24].

7.3.3 Pore water pressure data

In this research the analysis of seepage and pore-water pressure distribution is done using the computer program Geo Studio 2004 (SEEP/W).

7.3.4 Earthquake Data

Earth quake records often have some vibrating noise at the start of the record and at the end of the record .Altering the input motion record to the bare minimum is effective in mitigating the required computing time, especially during the early stage of a project. Later ones the response is fairly well understood, the analysis can be repeated with the entire record to refine the analysis. The seismic hazard can be defined as the probability that a ground motion at a specific site will be exceeded during a time period, i.e. return period, T (years).

For large dams the return period of the Maximum Design Earthquake is taken as 10,000 years. For dams with small or limited damage potential shorter return periods can be specified. An alternative interpretation is a 10% probability that this PGA will be exceeded in 50 years. For the 0.2 second period for instance, for the T=100 and 500 years return periods it can alternatively be interpreted as 10% probability of exceeding the PGA values of 0.18g and 0.31g once in 10 and 50 years, respectively.

This estimate is for soil site which can be considered as the worst case scenario. If it is a rock site, for return period of T = 100 and 500 years the PGA values are 0.15g and 0.2g, respectively, for 0.2 second period (Table 7.2).Ground motion amplitude in percentage of gravity (%g) for rock and soil sites at different period of motion and for different return periods at factory site two of the Kuraz Sugar Development Project.

Table 7.2: Ground Motion Amplitude in Percentage of Gravity (%g) for Rock and Soil Sites.

Return Period in Years	Ground Motion Amplitude in % of g for Boore-Joyner-Fumal (1993, 1997)					
	Period = 0.2 sec		Period = 1.0 sec		Period = 2.0 sec	
	Rock	Soil	Rock	Soil	Rock	Soil
50	12.15	13.83	4.61	5.70	3.26	3.88
100	15.82	18.20	5.51	6.95	3.82	4.63
200	20.51	23.33	6.58	8.46	4.48	5.53
500	27.44	31.46	8.33	10.98	5.53	6.97
1,000	33.53	38.47	9.96	13.38	6.48	8.31

For this project the horizontal earth quake is a destructive force and the earth quake record shown below is considered to be representative of the excitation. The record has a peak acceleration of about 0.182g .the duration of the record is 0.2sec. For dynamic analysis purpose it was selected 0.182, 0.0695 and 0.0463g amplitude for the soil part at 0.2,1 and 2 seconds period respectively.

QUAKE/W is formulated for direct integration in the time domain. This means the analyses are performed at many different moments in time and at certain time intervals. In QUAKE/W the integration follows a specified time stepping sequence. For this particular study the sequence consists of 500 steps at a constant interval of 0.02 seconds, making the total elapsed time 10seconds.The data, however may only be saved to file every tenth time step. There will consequently be 50 sets of data files.

7.3.5 Damping Ratio

During the soil layer seismic response analysis, the difference of dynamic modulus and damping ratio will affect earthquake ground motion greatly. Due to anisotropy of soil itself, experiment error, testing model error, statistical error, etc., the uncertainty of the two soil dynamic parameters can be quite large. The measured damping ratio was relatively unaffected by the number of cycles and only moderately affected by the soil improvement technique. over a wide range of inertia ratios the damping ratios range between 10 and 50%. With the typical range of interest for foundations under cyclic horizontal loading, the range is between 20 and 35%.for this particular project site there were no tested data [2]. Hence damping ratio 0.1 is adopted.

8 ANALYSIS OF SOFTWARE

For the analysis of the proposed alternatives using Geo-Studio 2004 which is applying the modern limit equilibrium method and new integrated tool for running GEO-SLOPE‟s leading suite of geotechnical modeling software products: SLOPE/W, SEEP/W, SIGMA/W, QUAKE/W. Using this software will increase the types of problems analyzed and make it easier to model geotechnical problems on a routine basis.

A key benefit to Geo-Studio is that all products now run in a single environment. This means that your model definition is shared among all products. For example, when you define boundary conditions and material properties in one product, they are available immediately to other products that use them. The shared analysis data lets you run many analyses on the same problem. You can use the results from one analysis in another, or you can use results from an external problem. You can still import files created by previous versions of the software.

Since modern limit equilibrium soft wares like Geo Studio gives better result and making it possible to handle ever increasing complexity within any analysis. Hence all the analysis below is based on Geo-slope, 2004 software. The loading conditions and boundary conditions stated below in table 8.1 are used to analyse seepage, stability, stress and settlement of a dam.

Table 8.1: Loading conditions and boundary conditions

S.No.	Analysis methods	Loading conditions	Boundary conditions
1	Seepage Analysis	Steady state seepage Steady state seepage with earthquake	On upstream side total head At the downstream there is horizontal filter. The granular material is highly permeable; This implies that there is no head loss in the drain relative to the small amount of seepage that will come through the low permeability embankment material. The total head around the rock toe perimeter and beyond is therefore assumed to be the elevation of the original ground surface (OGL).
2	Stability Analysis	End of construction Sudden drawdown Steady state seepage Steady state seepage with earthquake	Initial conditions
3	Stress and Settlement Analysis	Steady state seepage with earthquake	Displacement is zero in x-direction Displacement is zero in x and y-direction along the bottom horizontal boundary

8.1 Seepage Analysis and Filter Design

One of the basic requirements for design of embankment dams is to ensure safety against internal erosion, piping and development of excessive pore pressures in the dam. For this, vertical chimney drain composed of fine and coarse filters between the core and the shell have been used. To safely discharge the seepage water from the vertical chimney and to protect erosion of fines from the downstream shell, a horizontal drain composed of a coarse filter sandwiched by two transition filter layers is also provided.

8.2 Seepage Analysis

The Finite Element Models used in the analyses and the computed discharges are as shown on respective Figures. These discharges are used to calculate the Filter thickness. Besides, the computed phreatic surface as shown by the blue line is used to set up the pore water pressure line in the stability analysis of the dam.

The cross section of foundation layers, core and earth fill section of the first alternative using Geo-studio soft wares which is ready for the analysis is shown below in figure 8-1.

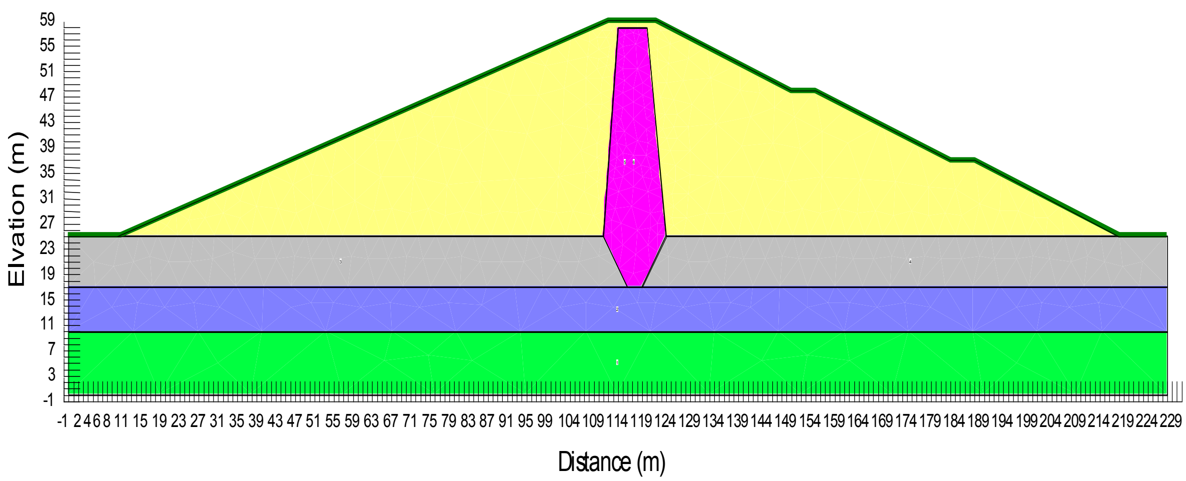

Figure 8-1. Foundation layers and Dam cross-section

The figure shown below is the result after analysis Using SEEP/W software and the blue line indicate the phreatic surface or seepage line.

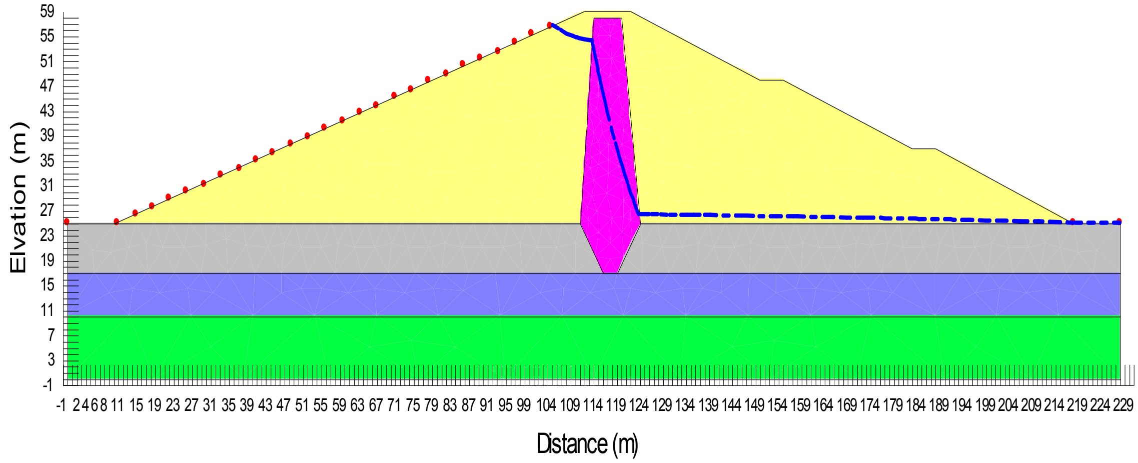

Figure 8-2. Seepage profile.

The figure shown below is the result after analysis of the first alternative earth fill dam Using SEEP/W software and the blue line indicate the phreatic surface or seepage line. And the number shows the afflux through the dam body only. This afflux is important to design the vertical chimney drain.

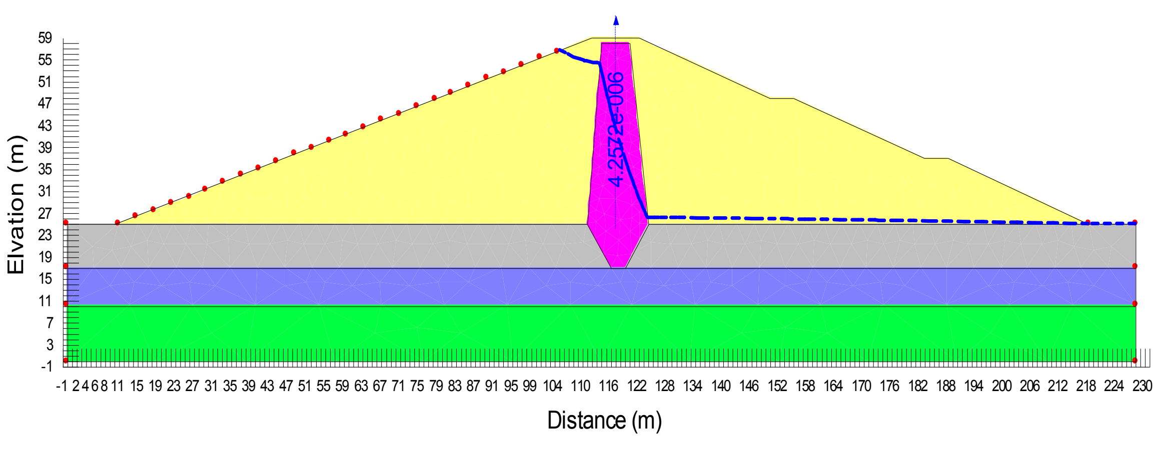

Figure 1 Seep/W analyses result for seepage through the dam only.

The figure shown below is the result after analysis of the first alternative earth fill dam Using SEEP/W software and the blue line indicate the phreatic surface or seepage line. And the number shows the afflux through both the dam body and foundation. This afflux is important to design the horizontal drain.

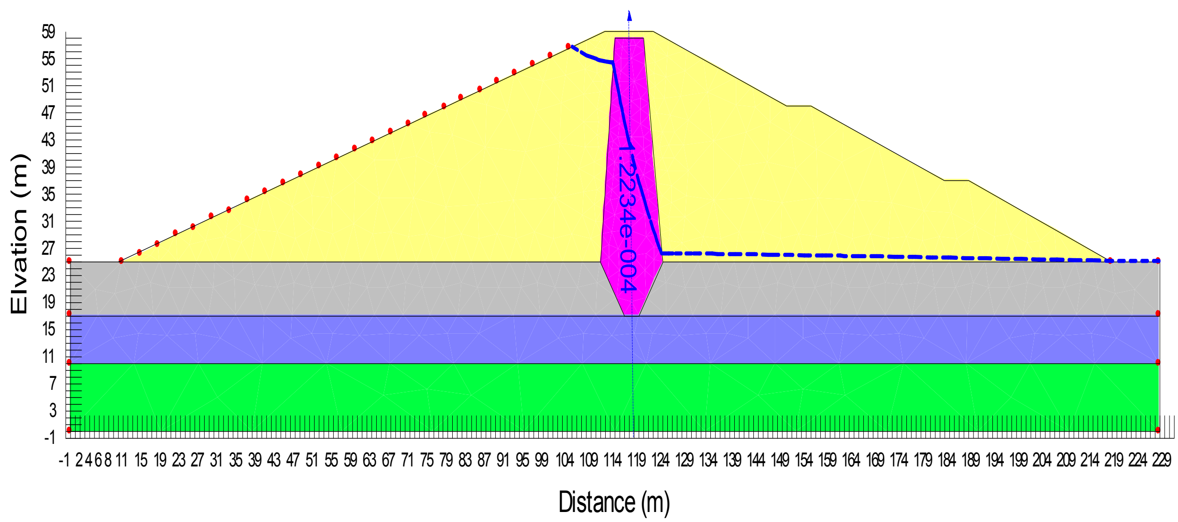

Figure 8-4. Seep/W analyses result for seepage through the dam plus the foundation.

8.3 Design of Seepage Control for the Embankment and Foundation

8.3.1 Design of Vertical Filter

The soils having the coefficient of permeability greater than 10^{-3} mm/s are classified as pervious and those with a value less than 10^{-5}mm/s are impervious. The soils with the coefficient of permeability between 10^{-5} to 10^{-3} mm/s are designated as semi-pervious [8].

Discharge, q = 4.2572*10^-6 m^3/s/m length (from seepage analysis by SEEP/W software). 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 = sinα = 1.

4.2572*10^-6 = 1.0 × 10^{-4} ×1 × t $\Rightarrow$ t = 0.0426m

Considering a factor of safety of 2, t = 0.085m.

However, filter design for chimney drains and vertical or inclined filters in embankment 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 minimum width of 2.4 to 3m vertical coarse filter. However, width up to 4.9m horizontally is preferred by contractors when bottom dumped loaders are used to place the material. Economic considerations sometimes dictate the use of width as narrow as 1m [8].

For this case let‟s provide 2m.In addition to the coarse filter a 2m fine filter should be provided as transition between the impervious core and the vertical filter.

8.3.2 Design of Horizontal Filter

Total discharge through dam & foundation, q = 1.2234*10^-4 m^3/s/m length (from seepage analysis by SEEP/W software). Permeability of filter is at least 25 times permeability of base material [8]. Hence take k = 1.0 × 10^{-3}m/s,

Length of blanket filter = 94 m

Thickness of blanket filter, t, is given by

$q = \frac{kt^2}{L}$ $\Rightarrow$1.2234*10^-4 = 1.0 × 10^{-3} m/s × t^2 /L

$\Rightarrow$ t = 3.39m.

It is proposed to provide a 3.5m thick coarse blanket filter. Moreover, the coarse blanket filter should be sandwiched by two 1m thick fine filters in order to protect internal erosion of fines from the shell.

8.4 Dam stability Calculations

8.4.1 Loading Conditions

After analysis of seepage through the dam and the foundation, by using SLOPE/W software one can analyse the stability of dam options for different loading conditions. Table 8.2 below summarizes the loading conditions and corresponding minimum factor of safety (FOS_{min}) requirements [13].

Table 8.2: Minimum required factors of safety for new earth and rock-fill dams.

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

8.4.2 Steady State Seepage Condition

The stability analysis for downstream slope under steady state condition has been checked by considering NWL or pond level for both: normal loading condition and with earthquake loading condition. The phreatic surface computed with the help of Seep/W in figure 8-5 below was used to set up the pore water pressure line in the stability analysis.

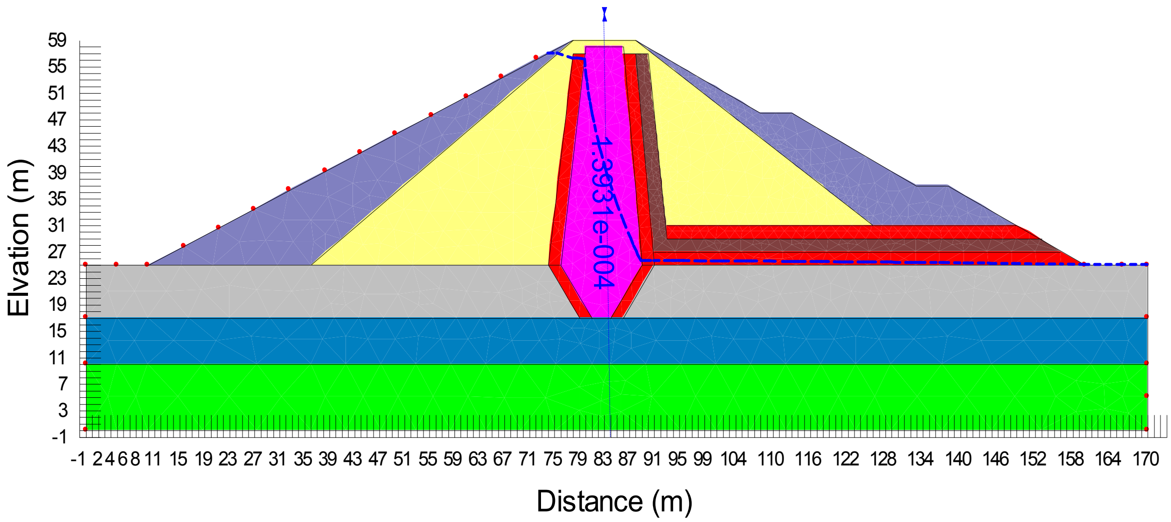

Figure 8-5. Seep/W analyses result for seepage through the dam plus the foundation.

8.4.3 Sudden Drawdown Condition

The drawdown condition is a classical scenario in slope stability, which arises when totally or partially submerged slopes experience a reduction of the external water level. Rapid drawdown conditions have been extensively analyzed in the field of dam engineering because reservoir water levels fluctuate widely due to operational reasons.Sudden drawdown stability computations were performed for the upstream slope for conditions occurring when the water level adjacent to the slope is lowered rapidly. For analysis purposes, it was assumed that drawdown is very fast, and no drainage occurs in materials with low permeability. For this Dam, in the impervious core and in the earthen shell, no drainage is assumed during sudden drawdown. The reason to assume no drainage during sudden drawdown in the shell zone is due to the presence of considerable fines in the shell materials.

Drawdown rates of 0.1 m/day are common and drawdown rates of 0.5m/day are quite significant. One meter/day and higher rates are rather exceptional [27].

Interestingly, in most of the experiences and reported failures the drawdown did not reach the maximum water depth but approximately half of it (from maximum reservoir elevation to approximately mid-dam level) [27].

8.4.4 During and at the End of Construction Condition

Computation of stability during and at the end of construction was performed using drained strengths in free-draining materials. For materials that drain slowly two alternatives can be used: Total stress analysis with undrained strengths and zero pore water pressure or Effective stress analysis modelling partially saturated condition with pore water pressure. The pore pressures developed within the body of the dam and in the foundation under steady state seepage has been initially estimated with the help of the SEEP/W software. These pore pressures in terms of head have been incorporated in the slope stability analysis. SEEP/W analysis results were earlier shown in Figure 8-5 above.

8.5 Analyses Results of Alternative-1 Embankment Sections

The basic consideration in this Dam has been to achieve safety consistent with economy. For this purpose, several embankment sections will be analysed. For the first alternative the selected dam type was an earth fill dam with central core.

8.5.1 Stability Analysis Results

The computed factors of safety against slope failures under the different loading conditions for the selected dam type are summarized in Table 8.3.

Table 8.3 : Computed factors of safety.

Loading Condition	**FoS_{min}**	**Computed Factors of Safety**	
		downstream	Upstream
During construction	1.3	2.200	2.676
End of construction	1.3	2.110	2.497
Steady state seepage	1.5	2.459	-
Steady state seepage with earthquake	1.1	2.299	2.130
Rapid drawdown	1.3	–	1.535

The downstream slope stability analysis of an earth fill dam under steady state seepage without earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

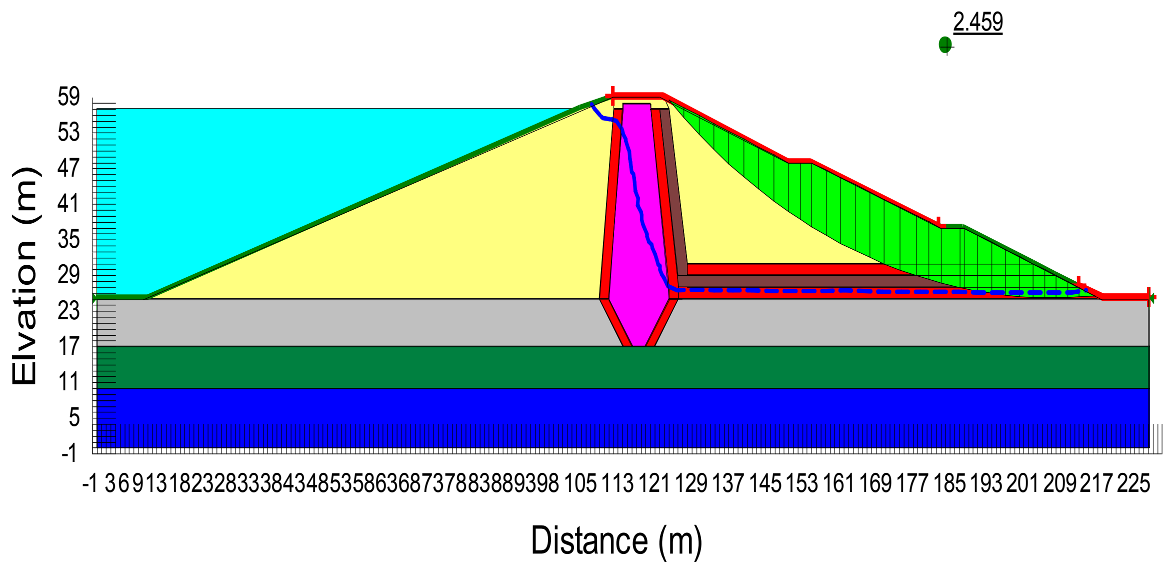

Figure 8-6. Steady state seepage without earthquake (downstream slope).

The downstream slope stability analysis of an earth fill dam under steady state seepage with earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

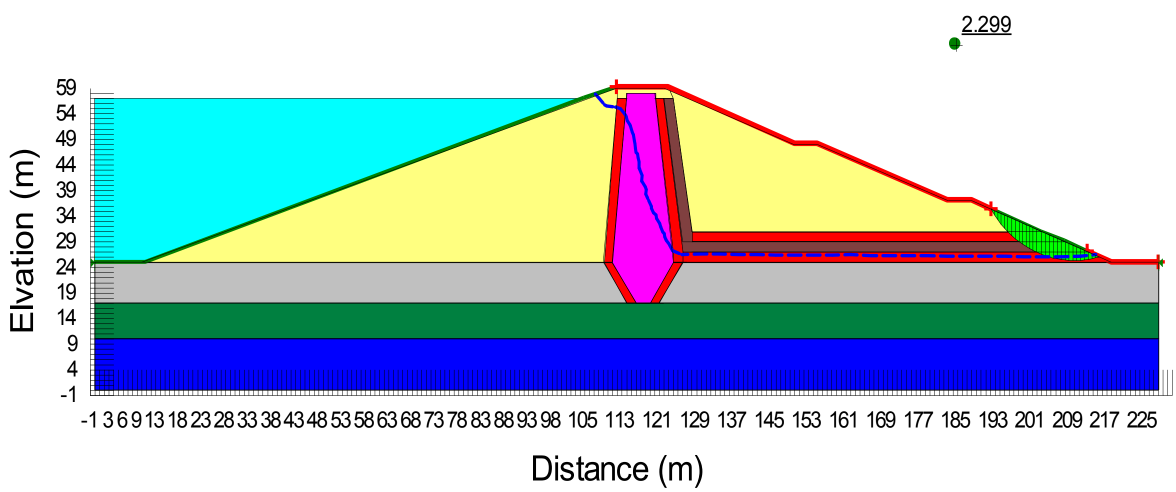

Figure 8-7. Steady state seepage with earthquake (downstream slope).

The downstream slope stability analysis of an earth fill dam during construction is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

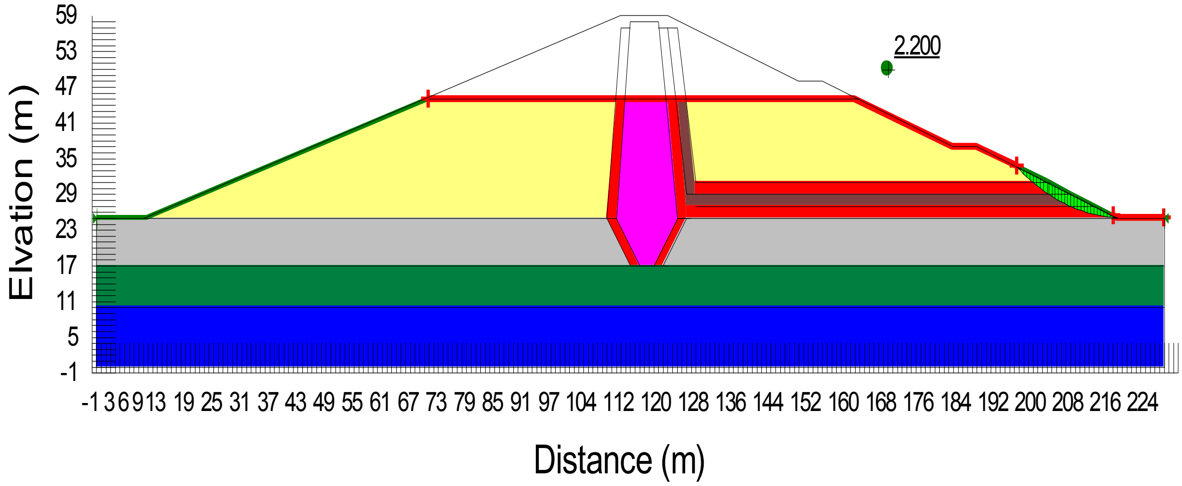

Figure 8-8. During construction condition (downstream slope).

The downstream slope stability analysis of an earth fill dam at the end of construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

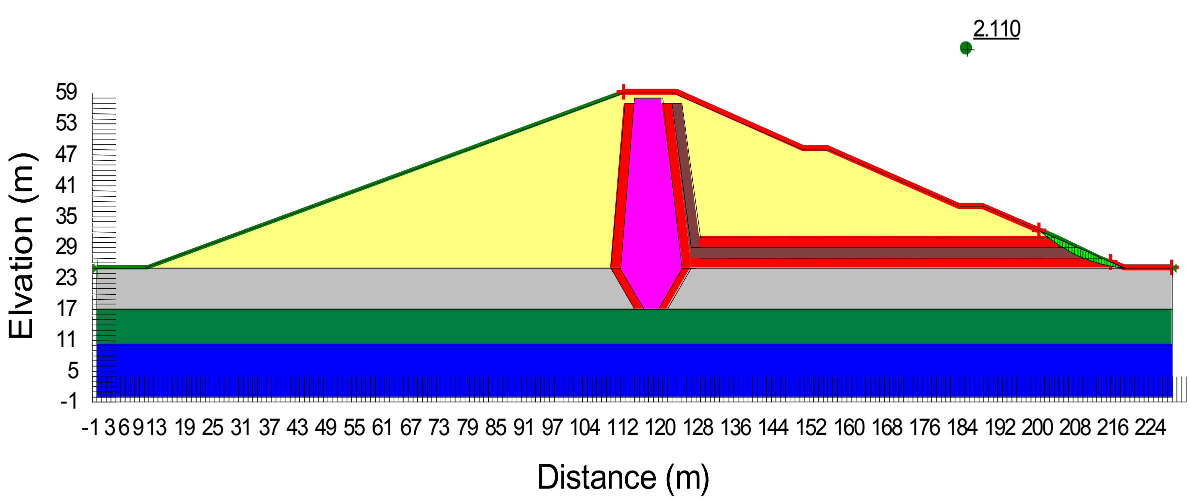

Figure 8-9. End of construction condition (downstream slope).

The upstream slope stability analysis of an earth fill dam under steady state seepage without earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

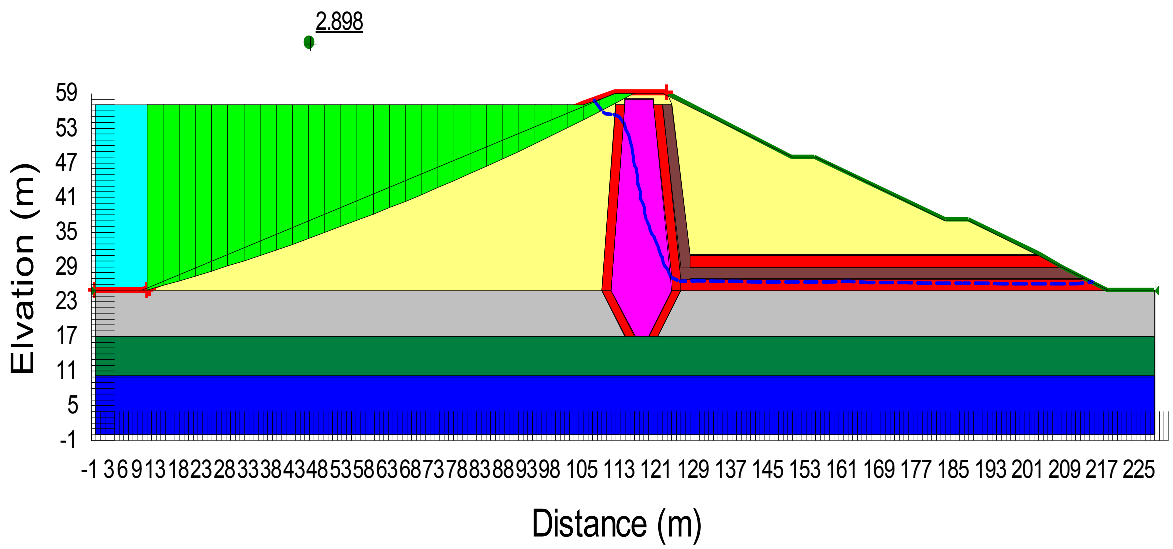

Figure 2 Steady state seepage without earthquake (upstream slope).

The upstream slope stability analysis of an earth fill dam under steady state seepage with earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

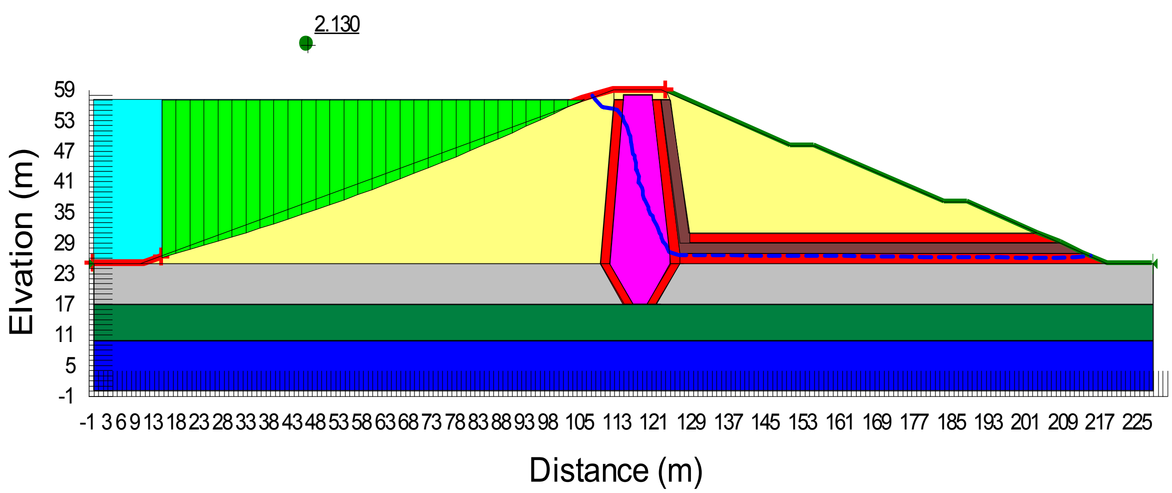

Figure 8-11. Steady state seepage with earthquake (upstream slope).

The upstream slope stability analysis of an earth fill dam under sudden drawdown condition is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

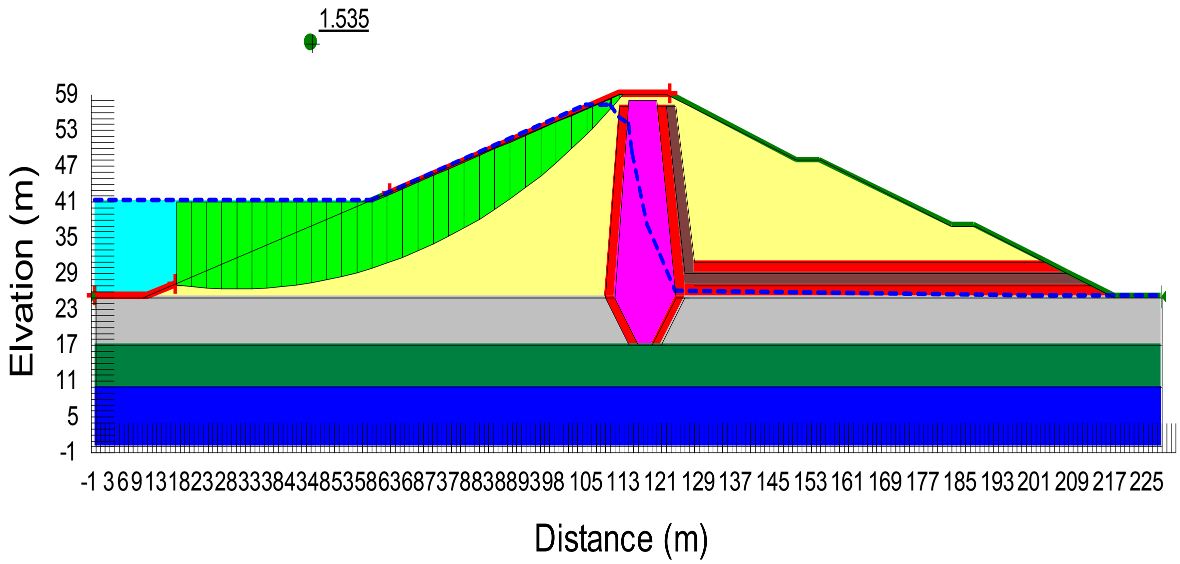

Figure 8-12. Sudden drawdown condition (upstream slope).

The upstream slope stability analysis of an earth fill dam during construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

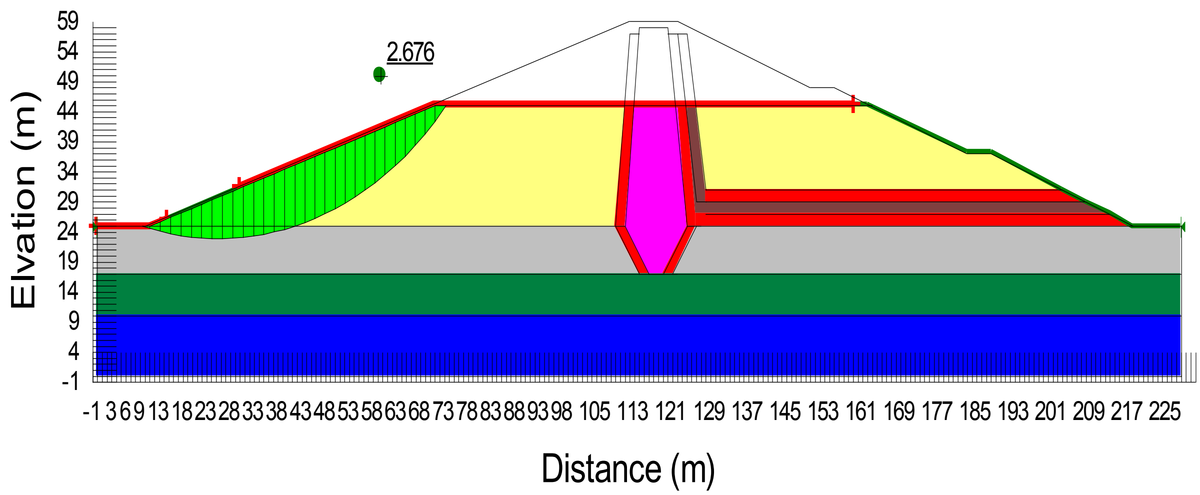

Figure 3. During construction condition (upstream slope).

The upstream slope stability analysis of an earth fill dam at the end of construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

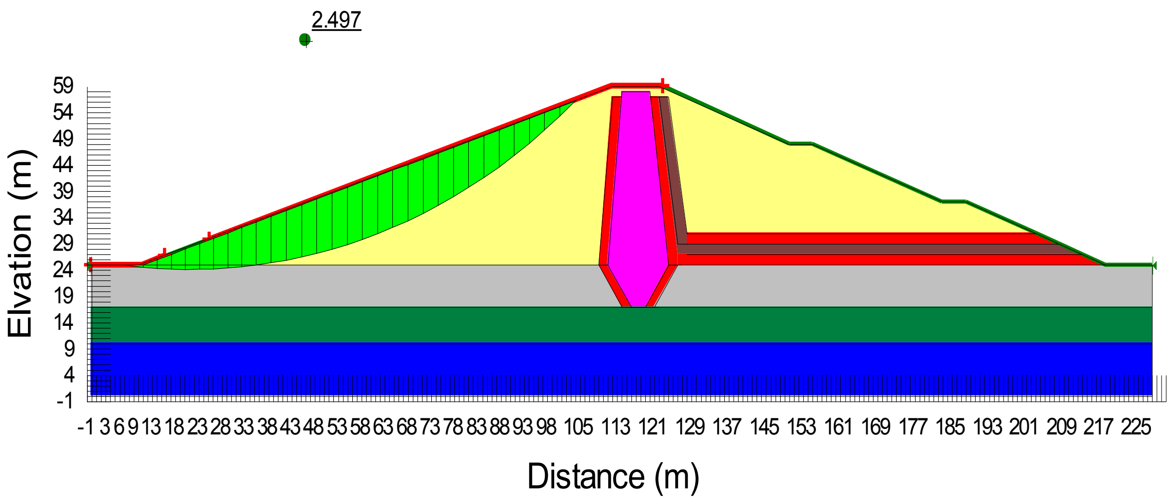

Figure 4 End of construction condition (upstream slope).

8.5.2 <u>Stress and Settlement Analysis</u>

Using the result after analysis of seepage through the dam and the foundation, using SIGMA/W software stress and settlement is analysed for both alternatives.

<u>Stress analysis</u>

Table 8.4: computed stress and presumed Bearing capacity for foundation materials.

Foundation materials	Total stress from the design kN/m² Earth fill dam	Total stress from the design kN/m² Earth-Rock fill dam
Layer-1	100	590
Layer-2	520	800
Layer-3	800	820

The stress analysis of an earth fill dam is shown below in figure 8-15 and figure 8-16 and the contour line indicates the total stress at different depth of the dam and foundation.

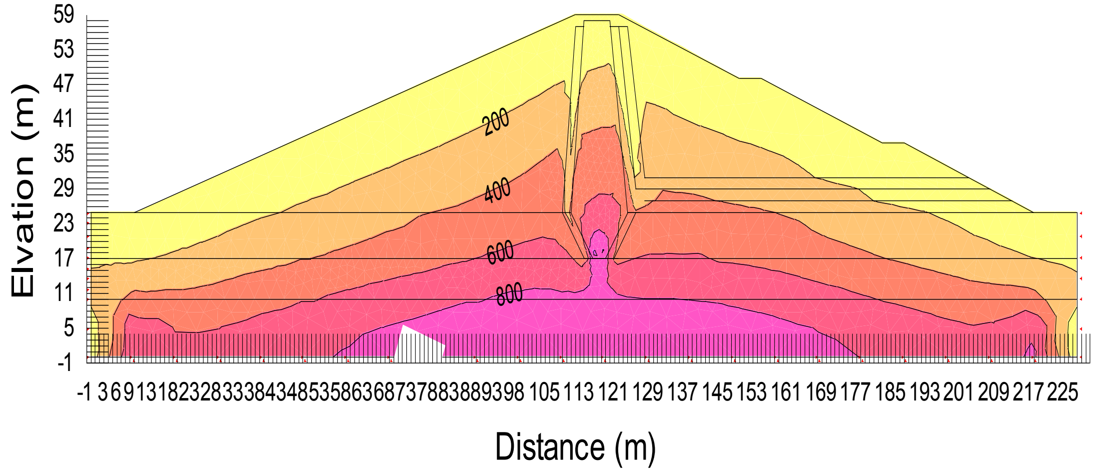

Figure 8-15. Total stress.

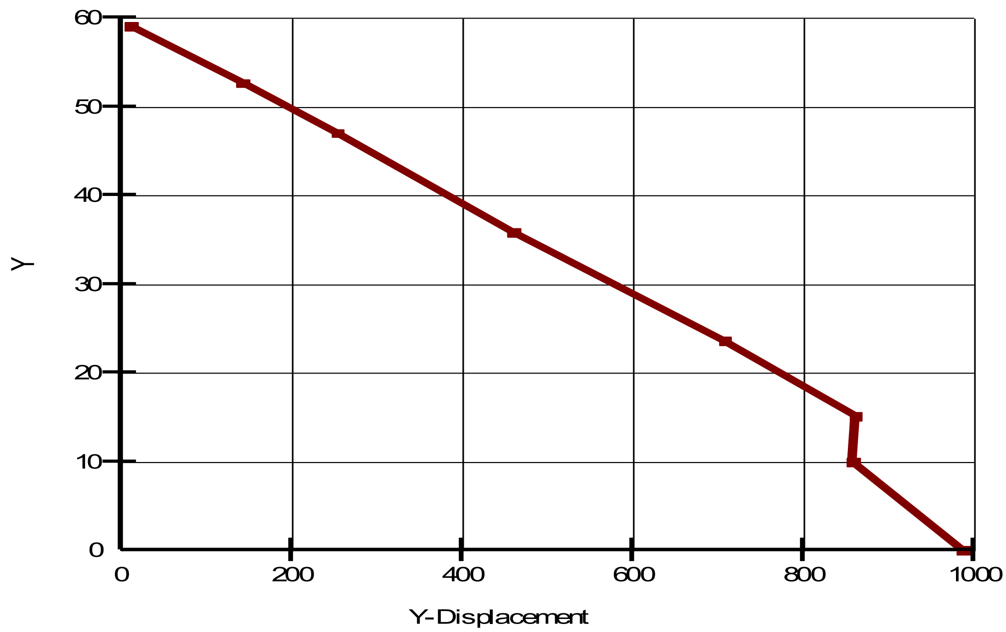

Figure 8-16. Graph of total stress.

Settlement analysis

The vertical displacement or settlement analysis of an earth fill dam is shown below in figure 8-17 which shows the settlement of the dam from its initial condition and figure 8-18 shows the vertical total displacement versus the vertical coordinate and the settlement result is important to check free board provided during dimensioning of the dam.

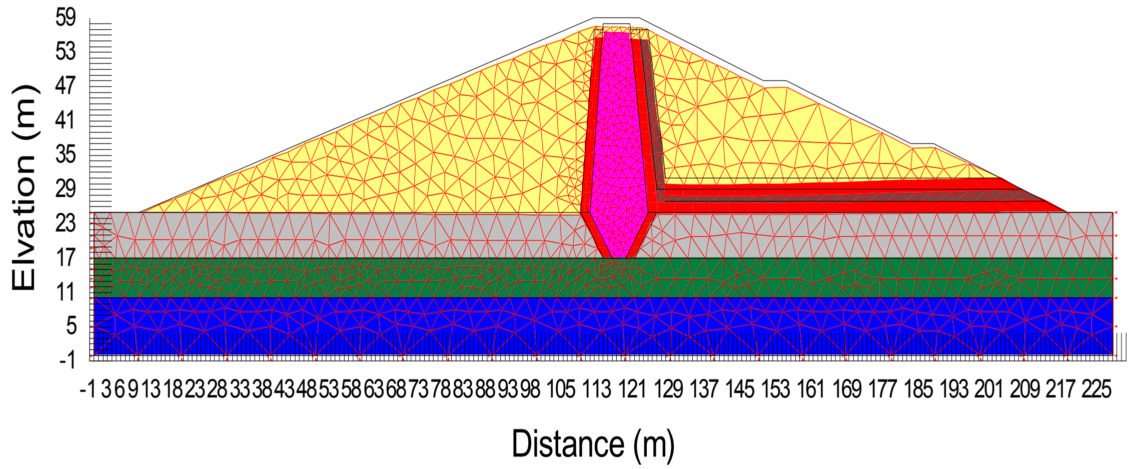

Figure 5 Vertical displacement.

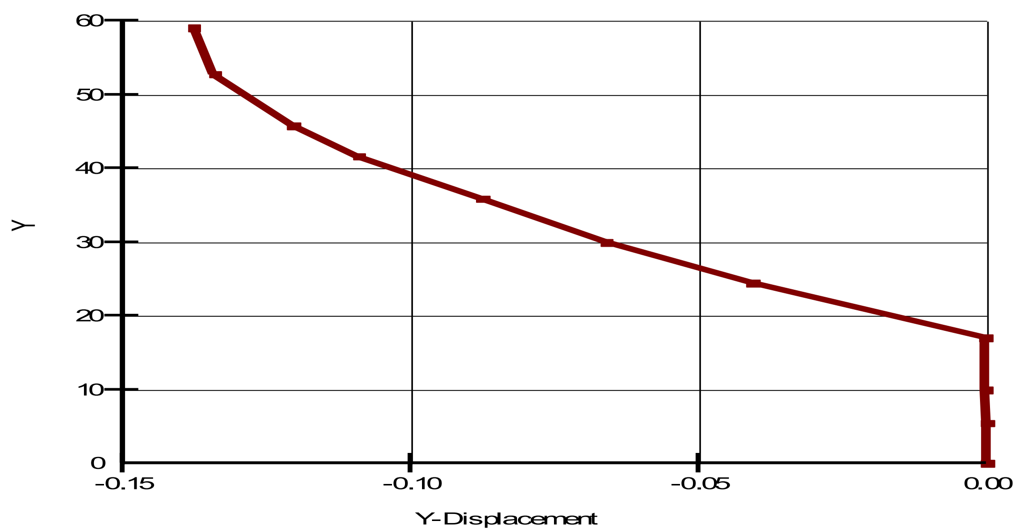

Figure 8-18. Total displacement.

8.5.3 Liquefaction Analysis

In evaluating the potential for liquefaction, the effect of settlements induced by seismic motions and loss of soil strength shall be considered. The analysis performed shall incorporate the effects of peak ground acceleration, appropriate earthquake magnitudes and duration consistent with the design earthquake ground motions as well as uncertainty and variability of soil properties across the site. Peak ground acceleration, seismically induced cyclic stress ratios and pore pressure development may be determined from a site-specific study taking into account soil amplification effects and ground motions appropriate for the seismic hazard. Numerical, liquefaction occurs very easily in a zone of low effective confining stress. Much of the upper half of the tailings has consequently liquefied

In a QUAKE/W analysis it is almost always necessary to establish the in situ stress conditions. Performing dynamic analysis is the main essence of QUAKE/W. the limits the pore-pressure that can be generated to the initial static confining stress.The thinking is that the Effective confining stress can not be less than zero.Further more, when the pore pressure reaches the effective confining stress the soil will be close to or in a liquified state.QUAKE/W allows to view zones where this condition has been reached.The figure shown below indicate the area susuptable to liquefaction in the foundation.But from the result of the soft ware when there is liquefaction zone it indicates with yellow colour.Hence in this dam there is no zone of liquefaction.

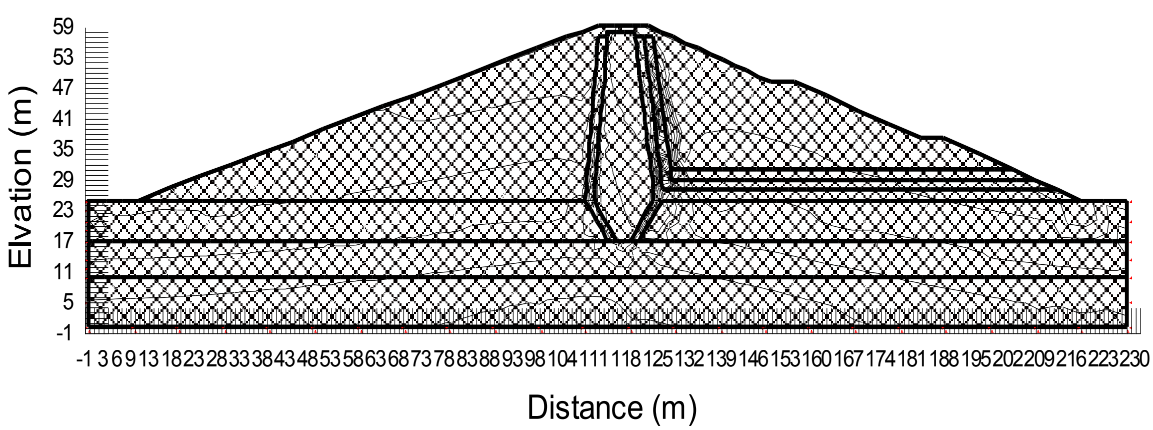

Figure 8-19. Liquefaction for alternative-1.

8.6 Analyses Results of Alternative-2 Embankment Sections

The computed factors of safety against slope failures under the different loading conditions for the selected earth fill- rock fill dam type are summarized in Table 8.5.

Table 8.5 : Computed factors of safety.

Loading Condition	**$FoS_{min.}$**	**Computed Factors of Safety**	
		downstream	Upstream
During construction	1.3	1.383	1.905
End of construction	1.3	1.511	2.074
Steady state seepage	1.5	1.964	-
Steady state seepage with earthquake	1.1	1.827	1.964
Rapid drawdown	1.3	–	1.309

The downstream slope stability analysis of an earth fill-rock fill dam under steady state seepage without earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

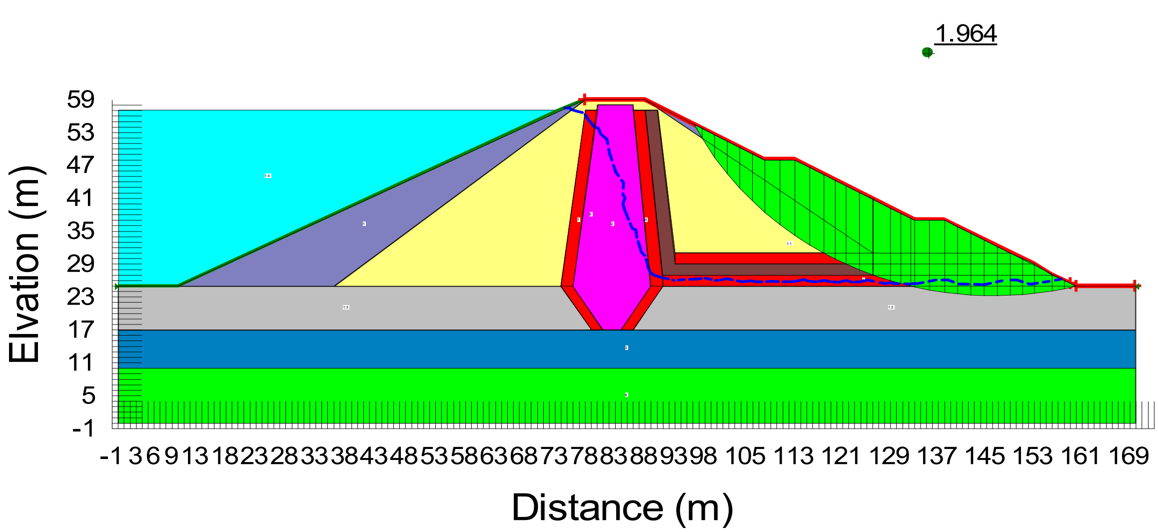

Figure 8-20. Steady state seepage without earthquake (downstream slope).

The downstream slope stability analysis of an earth fill –rock fill dam under steady state seepage with earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

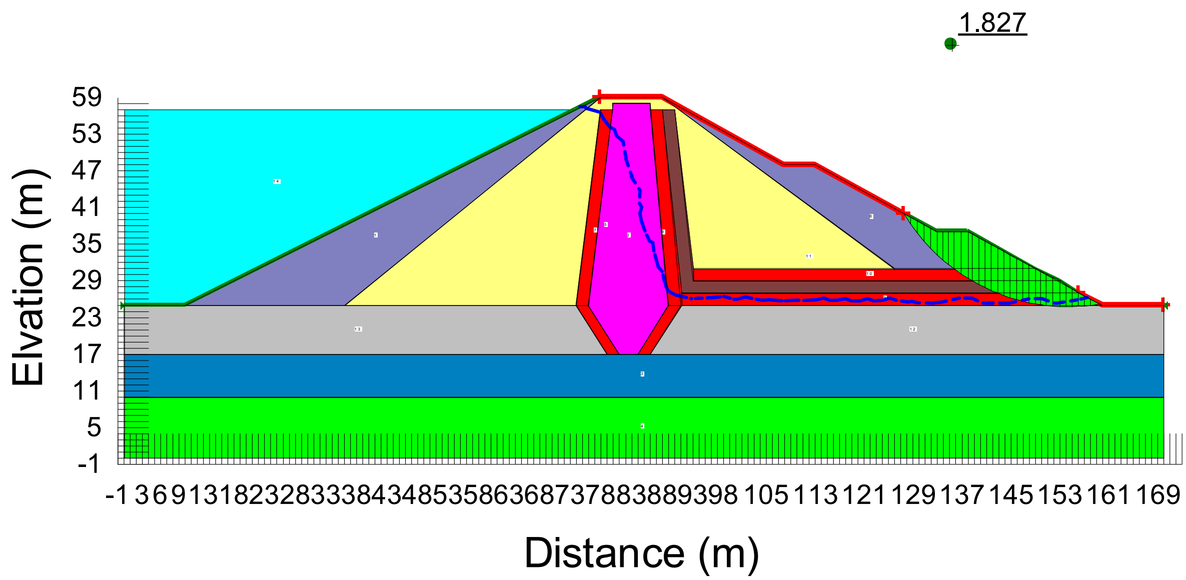

Figure 8-21. Steady state seepage with earthquake (downstream slope).

The downstream slope stability analysis of an earth fill-rock fill dam during construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

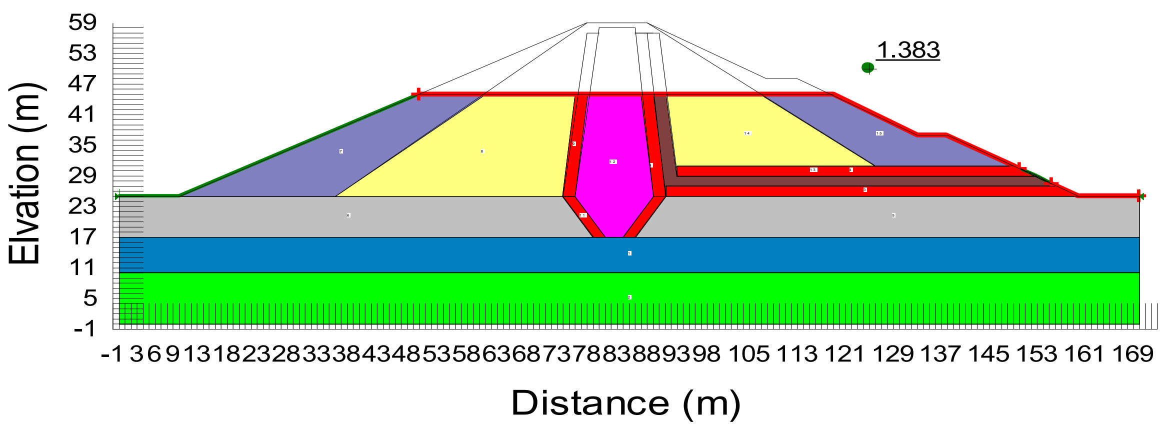

Figure 6 During construction condition (downstream slope).

The downstream slope stability analysis of an earth fill-rock fill dam at the end of construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

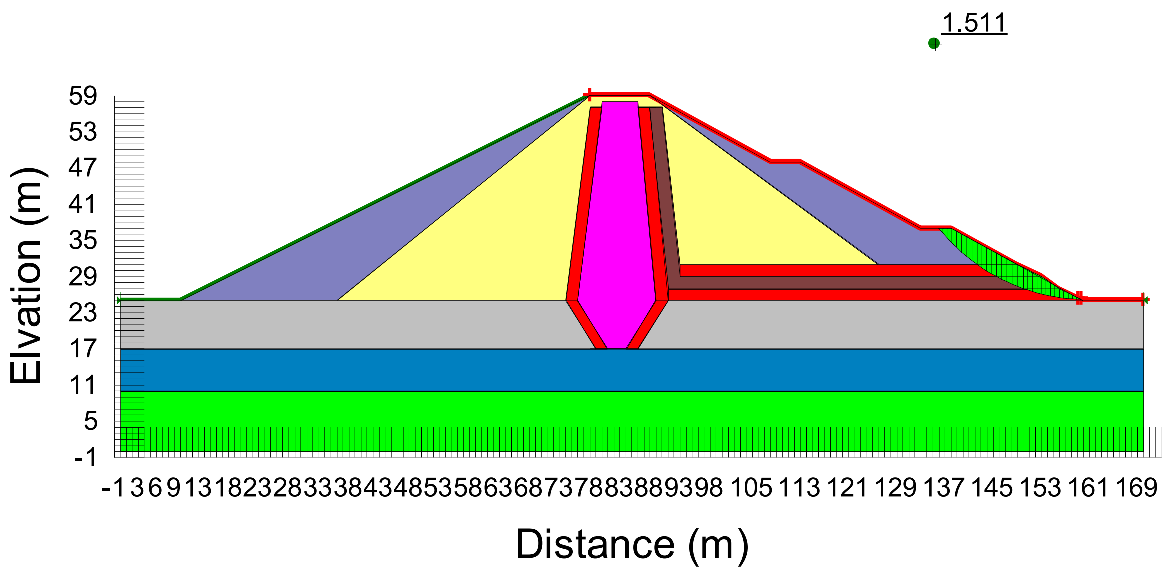

Figure 8-23. End of construction condition (downstream slope)

The upstream slope stability analysis of an earth fill-rock fill dam under steady state seepage without earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

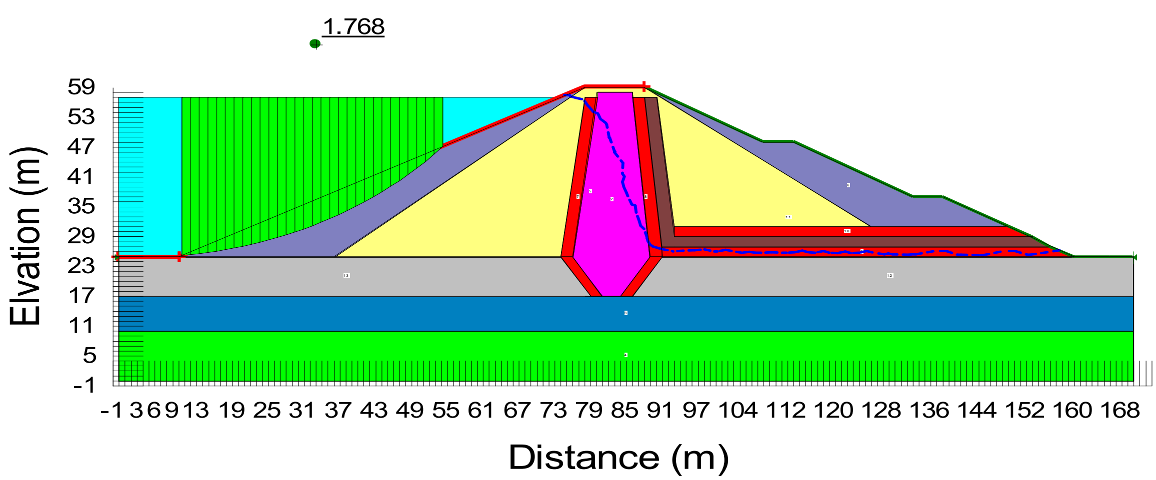

Figure 7. Steady state seepage without earthquake (upstream slope).

The upstream slope stability analysis of an earth fill-rock fill dam under steady state seepage with earth quake is shown below in figure and the green colour indicate the critical slip surface and factor of safety.

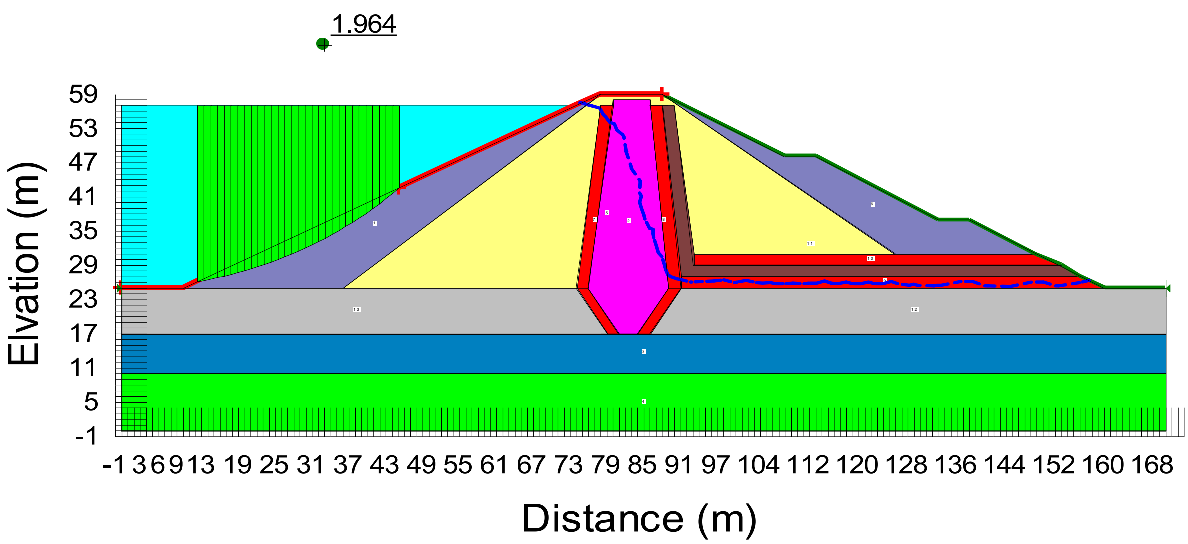

Figure 8-25. Steady state seepage with earthquake (upstream slope).

The upstream slope stability analysis of an earth fill-rock fill dam during sudden draw dawn is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

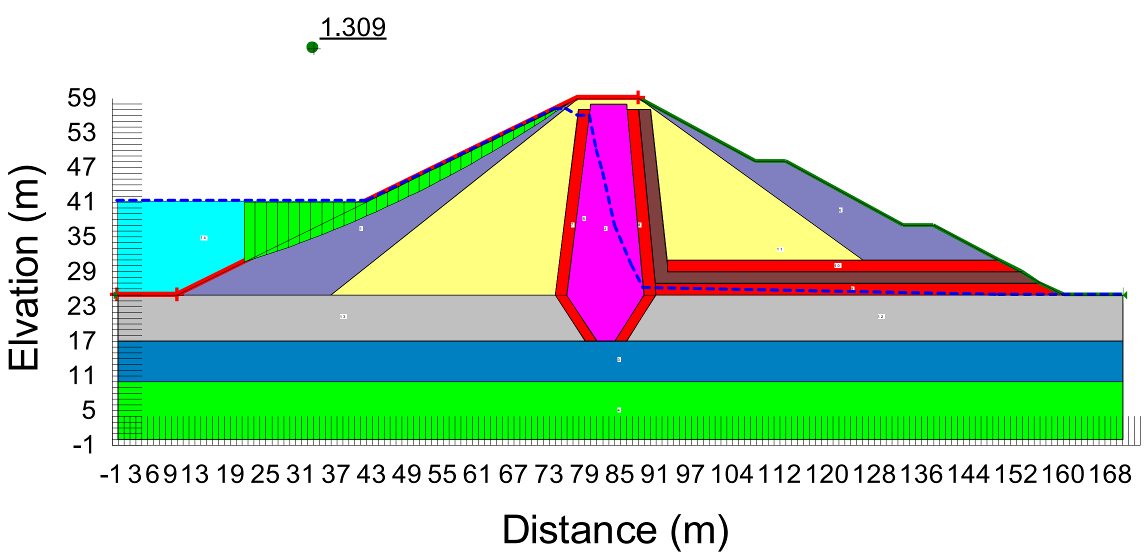

Figure 8-26. Sudden drawdown condition (upstream slope).

The upstream slope stability analysis of an earth fill-rock fill dam during construction is shown below in figure and the green colour indicate the critical slip surface and factor of safety

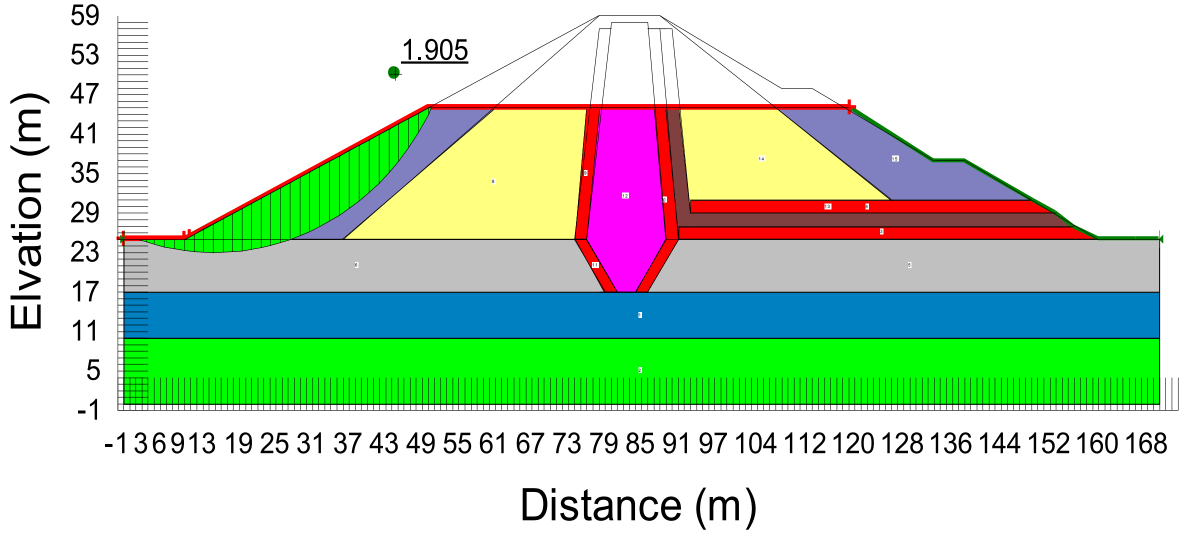

Figure 8-27 During construction condition (upstream slope)

The upstream slope stability analysis of an earth fill-rock fill dam at the end of construction is shown below in figure and the green colour indicates the critical slip surface and factor of safety.

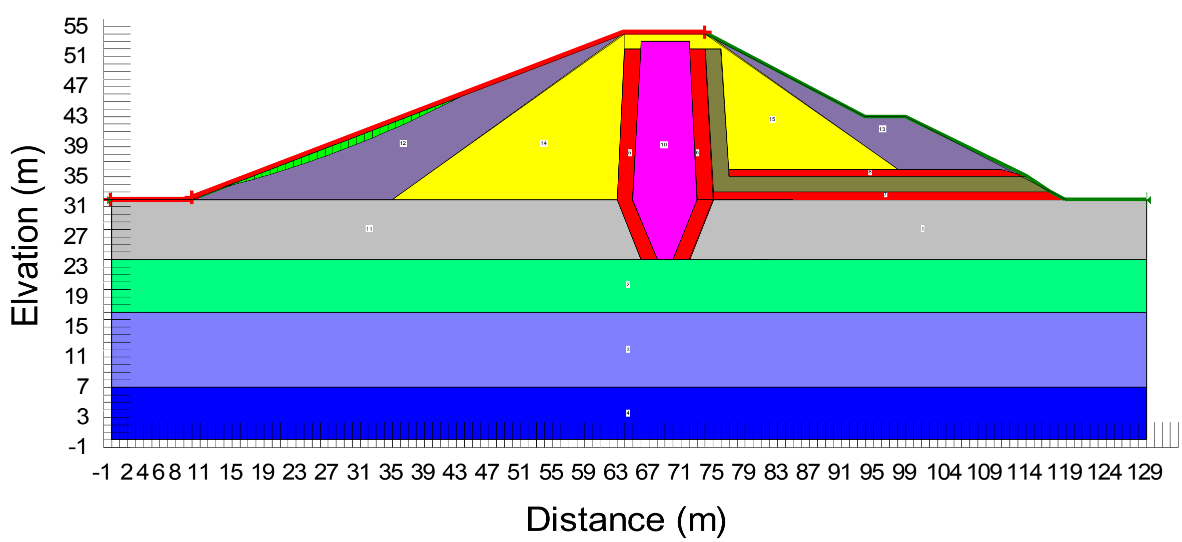

Figure 8-28. End of construction condition (upstream slope)

The vertical displacement or settlement analysis of an earth fill-rock fill dam is shown below in figure 8-29 which shows the settlement of the dam from its initial condition and figure 8-30 shows the vertical total displacement versus the vertical coordinate and the settlement result is important to check free board provided during dimensioning of the dam.

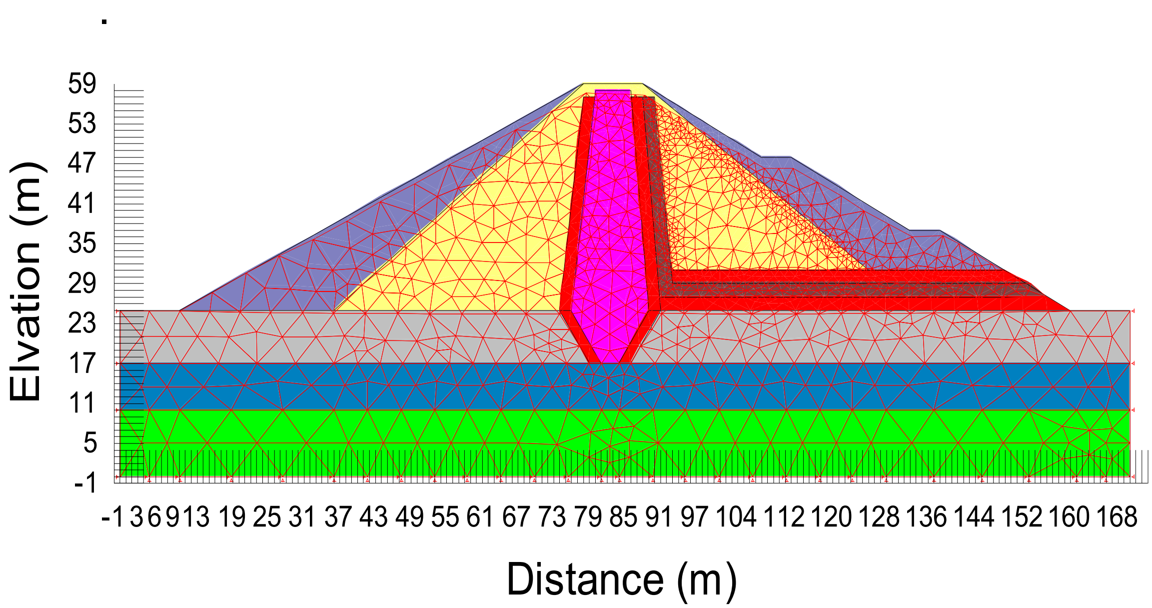

Figure 8-29. Vertical settlement.

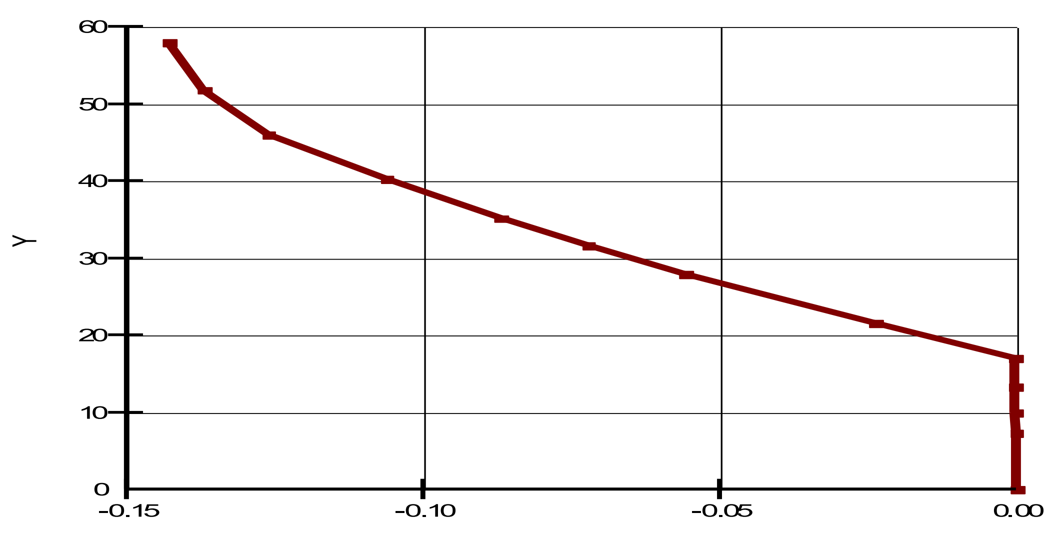

Figure 8 Vertical total displacement.

The stress analysis of an earth fill dam is shown below in figure 8-31 and figure 8-32 and the contour line indicates the total stress at different depth of the dam and foundation.

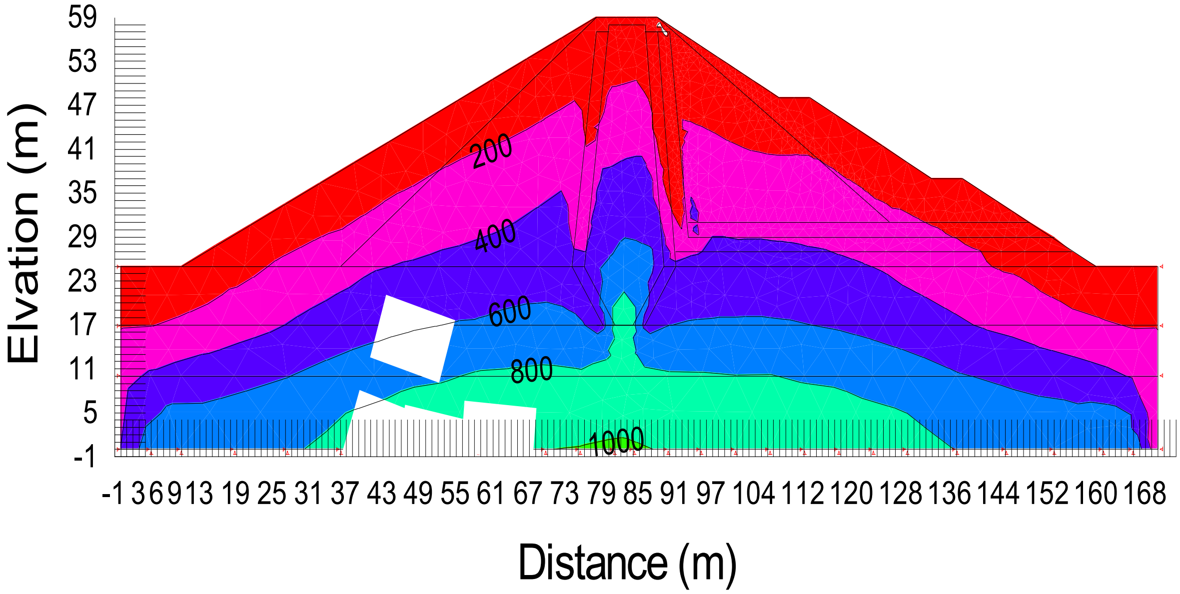

Figure 8-31. Total stress.

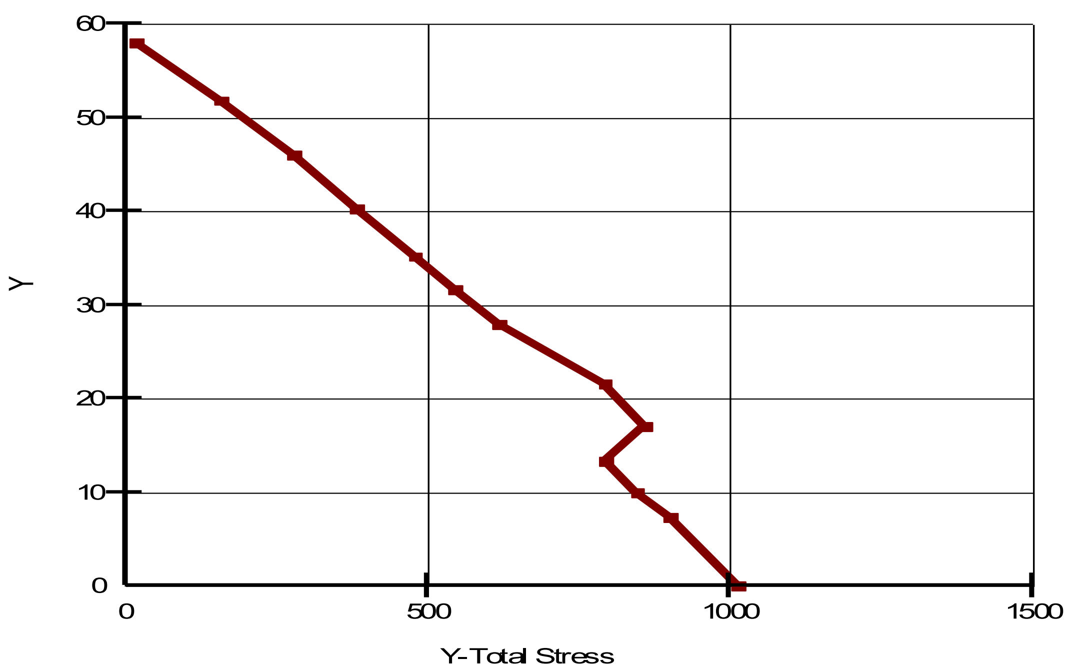

Figure 8-32. Total stress.

The figure shown below indicate the area susuptable to liquefaction in the foundation for the second altenative.But from the result of the soft ware when there is liquefaction zone it indicates with yellow colour.Hence in this dam also there is no zone of liquefaction.

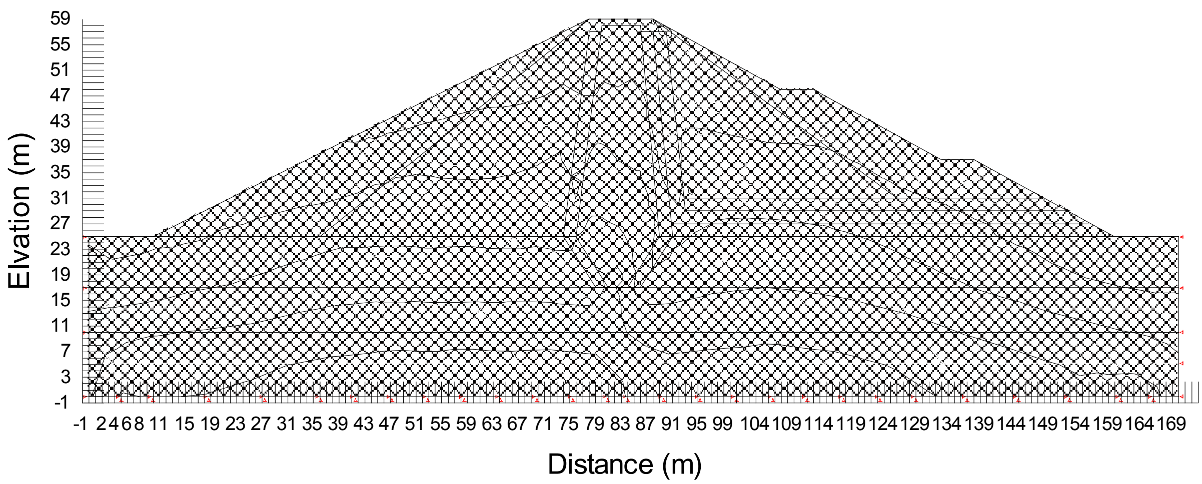

Figure 8-33. Liquefaction for alternative-2.

8.7 Alternative 3

The third alternative is construction of concrete weir in the entire section of the river with special treatment of the foundation. The adopted structure is an Ogee Shaped over flow weir structure**.** The over flow length of the ogee shaped weir section was 322 m and the abutment to abutment length was 386m. There were two silt excluders under sluices, one on each flank of the river of total opening size of 26 m (8 ways of 3.25m each). Especially the island and left side of the foundation is constructed using foundation treatment and pile foundation in order to get the hard rock formation.

From the Hydraulic design result the weir has two components; weir proper of total length 349m and two under sluices one on each flank of the river of total width 51.5m including one fish ladder on right bank. The both side proposed under sluices will facilitate to pass minimum required environmental release and also to remove the silt from the river bed so that almost silt free water may enter into the both main canals. It is also proposed to provide gated silt excluders in each alternate block of weir at invert level of 469.573 in bell mouth shaped (opening size 1.2m X 1.2m) of circular size 800mm diameter. The over flowing type ogee shaped weir section has been designed to pass 85% of Q_{100} and both side under sluices has to pass 15% of the Q_{100}.The crest level of the under sluices has been kept at the average river bed level i.e., at an elevation of 461.423m whereas, the crest level of weir proper is kept at elevation of 477.723m i.e., pond level. Over the Ogee shaped weir, a foot bridge has been proposed to cross the Omo River from either bank side, resting on 30 number piers of 31 bays on the weir crest. Two divide walls have been proposed on either side of the weir proper to divide the flow from weir proper to under sluice. A fish ladder of opening width of 1.5m wide has been proposed all along the right bank abutment. The fish ladder and right side under sluice has been partitioned by a pier of width 2.0m. Due to the limitation for the fixation of pond level at elevation 477.723m and the available river flow width for Q_{100} at weir site, the afflux has been estimated as 4.64m (for design flood Q_{100}) above the crest level of the proposed weir. The pond level has been fixed by providing 1m working head to the FSL of the main canals. The result of detail hydraulic design is shown below in the figure.

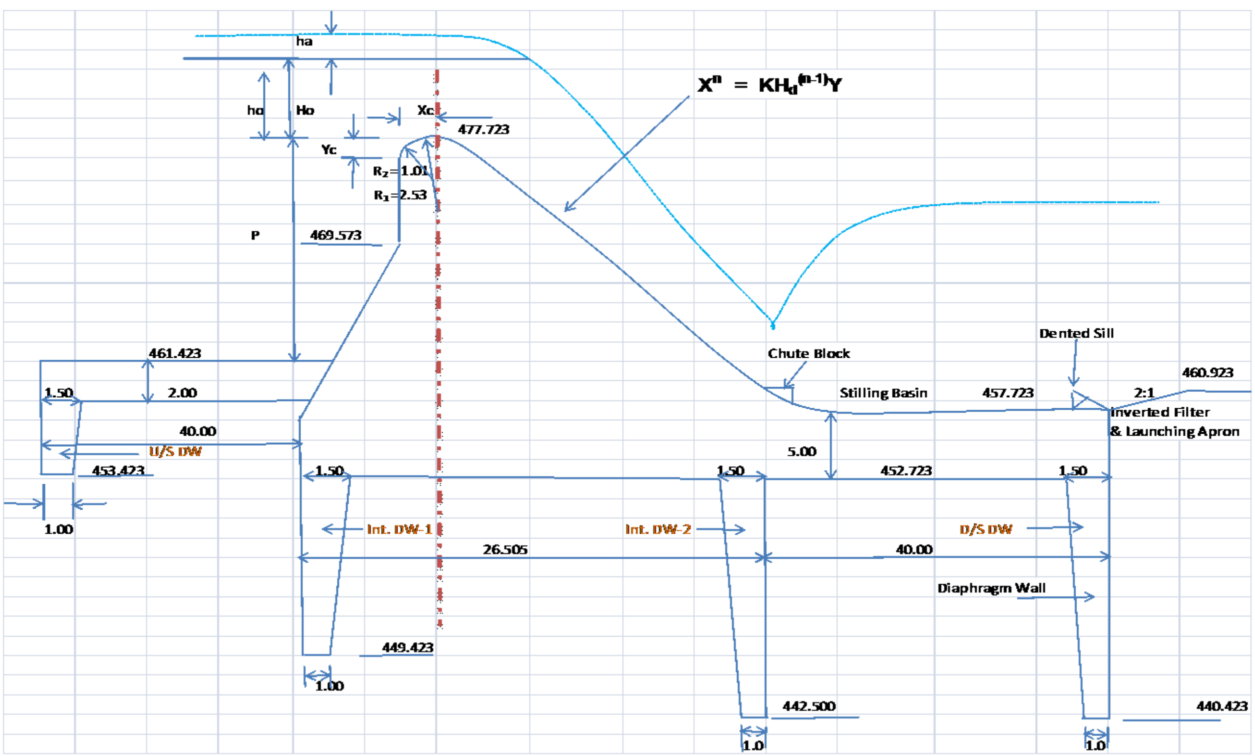

Figure 8-34. Concrete weir section

8.7.1 Stress Analysis of concrete weir

The stress analysis of the third alternative an over flow weir section on sandy silt foundation is done using QUAKE/W software under static seepage condition without earth quake shown below in figure 8-35 and figure 8-36.

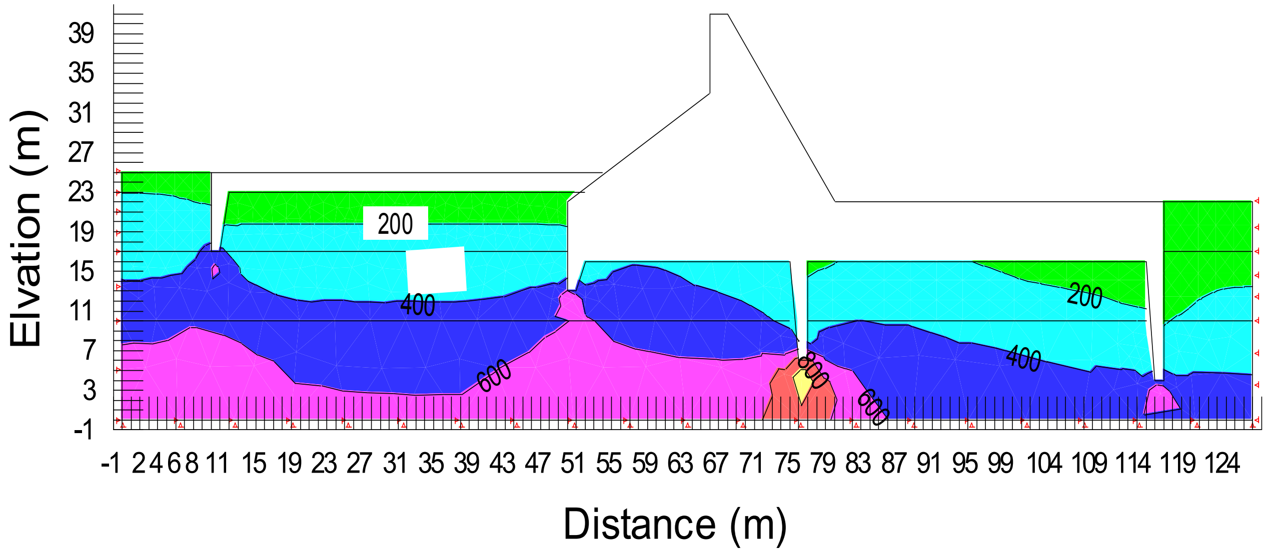

Figure 8-35. Total Stress analysis for alternative-3 without earth quake.

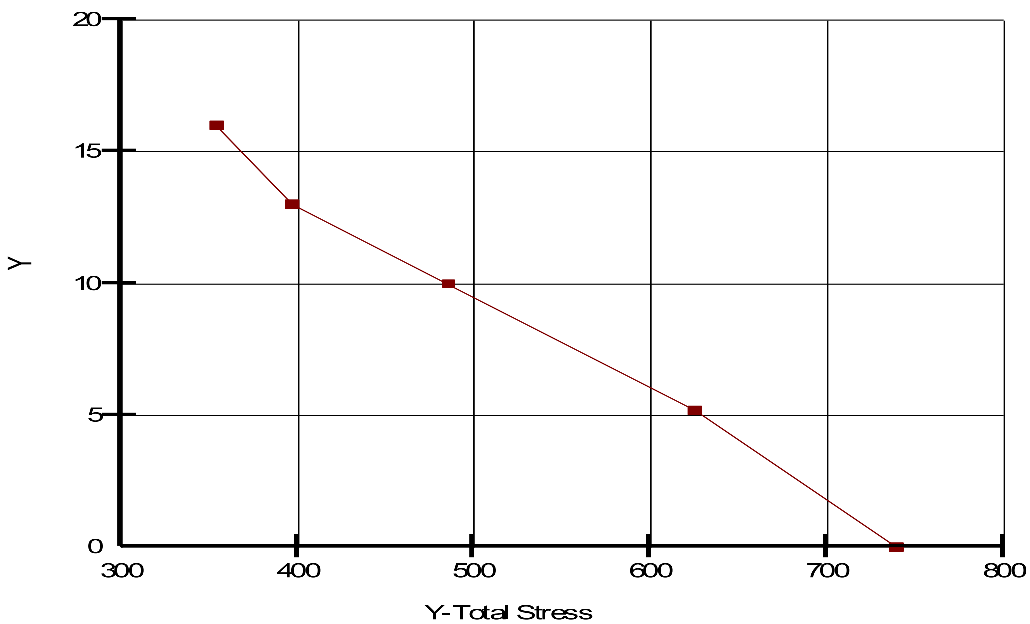

Figure 8-36. Total Stress graph for alternative-3 without earth quake.

The stress analysis of the third alternative an over flow weir section on sandy silt foundation is done using QUAKE/W software under static seepage condition with earth quake shown below in figure 8-37 and figure 8-38 .

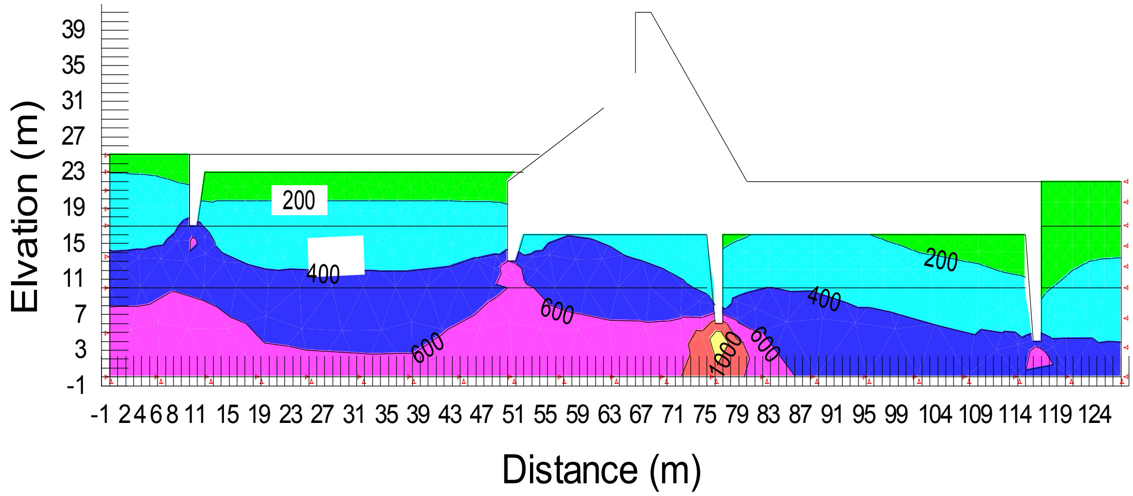

Figure 8-37. Total Stress analysis for alternative-3 with earth quake.

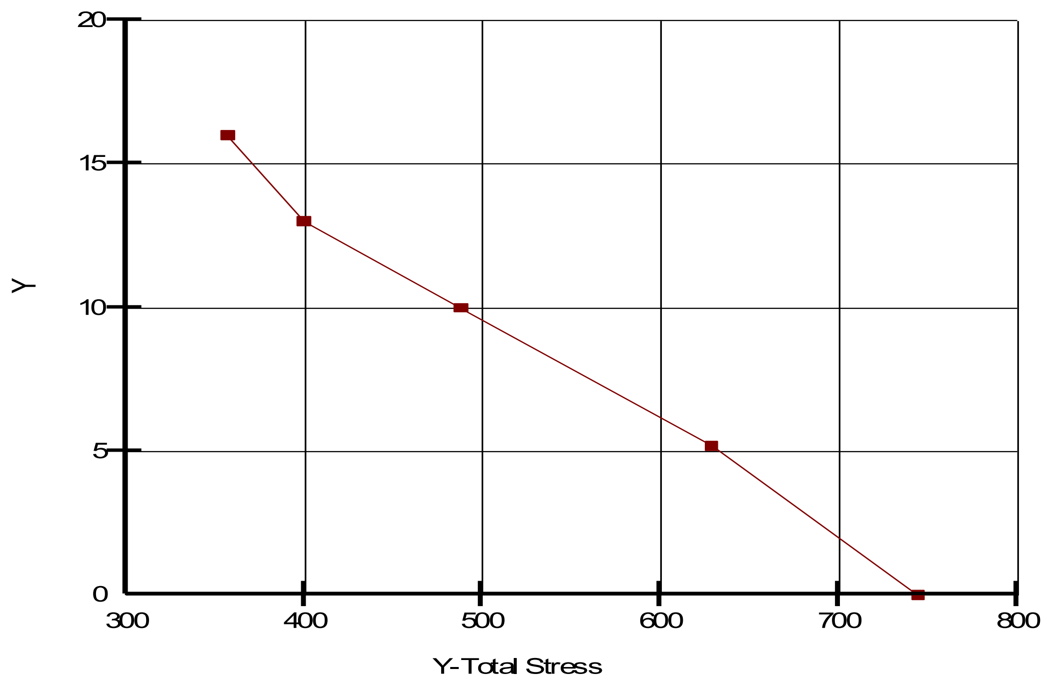

Figure 8-38. Total Stress graph for alternative-3 with earth quake.

9 SELECTION OF BEST ALTERNATIVE

The selection of best headwork alternative from the proposed alternatives through comparing based on the suitability of the structure for existing foundation conditions of the site, economic, functional and practically point of view.

9.1 Economic Analysis of the Proposed Alternatives

The results of the quantifying and the pricing, conforming to the different work items of the implementation of the project, are presented in Table 9.1 below.

Table 9.1 : Estimated Project Costs.

S.No	Alternative Types		Total Cost (ETB)
	On Island and Left side	Right side	
1	Earth fill Dam	Concrete Weir	1,410,751,856
2	Earth fill-Rock fill Dam	Concrete Weir	1,864,042,387
3	Concrete Weir	Concrete Weir	2,389,566,272

9.2 Selection of Functional and Practically Plausible Alternative

The selection of functionally and practically plausible alternative is done based on the scientific and existing construction technological experience of the world under such challenging foundation conditions of the site.

In the selection of diversion site there will face challenging foundation conditions. These will include differential settlement, islands and discontinuities such as bedding planes, joints, shear zones and faults. Geological and geotechnical investigation of a dam site selected for detailed evaluation is directed to determination of geological structure, stratigraphy, faulting, foliation and jointing, and to establishing ground and groundwater conditions adjacent to the dam site, including the abutments.

It is also important to recognize that many major dams are now necessarily built on less favorable and more difficult sites. For obvious reasons, the most attractive sites have generally been among the first to be exploited.

A proportion of sites developed today would, in the past, have been rejected as uneconomic or even as quite unsuitable for a dam. The ability to build successfully on less desirable foundations is a reflection of advances in geo-technical understanding and of confidence in modern ground-improvement processes.

Embankment dams are numerically dominant for technical and economic reasons, and account for an estimated 85–90% of all dams built [1]. As the embankment dam evolved it has proved to be increasingly adaptable to a wide range of site circumstances including adaptability to a broad range of foundation conditions, ranging from competent rock to soft and compressible or relatively pervious soil formations and properly designed, the embankment can safely accommodate an appreciable degree of settlement–deformation without risk of serious cracking and possible failure. Earth fill embankments remain dominant, but rock fill is to some extent displacing earth fill for larger structures as it offers several advantages. In contrast, concrete dams and their masonry predecessors are more demanding in relation to foundation conditions, requiring sound and stable rock. Historically, they have also proved to be dependent upon relatively advanced and expensive construction skills and plant. The relative disadvantages of the embankment dam are few. The most important include an inherently greater susceptibility to damage or destruction by overtopping, with a consequent need to ensure adequate flood relief and a separate spillway, and vulnerability to concealed leakage and internal erosion in dam or foundation. Hence for the kuraz weir site on the left part of the river section an earth fill dam and for the right side or rocky foundation concrete weir is most feasible Because of the functional and practical plausibility.

10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

For the problems encountered for this research the selection of best alternative structures is done after analyzing best economic, practical and functional plausible alternative. The economic analysis of the different items composing all the alternative structures were calculated from the three design alternative dimensions, feasibility study plans, draft hydraulic design report result and the material availability with accessible for haulage and nearby site to reduce haulage distance indicate that the first alternative which is concrete weir on the right side and earth fill dam on the left side of the weir is best in economic.

The selection of functionally and practically plausible alternative is done based on the scientific and existing construction technological experience of the world under such challenging foundation conditions of the site. Since the embankment dam evolved it has proved to be increasingly adaptable to a wide range of site circumstances including adaptability to a broad range of foundation conditions, ranging from competent rock to soft and compressible or relatively pervious soil formations and properly designed, the embankment can safely accommodate an appreciable degree of settlement deformation without risk of serious cracking and possible failure. Earth fill embankments remain dominant, but rock fill is to some extent displacing earth fill for larger structures as it offers several advantages. In contrast, concrete dams and their masonry predecessors are more demanding in relation to foundation conditions, requiring sound and stable rock. Hence for the Kuraz weir site on the left part of the river section an earth fill dam and for the right side or rocky foundation concrete weir is most feasible Because of the economical, functional and practical plausibility.

The scientific contribution of this research is as follows

- How different headwork alternatives are selected, designed and analyzed for the challenging foundation conditions at the sites.

- It describes how the spillway type is affected by the structure that we adopt for the specific site and foundation conditions. For the case of kuraz when we adopt concrete weir across the whole cross section we can adopt free overflow spillway. But for the case of composite dam of concrete and embankment section the spillway is controlled or gated spillway.
- It also indicate the possible options and measures that we use scientifically when problems are faced in the foundations of selected site during construction face of a project.

10.2 Recommendations

- During the prefeasibility study of any project the site should be selected after detail investigation of all the necessary geotechnical information of the foundation.
- This thesis investigates only the hydraulic design and analysis of diversion structures on the challenging foundation conditions. Hence further study of the structural design of the diversion structures on this challenging foundation structures may be a topic for further study.
- For the existing condition of the project instead of ignoring the existing coffer Dam it is better to rehabilitate with a number of provisions like an impervious concrete cutoff along the axis intercepting the river alluvium and partly extending into the abatement, a draining layer and additional rock fill buttress and the coffer Dam will therefore be upgraded to become an earth-and-rock fill Dam.
- During the construction of the main structure it should be carefully selected the height of the coffer Dam because sometimes when we select the height of the coffer dam the coffer dam height is larger than the main diversion structure. But after construction of the dam it will be difficult to remove the coffer dam, especially for the weir type diversion structures.

11 APPENDICES

11.1 Appendix to Chapter Five

11.1.1 Weir Downstream Ogee profile

x	y=-0.1234x^1.865
0	0.000
0.5	-0.034
1	-0.123
1.5	-0.263
2	-0.450
2.5	-0.682
3	-0.958
3.5	-1.276
4	-1.637
4.5	-2.040
5	-2.483
5.5	-2.965
6	-3.488
6.5	-4.049
7	-4.650
7.5	-5.288
8	-5.965
8.3	-6.388

11.1.2 Weir floor pressure

For pile-1				
	b=	136.810		
	d=	6.000		
	α=	22.802		
	λ=	11.912		
	ϕE=	0.187	18.723	
	ϕD=	0.131	13.143	
	ϕC1=	81.277		
	ϕD1=	86.857		
correction for ϕC1				
	a	for pile 2 interfrance		
		D=	6.000	
		d=	5.000	
		b=	136.810	
		b'=	55.500	
		C1	0.502	positive
	b	**for the slope:** corection is nil		
	c	**for the floor thickness**		
		ϕD1=	86.857	
		ϕC1=	81.277	
		t=ASSUMED	1.000	
		dis.b/n CD	6.000	
		C2	0.930	positive
	hence, corrected ϕC1 =		82.709	
		ϕE1 =	100.000	

	for pile-2					
	b1=	56.500				
	b2=	79.310				
	d=	7.000				
	α1=	8.071				
	α2=	11.330				
	λ=	9.754				
	λ1=	-1.620				
	ϕE2=	0.587	58.688			
	ϕD2=	0.553	55.341			
	ϕC2=	0.521	52.053			
	correction for ϕE2					
	a	for pile 1 interfrance				
		D=	5.000			
		d=	6.000			
		b=	136.810			
		b'=	55.500			
		C1	0.459	negative		
	b	for the slope: corection is nil				
	c	for the floor thickness				
		ϕE2=	58.688			
		ϕD2=	55.341			
		t=ASSUMED	1.000			
	dis.b/n E2D2		7.000			
		C2	0.478	negative		
	hence, corrected ϕE2 =		57.751			
	correction for ϕC2					
	a	for pile 3 interfrance				
		D=	6.803			
		d=	6.000			
		b=	136.810			
		b'=	78.310			
		C1	0.524	POSITIVE		
	b	for the slope	nill			
		from table for 0.7H:1V,slope correc.factor= 13.127				
		from Garg book				
		bs=	10.000			
		b1=	78.310			
		c2=	0.000	negative		

c	for the floor thickness,same correction as E2D2 but oposite sign					
	ϕE2=	58.688				
	ϕD2=	55.341				
	t=assumed	1.000				
	dis.b/n E2D2	7.000				
	C2	0.478	POSITIVE			
hence corrected ϕC2 =		53.055				

for pile-3			
b=	136.810		
d=	12.000		
α=	11.401		
λ=	6.222		
ϕE3=	0.263	26.273	
ϕD3=	0.183	18.307	
ϕC3=	0.000		
correction for ϕE3			
a	for pile 2 interfrance		
	D=	8.000	
	d=	10.500	
	b=	136.810	
	b'=	94.500	
	C1	0.748	negative
b	for the slope: corection is nil		
c	for the floor thickness		
	ϕD3=	18.307	
	ϕE3=	26.273	
	t=assumed	1.500	
	dis.b/n E3D3	12.000	
	C2	0.996	negative
hence ,corrected ϕE3 =		24.530	

11.1.3 Corrected pressure

pile -1	**%**	**pile-2**	**%**	**pile-3**	**%**
ϕE1=	100.000	ϕE2=	57.751	ϕE3=	24.530
ϕD1=	86.857	ϕD2=	55.341	ϕD3=	18.307
ϕC1=	82.709	ϕC2=	53.055	ϕC3=	0.000

11.1.4 Elevation of Hydraulic grade line HGL

Flow co	**U/S WL**	**d/s WL**	**Head,m**		**ϕE1(%)**	**ϕD1(%)**	**ϕC1(%)**	**ϕE2(%)**	**ϕD2(%)**	**ϕc2(%)**	**ϕE3(%)**	**ϕd3(%)**
					100.000	86.857	82.709	57.751	55.341	53.055	24.530	18.307
pond leve	477.723	462.033	15.690	press.h	15.690	13.628	12.977	9.061	8.683	8.324	3.849	2.872
u/s				loss	0.000	2.062	2.713	6.629	7.007	7.366	11.841	12.818
no TW				HGL	477.723	475.661	475.010	471.094	470.716	470.357	465.882	464.905
high flood	482.434	470.473	11.961	press.h	11.961	10.389	9.893	6.908	6.619	6.346	2.934	2.190
with cond				loss	0.000	1.572	2.068	5.053	5.342	5.615	9.027	9.771
				HGL	482.434	480.862	480.366	477.381	477.092	476.819	473.407	472.663
pond leve	477.723	464.923	12.800	press.h	12.800	11.118	10.587	7.392	7.084	6.791	3.140	2.343
with con				loss	0.000	1.682	2.213	5.408	5.716	6.009	9.660	10.457
				HGL	477.723	476.041	475.510	472.315	472.007	471.714	468.063	467.266

11.1.5 Floor thickness calculation

dist.from d/s end ,m	d/s end	10.000	30.000	45.000
unbalanced pressure,m	2.887	3.307	4.148	4.779
thickness	2.062	2.362	2.963	3.414
design thickness	2.100	2.500	3.000	3.500

11.1.6 Stilling basin trial-1 assume EL = 456.62

Prejump Calculation

Depth of water above crest	EL of water above crest	Ratio of head to design head	coefficient	Discharge	discharge intensity	Spesific energy	depth before jump	calc. spesific energy	velocity before jump	Froud number
H	**(TEL)u/s**	**X**	**Cd**	**Q**	**q**	**(E1)d/s**	**d1**	**Ecal**	**v1**	**Fr1**
0.000	477.723	0.000	0.384	0.000	0.000	21.103	21.103	21.103	0.000	0.000
0.400	478.123	0.079	0.397	44.892	0.445	21.503	0.022	21.503	20.530	44.533
0.800	478.523	0.158	0.409	130.766	1.295	21.903	0.063	21.903	20.700	26.419
1.200	478.923	0.236	0.420	246.660	2.444	22.303	0.117	22.303	20.863	19.464
1.600	479.323	0.315	0.430	388.914	3.853	22.703	0.183	22.703	21.020	15.675
2.000	479.723	0.394	0.439	555.401	5.502	23.103	0.260	23.103	21.170	13.258
2.400	480.123	0.473	0.448	744.623	7.377	23.503	0.346	23.503	21.315	11.568
2.800	480.523	0.552	0.456	955.415	9.465	23.903	0.441	23.903	21.455	10.313
3.200	480.923	0.631	0.464	1186.810	11.758	24.303	0.545	24.303	21.590	9.341
3.600	481.323	0.709	0.471	1437.978	14.246	24.703	0.656	24.703	21.721	8.563
4.000	481.723	0.788	0.478	1708.184	16.923	25.103	0.775	25.103	21.848	7.926
4.400	482.123	0.867	0.484	1996.772	19.782	25.503	0.900	25.503	21.970	7.393
4.800	482.523	0.946	0.490	2303.142	22.817	25.903	1.027	26.177	22.214	6.998
5.074	482.797	1.000	0.494	2522.982	24.995	26.177	1.127	26.177	22.169	6.666

After jump Calculation

depth after jump	fig 9.42 design of small dam	water level after jump	length of jump;LJ		
			USBR chart	Lj=3.5d1F1^1.5 ,(F1<5) Lj=8d1F1 ,(F1> 5)	Lj=Kd1F1^1.5 (K=1.7)
d2	**L/d2**	**(EL)j**	**type II**		
0.000	2.010	458.620	0.000	0.000	0.000
1.354	3.020	459.974	4.088	7.718	10.945
2.307	3.820	460.927	8.813	13.227	14.447
3.166	3.900	461.786	12.347	18.238	17.098
3.973	4.200	462.593	16.686	22.986	19.339
4.745	4.350	463.365	20.641	27.567	21.330
5.492	4.380	464.112	24.053	32.029	23.149
6.218	4.380	464.838	27.233	36.399	24.839
6.927	4.300	465.547	29.785	40.695	26.430
7.621	4.280	466.241	32.620	44.930	27.940
8.303	4.200	466.923	34.874	49.113	29.381
8.974	4.190	467.594	37.600	53.249	30.765
9.665	4.180	468.285	40.398	57.503	32.325
10.080	4.130	468.700	41.630	60.125	32.987

11.1.7 Stilling basin trial-2 assume EL = 457.62

Prejump Calculation

Depth of water above crest	EL of water above crest	Ratio of head to design head	coeff icient	Discharge	discharge intensity	Spesific energy	depth before jump	calc. spesific energy	velocity before jump	Froud number
H	(TEL)u/s	X	Cd	Q	q	(E1)d/s	d1	Ecal	v1	Fr1
0.000	477.723	0.000	0.384	0.000	0.000	20.103	20.103	20.103	0.000	0.000
0.400	478.123	0.079	0.397	44.892	0.445	20.503	0.022	20.503	20.046	42.968
0.800	478.523	0.158	0.409	130.766	1.295	20.903	0.064	20.903	20.220	25.505
1.200	478.923	0.236	0.420	246.660	2.444	21.303	0.120	21.303	20.387	18.800
1.600	479.323	0.315	0.430	388.914	3.853	21.703	0.188	21.703	20.546	15.148
2.000	479.723	0.394	0.439	555.401	5.502	22.103	0.266	22.103	20.699	12.818
2.400	480.123	0.473	0.448	744.623	7.377	22.503	0.354	22.503	20.846	11.188
2.800	480.523	0.552	0.456	955.415	9.465	22.903	0.451	22.903	20.988	9.979
3.200	480.923	0.631	0.464	1186.810	11.758	23.303	0.557	23.303	21.125	9.041
3.600	481.323	0.709	0.471	1437.978	14.246	23.703	0.670	23.703	21.258	8.291
4.000	481.723	0.788	0.478	1708.184	16.923	24.103	0.791	24.103	21.386	7.676
4.400	482.123	0.867	0.484	1996.772	19.782	24.503	0.920	24.503	21.511	7.162
4.800	482.523	0.946	0.490	2303.142	22.817	24.903	1.055	24.903	21.631	6.724
5.074	482.797	1.000	0.494	2522.982	24.995	25.177	1.151	25.177	21.711	6.461

After jump Calculation

depth after jump	fig 9.42 design of small dam	water level after jump	length of jump;LJ		
			USBR chart	Lj=3.5d1F1^1.5 ,(F1<5) Lj=8d1F1 ,(F1> 5)	Lj=Kd1F1^1.5 (K=1.7)
d2	L/d2	(EL)j	type II		
0.000	2.010	458.620	0.000	0.000	0.000
1.337	3.100	459.957	4.145	7.626	10.623
2.279	3.900	460.899	8.889	13.073	14.029
3.128	4.000	461.748	12.510	18.028	16.611
3.925	4.220	462.545	16.562	22.725	18.795
4.688	4.380	463.308	20.532	27.258	20.738
5.425	4.380	464.045	23.762	31.674	22.514
6.143	4.380	464.763	26.904	36.000	24.166
6.843	4.280	465.463	29.289	40.255	25.721
7.530	4.230	466.150	31.850	44.449	27.197
8.203	4.200	466.823	34.454	48.591	28.608
8.866	4.180	467.486	37.058	52.688	29.963
9.518	4.130	468.138	39.307	56.744	31.269
9.959	4.100	468.579	40.830	59.501	32.138

11.1.8 Stilling basin trial-3 assume EL = 458.62

Prejump Calculation										
Depth of water above crest	EL of water above crest	Ratio of head to design head	coeff icient	Discharge	discharge intensity	Spesific energy	depth before jump	calc. spesific energy	velocity before jump	Froud number
H	(TEL)u/s	X	Cd	Q	q	(E1)d/s	d1	Ecal	v1	Fr1
0.000	477.723	0.000	0.384	0.000	0.000	19.103	19.103	19.103	0.000	0.000
0.400	478.123	0.079	0.397	44.892	0.445	19.503	0.023	19.503	19.550	41.384
0.800	478.523	0.158	0.409	130.766	1.295	19.903	0.066	19.903	19.728	24.580
1.200	478.923	0.236	0.420	246.660	2.444	20.303	0.123	20.303	19.898	18.129
1.600	479.323	0.315	0.430	388.914	3.853	20.703	0.192	20.703	20.061	14.615
2.000	479.723	0.394	0.439	555.401	5.502	21.103	0.272	21.103	20.216	12.372
2.400	480.123	0.473	0.448	744.623	7.377	21.503	0.362	21.503	20.366	10.804
2.800	480.523	0.552	0.456	955.415	9.465	21.903	0.461	21.903	20.511	9.640
3.200	480.923	0.631	0.464	1186.810	11.758	22.303	0.569	22.303	20.650	8.737
3.600	481.323	0.709	0.471	1437.978	14.246	22.703	0.685	22.703	20.784	8.015
4.000	481.723	0.788	0.478	1708.184	16.923	23.103	0.809	23.103	20.914	7.423
4.400	482.123	0.867	0.484	1996.772	19.782	23.503	0.940	23.503	21.040	6.928
4.800	482.523	0.946	0.490	2303.142	22.817	23.903	1.078	23.903	21.162	6.507
5.074	482.797	1.000	0.494	2522.982	24.995	24.177	1.177	24.177	21.243	6.253

After jump Calculation					
depth after jump	fig 9.42 design of small dam	water level after jump	length of jump;LJ		
			USBR chart	Lj=3.5d1F1^1.5 ,(F1<5) Lj=8d1F1 ,(F1> 5)	Lj=Kd1F1^1.5 (K=1.7)
d2	L/d2	(EL)j	type II		
0.000	2.010	458.620	0.000	0.000	0.000
1.320	3.200	459.940	4.224	7.532	10.296
2.250	4.100	460.870	9.225	12.913	13.604
3.088	4.020	461.708	12.413	17.811	16.115
3.875	4.300	462.495	16.661	22.455	18.242
4.628	4.350	463.248	20.132	26.939	20.135
5.356	4.310	463.976	23.086	31.307	21.868
6.065	4.290	464.685	26.018	35.588	23.480
6.757	4.240	465.377	28.648	39.799	24.999
7.434	4.200	466.054	31.224	43.951	26.441
8.100	4.190	466.720	33.937	48.052	27.821
8.754	4.170	467.374	36.502	52.109	29.145
9.397	4.100	468.017	38.529	56.126	30.423
9.833	4.080	468.453	40.117	58.856	31.274

11.1.9 Stilling basin trial-4 assume EL = 462.62

Prejump Calculation										
Depth of water above crest	EL of water above crest	Ratio of head to design head	coefficient	Discharge	discharge intensity	Spesific energy	depth before jump	calc. spesific energy	velocity before jump	Froud number
H	(TEL)u/s	X	Cd	Q	q	(E1)d/s	d1	Ecal	v1	Fr1
0.000	477.723	0.000	0.384	0.000	0.000	15.103	15.103	15.103	0.000	0.000
0.400	478.123	0.079	0.397	44.892	0.445	15.503	0.026	15.146	17.224	34.221
0.800	478.523	0.158	0.409	130.766	1.295	15.903	0.074	15.903	17.623	20.753
1.200	478.923	0.236	0.420	246.660	2.444	16.303	0.137	16.303	17.809	15.350
1.600	479.323	0.315	0.430	388.914	3.853	16.703	0.214	16.703	17.986	12.408
2.000	479.723	0.394	0.439	555.401	5.502	17.103	0.303	17.103	18.155	10.529
2.400	480.123	0.473	0.448	744.623	7.377	17.503	0.403	17.503	18.317	9.215
2.800	480.523	0.552	0.456	955.415	9.465	17.903	0.512	17.903	18.472	8.239
3.200	480.923	0.631	0.464	1186.810	11.758	18.303	0.631	18.303	18.620	7.481
3.600	481.323	0.709	0.471	1437.978	14.246	18.703	0.759	18.703	18.763	6.875
4.000	481.723	0.788	0.478	1708.184	16.923	19.103	0.879	19.782	19.258	6.559
4.400	482.123	0.867	0.484	1996.772	19.782	19.503	1.039	19.503	19.033	5.961
4.800	482.523	0.946	0.490	2303.142	22.817	19.903	1.191	19.903	19.161	5.606
5.074	482.797	1.000	0.494	2522.982	24.995	20.177	1.299	20.177	19.246	5.392

After jump Calculation					
depth after jump	fig 9.42 design of small dam	water level after jump	length of jump;LJ		
			USBR chart	Lj=3.5d1F1^1.5 ,(F1<5) Lj=8d1F1 ,(F1> 5)	Lj=Kd1F1^1.5 (K=1.7)
d2	L/d2	(EL)j	type II		
0.000	2.010	458.620	0.000	0.000	0.000
1.237	3.460	459.857	4.279	7.069	8.788
2.121	3.820	460.741	8.102	12.204	11.814
2.911	4.100	461.531	11.934	16.850	14.029
3.653	4.380	462.273	16.001	21.263	15.916
4.364	4.390	462.984	19.157	25.529	17.603
5.051	4.280	463.671	21.619	29.691	19.153
5.720	4.210	464.340	24.080	33.773	20.600
6.373	4.200	464.993	26.765	37.793	21.966
7.012	4.180	465.632	29.311	41.759	23.268
7.724	4.110	466.344	31.744	46.110	25.095
8.257	4.050	466.877	33.441	49.561	25.713
8.864	3.910	467.484	34.659	53.406	26.871
9.275	3.880	467.895	35.987	56.020	27.642

11.1.10 Design of under sluice on Silt foundation

1. Design Input Parameters	Quantity	Units		
Average deepest river bed level at weir site	461.423	m		
Design discharge after Gilgel Gibe III dam will in operation				
Design dischargefor 1 in 100 years flood	7931.000	cumecs		
Design dischargefor 1 in 500 years flood	13610.000	cumecs		
HFL for 1 in 100 years flood before construction of Weir	470.473	m		
HFL for 1 in 500 years flood before construction of Weir	472.473	m		
Full supply level of R/B & L/B Main Canal at Offtake Point	476.723	m		
Lacey's silt factor 'f'	2.000			
Assumed Concentration	0.200	%		
Assumed river bed retrogression	0.500	m		
2. Fixing of Crest Level and Waterways				
For silty part Lacey's wetted perimeter required P = 4.83*(Q)0.5	430.141	m		
River bed width Abut. to Abut. available	397.000	m		
Looseness Factor =	0.923			
Crest Level				
No. of offtaking canal	2.000	No.		
No. of under sluices	2.000	No.		
undersluice crest levels same as U/S floor	461.423	m		
The height of weir portion	16.300	m		
Working head to feed both sides of canals to FSL	1.000	m		
The crest level of the weir portion	477.723	m		
Waterway Proposed				
For silty part Lacey's wetted perimeter required P = 4.83*(Q)0.5	430.141	m		
eight under sluice bays of 3.5 m width each,total 8*3.5	28.000	m		
The clear water way width adopted for the weir bay is	105.000	m		
The embankment width adopted is	238.000	m		
Width of bridge piers= 10*0.90	9.000	m		
Fish ladder with 1.5 m width and a partition wall of 2 m width.	3.500	m		
thickness of two divide walls =2*2.5	5.000	m		
six undersluice peirs of total width = 6*2	12.000	m		
The total waterway provided between Abut. To Abut.	397.000	m		
Looseness Factor =	0.923			
Average dischareg intensity ,q = Q/ Total overall water way	19.977	m3/s/m		
Scour Depth ,R =0.473*(Q2/f)^1/3	7.487	m		
Velocity of approach v = q/R =	2.668	m/s		
Velocity head = v2/2g =	0.363	m		
Head over the crest of weir for Q1oo flood	4.711	m		

HFL with afflux at weir site	482.434	m
Head over the crest of weir for Q5oo flood	6.966	m
HFL with afflux at weir site	484.689	m
U/S TEL= (Pond level with Afflux) + Velocity head	482.797	m
D/S TEL=D/S water level (HFL)+Velocity head =	470.836	m
Head over the undersluice way crest = H1	21.374	m
Head over the ogee shaped weir crest = H2	5.074	m
Design of Under sluice portion		
1. For maximum design flood		
(a) Without concentration and retrogression of river bed		
During maximun design flood , the u/s water level with afflux	482.434	m
The HFL d/s during high flood=	470.473	m
Therefore, head causing flow ,h =	11.961	m
Let the gate opening be 'x' m ; the discharge can then be calculated with the help of		
submerged orifice formula as given below :-		
Q = Cd*A*(2*g*h)0.5 ,where Cd = 0.62 and A = 8*3.5*X		
Therefore, to pass 15% of Q100	1189.650	cumecs
0.15*Q100 = 0.62*8*3.5*X(2*9.81*h)^0.5		
0.15*7931 = 0.62*8*3.5*X (2*9.81*11.961)^0.5		
X =	4.500	m
The velocity of flow through the opening =Q15%/A	9.442	m/s
Loss of head at entry =0.5*v2/2g	2.272	m
Velocity head calculated above =	0.363	m
T.E.L at just U/S of gate =	482.797	m
T.E.L at just D/S of gate =	480.525	m
U/S T.E.L =	482.797	m
D/S T.E.L without retrogression ==	470.836	m
Head Loss =	9.689	m
Discharge intensity 'q' = Q/width of opening =	42.488	cumecs/m
(b) With 20% concentration and 0.5 m retrogression of river bed		
Discharge intensity, q = 1.2*q	50.985	cumecs/m
Head causing the discharge intensity h =	17.020	m
50.985 = Cd*X*(2gh)^0.5		
Therefore, U/S water level = HFL d/s during high flood + h	487.493	m
The D/S water level with retrogression of 0.5m =	469.973	m
Velocity of flow through the opening = Q15%/A	11.330	m/s
Loss of head at entry =0.5*v2/2g	3.271	m
T.E.L at just U/S of gate =	487.856	m
T.E.L at just D/S of gate =	484.585	m
U/S T.E.L =	487.856	m
D/S TEL with retrogression =	470.336	m
Therefore, the head loss HL =	14.249	m

2. Pond level flow condition		
(a) For no concentration and no retrogression		
pond level	477.723	m
From the rating curve at weir axis,the Q value at 477.723m is projected as	9257.000	cumecs
Average discharge intensity at the pond level to pass 15% of Q100 (q)=	49.591	cumecs/m
Scour depth R =0.473*(Q/f)1/3 (Assume silt factor f= 2)	4.188	m
Velocity of approach=q/R	11.840	m/s
Veocity head=v2/2g	7.145	m
U/S TEL=	484.868	m
From the rating curve,the D/S water level for 15 % of design discharge the level is	464.923	m
D/S TEL	472.068	m
Let the gate opening be 'x' m ; the discharge can then be calculated with the help of		
submerged orifice formula as given below :-		
Q = Cd*A*(2*g*h)0.5 ,where Cd = 0.62 and A = 8*3.5*X		cumecs
Therefore, to pass 15% of Q100		
0.15*Q100 = 0.62*8*3.5*X(2*9.81*h)^0.5		
0.15*9257 = 0.62*8*3.5*X (2*9.81*11.961)^0.5		
X =	5.050	m
velocity through opening = q/x	9.820	m/s
Loss of head at entry = 0.5*v2/2g	2.458	m
T.E.L at just D/S of gate =	482.411	m
Head loss =	10.342	m
Discharge intensity through under sluice =	49.591	cumecs/m
(b) Pond Level Flow With 20% concentration & 0.5 m river bed retrogression		
Discharge intensity with 20% concentration	59.509	cumecs/m
Scour depth R =0.473*(Q/f)1/3 (Assume silt factor f= 2)	4.451	m
Velocity of approach=q/R	13.371	m/s
Veocity head=v2/2g	9.112	m
From the rating curve,the D/S water level for 15 % of design discharge the level is	464.923	m
D/S TEL	474.035	m
Head causing the discharge intensity h =	18.400	m
Therefore, U/S water level =	483.323	m
The D/S water level with retrogression=	464.423	m
D/S TEL with retrogression =	473.535	m
Velocity of flow through the opening =(0.15*9257*1.2)/(28*4.5)=	13.224	m/s
Loss of head at entry = 0.5*v2/2g	4.457	m
T.E.L at just U/S of gate =	483.686	m
T.E.L at just D/S of gate =	479.229	m
Therefore, the head loss HL=	5.694	m

11.1.11 Length of concrete floor required and downstream cistern level

S.No	Item	high flood flow without conc. and retrogn.	high flood flow with conc. and retrogn.	pond level flow without conc. and retrogn.	pond level flow with conc. and retrogn.
1	Discharge intensity,q in cumecs/m	42.488	50.985	49.591	59.509
2	Upsteam water level	482.434	482.434	477.723	477.723
3	Downstream water level	470.473	469.973	464.923	464.423
4	U/s TEL	480.525	484.585	482.411	479.229
5	D/s TEL	470.836	470.336	472.068	473.535
6	Head loss HL	9.689	14.249	10.342	5.694
7	(8*q2*HL)/g = (-1.5*y1+((y12/4) +((2*q2)/(g*y1)))0.5)3*(0.5*y1+((y12/4)+((2*q2)/(g*y1)))0.5)				
	(8*q2*HL)/g	14263.683	30204.988	20742.117	16444.984
	let the value of y1	2.128	2.212	2.386	3.290
	(-1.5*y1+((y12/4) +((2*q2)/(g*y1)))0.5)	14265.944	30203.671	20753.833	16400.798
	(0.5*y1+((y12/4)+((2*q2)/(g*y1)))0.5)				
	After ittiration we get the value of y1	2.128	2.212	2.386	3.290
	y2 =(-0.5*y1+((y12/4)+((2*q2)/(g*y1)))	12.130	14.414	13.352	13.260
	Ef2=y2+(q2/2gy22)	12.755	15.051	14.055	14.286
	Ef1 = H L + Ef2	22.444	29.300	24.398	19.981
8	Level at which jump will form.i.e (d/s T	458.081	455.285	458.013	459.248
9	length of concrete floor required = 5*(y	50.009	61.010	54.830	49.849
10	Froud No. Fr = q/(gy13)0.5	4.370	4.949	4.296	3.184

11.1.12 Cut-off design of under sluice on silty foundation

discharge caried by undersluice	1189.650	cumecs
total water way of undersluice	28.000	m
discharge intensity	42.488	cumecs/m
scour depth , R = 1/3(q2/f)^1/3	13.047	m
Downstream cut-off:		
Provide d/s cut-off at 1.5R below the d/s water level which is 469.973	19.570	m
with 0.5 m retrogration. Hence, reduced level of bottom of downstream cut-off	450.403	m
Depth of d/s cut-off = d/s floor level - 0.5 - RL of d/s cutoff	10.520	m
provide 12m cutoff wall Up to RL of	448.923	m
Providing a diaphrgm wall as cut-off wall on D/S as foundation is resting on silt		
upstream stream cut-off:		
provide u/s cut-off at depth of 1.25R below the u/s water level	16.308	m
Level of bottom of U/S cut-off = U/S water level -1.25R =	466.126	m
Provide level of bottom of U/S cut-off =	455.423	m
Depth of U/S cut-off =	6.000	m
Intermidiet cut-off:		
provide 10m intermidiet cutoff to reduce pressure	10.000	m

11.1.13 Under sluice uplift pressure calculation on silty foundation

for pile-1					
	b=	150.000			
	d=	6.000			
	α=	25.000			
	λ=	13.010			
	ϕE=	0.179	17.893		
	ϕD=	0.126	12.568		
	ϕC1=	82.107			
	ϕD1=	87.432			
correction for ϕC1					
	a	for pile 2 interfrance			
		D=	9.000		
		d=	5.000		
		b=	150.000		
		b'=	52.500		
		C1	0.734	positive	
	b	for the slope: corection is nil			
	c	for the floor thickness			
		ϕD1=	87.432		
		ϕC1=	82.107		
		t=ASSUMED	1.000		
		dis.b/n CD	6.000		
		C2	0.888	positive	
	hence, corrected ϕC1 =		83.729		
		ϕE1 =	100.000		
for pile-2					
b1=	53.500				
b2=	95.500				
d=	10.000				
α1=	5.350				
α2=	9.550				
λ=	7.522				
λ1=	-2.080				
ϕE2=	0.635	63.459			
ϕD2=	0.589	58.947			
ϕC2=	0.546	54.613			

a	for pile 1 interfrance					
	D=	5.000				
	d=	9.000				
	b=	150.000				
	b'=	52.500				
	C1	0.547	negative			
b	for the slope: corection is nil					
c	for the floor thickness					
	ϕE2=	63.459				
	ϕD2=	58.947				
	t=ASSUMED	1.000				
dis.b/n E2D2		10.000				
	C2	0.451	negative			
hence, corrected ϕE2 =		62.460				
correction for ϕC2						
a	for pile 3 interfrance					
	D=	11.500				
	d=	9.000				
	b=	150.000				
	b'=	95.400				
	C1	0.902	POSITIVE			
b	for the slope					
	from table for 3H:1V,slope correc.factor=					
	bs=	19.000				
	b1=	94.500				
	c2=	0.905	POSITIVE			
c	for the floor thickness,same correction as E2D2 but oposite sign					
	ϕE2=	63.459				
	ϕD2=	58.947				
	t=assumed	1.000				
	dis.b/n E2D2	10.000				
	C2	0.451	POSITIVE			
hence corrected ϕC2 =		56.870				

for pile-3			
b=	150.000		
d=	12.000		
α=	12.500		
λ=	6.770		
ϕE3=	0.251	25.126	
ϕD3=	0.175	17.530	
ϕC3=	0.000		
correction for ϕE3			
a	for pile 2 interfrance		
	D=	8.000	
	d=	10.500	
	b=	150.000	
	b'=	94.500	
	C1	0.682	negative
b	for the slope: corection is nil		
c	for the floor thickness		
	ϕD3=	17.530	
	ϕE3=	25.126	
	t=assumed	1.500	
	dis.b/n E3D3	12.000	
	C2	0.950	negative
hence ,corrected ϕE3 =		23.495	

11.1.14 Corrected pressure calculation on silty foundation

pile -1	**%**	**pile-2**	**%**	**pile-3**	**%**
ϕE1=	100.000	ϕE2=	62.460	ϕE3=	23.495
ϕD1=	87.432	ϕD2=	58.947	ϕD3=	17.530
ϕC1=	83.729	ϕC2=	56.870	ϕC3=	0.000

11.1.15 Elevation of Hydraulic grade line HGL

d/s WL	Head		ϕE1	ϕD1	ϕC1	ϕE2	ϕD2	ϕc2	ϕE3	ϕd3	ϕc3
	m		(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
			100.000	87.432	83.729	62.460	58.947	56.870	23.495	17.530	0.000
462.033	15.690	press.h	15.690	13.718	13.137	9.800	9.249	8.923	3.686	2.751	0.000
		loss	0.000	1.972	2.553	5.890	6.441	6.767	12.004	12.939	15.690
		HGL	477.723	475.751	475.170	471.833	471.282	470.956	465.719	464.784	462.033
469.973	12.461	press.h	12.461	10.895	10.433	7.783	7.345	7.087	2.928	2.184	0.000
		loss	0.000	1.566	2.028	4.678	5.116	5.374	9.533	10.277	12.461
		HGL	482.434	480.868	480.406	477.756	477.318	477.060	472.901	472.157	469.973
464.423	13.300	press.h	13.300	11.628	11.136	8.307	7.840	7.564	3.125	2.332	0.000
		loss	0.000	1.672	2.164	4.993	5.460	5.736	10.175	10.968	13.300
		HGL	477.723	476.051	475.559	472.730	472.263	471.987	467.548	466.755	464.423

11.1.16 Floor thickness calculation using corrected pressure

dist.from d/s end ,m	d/s end	10.000	30.000	60.000
unbalanced pressure,m	3.686	4.225	5.303	6.919
thickness	2.633	3.018	3.788	4.942
design thickness	3.000	3.200	3.800	5.000

11.1.17 Pre-jump Profile Calculations

Distance from the d/s end of the crest,i.e the start of glacis,m	glacis level (m)	high flood flow q =50.985 Ef1 =u/s TEL-Glacis level 484.585- col.(2)	y1 from plate 10.2	pond level flow q= 49.591 Ef1 =u/s TEL-Glacis lev 482.411- col.(2)	y1 from plate 10.2
col.1	col.2	col.3	col.4	col.5	col.6
0.000	461.423	23.162	2.940	20.988	3.190
3.000	460.423	24.162	2.865	21.988	3.090
5.600	459.556	25.029	2.805	22.855	3.020
8.500	458.590	25.995	2.740	23.821	2.940
10.230	458.013	26.572	2.710	24.398	2.900

11.1.18 Post-jump Profile Calculations

Frude Nr. for high flow condition,Fr =	4.949	**Fr2 =**	**24.496**		
Depth y1 for high flood condition =	2.212				
Frude Nr. for pond level flow condition,Fr =	4.296	**Fr2 =**	**18.456**		
Depth y1 pond level flow condition =	2.386				

	high flood flow			**pond level flow**		
x/y1	**Fr2 =24.496, y1 = 2.212**			**Fr2= 18.456, y1 = 2.386**		
plate 10.3(a)	**y/y1**	**y = 2.212* col.2**	**X = col .1 * 2.212**	**y/y1**	**y =2.386*col.5**	**X = col 1 * 2.386**
col-1	**col 2**	**col 3**	**col 4**	**col 5**	**col 6**	**col 7**
0.500	1.100	2.433	1.106	1.100	2.625	1.193
1.000	1.200	2.654	2.212	1.200	2.863	2.386
2.500	1.500	3.318	5.530	1.500	3.579	5.965
5.000	2.700	5.972	11.060	2.700	6.442	11.930
8.500	3.600	7.963	18.802	3.700	8.828	20.281
10.000	3.850	8.516	22.120	3.850	9.186	23.860
12.000	4.000	8.848	26.544	3.920	9.353	28.632
13.000	4.200	9.290	28.756	4.000	9.544	31.018
16.500	4.520	9.998	36.498	4.150	9.902	39.369
17.798	4.800	10.618	39.369	4.230	10.093	42.466
20.000	5.150	11.392	44.240	4.650	11.095	47.720
25.600	5.300	11.724	56.627	4.830	11.524	61.082
30.000	6.100	13.493	66.360	5.800	13.839	71.580
31.000	6.200	13.714	68.572	6.020	14.364	73.966
33.500	6.600	14.599	74.102	6.200	14.793	79.931
34.630	6.820	15.086	76.602	6.500	15.509	82.627

11.1.19 Unbalanced heads for High Flood flow conditions

Hor.dist. from start of glacis,x	Flow depth	Glacis or d/s apron level	Actual w.s e due high flo d/s discharge	W.S. ELV. Col.(2) +Col.	H.G.L elv.	Unbalanced head col 6-col.5	Remark
col-1	col-2	col-3	col-4	col-5	col-6	col-7	col-8
0.000	2.940	461.423	469.973	472.913	477.372	4.459	Pre-jump
3.000	2.865	460.423	469.973	472.838	477.234	4.396	
5.600	2.805	459.556	469.973	472.778	477.114	4.336	
8.500	2.740	458.590	469.973	472.713	476.981	4.268	
10.230	2.710	458.013	469.973	472.683	476.901	4.218	
13.500	2.645	456.923	469.973	472.618	476.751	4.133	
17.250	2.577	455.673	469.973	472.550	476.579	4.029	
18.969	2.550	455.100	469.973	472.523	476.499	3.976	
20.075	2.433	455.100	469.973	472.406	476.449	4.042	P0st-jump
21.181	2.654	455.100	469.973	472.627	476.398	3.770	
24.499	3.318	455.100	469.973	473.291	476.245	2.954	
30.029	5.972	455.100	469.973	475.945	475.991	0.045	
37.771	7.963	455.100	469.973	477.936	475.635	-2.302	
41.089	8.516	455.100	469.973	478.489	475.482	-3.007	
45.513	8.848	455.100	469.973	478.821	475.278	-3.543	
47.725	9.290	455.100	469.973	479.263	475.177	-4.087	
55.467	9.998	455.100	469.973	479.971	474.821	-5.151	
58.338	10.618	455.100	469.973	480.591	474.688	-5.902	
63.209	11.392	455.100	469.973	481.365	474.464	-6.900	
75.596	11.724	455.100	469.973	481.697	473.895	-7.802	
85.329	13.493	453.910	469.973	483.466	473.447	-10.019	
87.541	13.714	455.181	469.973	483.687	473.345	-10.342	
93.071	14.599	458.360	469.973	484.572	473.091	-11.481	
95.571	15.086	460.943	469.973	485.059	472.976	-12.083	

11.1.20 Unbalanced heads for pond level flow condition.

Hor.dist. from start of glacis,x	**Flow depth**	**glacis or d/s apron level**	**actual w.s el. due pond level flow**	**W.S. ELV. Col.(2) +Col.(4)**	**H.G.L elv.**	**unbalanced head, col 6-col.5**	**remark**
col-1	**col-2**	**col-3**	**col-4**	**col-5**	**col-6**	**col-7**	**col-8**
0.000	3.190	461.423	464.923	468.113	472.320	4.207	Pre-jump
3.000	3.090	460.423	464.923	468.013	472.173	4.160	
5.600	3.020	459.556	464.923	467.943	472.045	4.102	
8.500	2.940	458.590	464.923	467.863	471.903	4.040	
10.230	2.900	458.013	464.923	467.823	471.818	3.995	
13.500	2.829	456.923	464.923	467.752	471.657	3.905	
17.250	2.740	455.673	464.923	467.663	471.473	3.810	
18.969	2.708	455.100	464.923	467.631	471.389	3.758	
20.162	2.625	455.100	464.923	467.548	471.330	3.783	P0st-jump
21.355	2.863	455.100	464.923	467.786	471.272	3.486	
24.934	3.579	455.100	464.923	468.502	471.096	2.594	
30.899	6.442	455.100	464.923	471.365	470.803	-0.562	
39.250	8.828	455.100	464.923	473.751	470.393	-3.358	
42.829	9.186	455.100	464.923	474.109	470.218	-3.892	
47.601	9.353	455.100	464.923	474.276	469.983	-4.293	
49.987	9.544	455.100	464.923	474.467	469.866	-4.601	
58.338	9.902	455.100	464.923	474.825	469.456	-5.369	
61.435	10.093	455.100	464.923	475.016	469.304	-5.712	
66.689	11.095	455.100	464.923	476.018	469.046	-6.972	
80.051	11.524	455.100	464.923	476.447	468.390	-8.057	
90.549	13.839	456.910	464.923	478.762	467.875	-10.887	
92.935	14.364	458.282	464.923	479.287	467.758	-11.529	
98.900	14.793	460.923	464.923	479.716	467.465	-12.251	
101.596	15.509	460.923	464.923	480.432	467.333	-13.099	

11.1.21 Unbalanced head for Maximum Static Head Condition

Hor.dist. from start of glacis,x	Flow depth	Glacis or d/s apron level	Actual w.s el. due min discharge	W.S. ELV. Col.(2) +Col.(4)	H.G.L elv.	Unbalanced head, col 6-col.5	Remark
col-1	col-2	col-3	col-4	col-5	col-6	col-7	col-8
0.000	0.610	461.423	462.033	462.033	471.349	9.316	Pre-jump
3.000	0.610	460.423	462.033	462.033	471.175	9.142	
5.600	0.610	459.556	462.033	462.033	471.025	8.992	
8.500	0.610	458.590	462.033	462.033	470.857	8.824	
10.230	0.610	458.013	462.033	462.033	470.757	8.724	
13.500	0.610	456.923	462.033	462.033	470.567	8.534	
17.250	0.610	455.673	462.033	462.033	470.350	8.317	
18.969	0.610	455.100	462.033	462.033	470.250	8.217	
23.651	0.610	455.100	462.033	462.033	469.979	7.946	P0st-jump
25.204	0.610	455.100	462.033	462.033	469.889	7.856	
29.863	0.610	455.100	462.033	462.033	469.619	7.586	
37.628	0.610	455.100	462.033	462.033	469.170	7.137	
48.499	0.610	455.100	462.033	462.033	468.540	6.507	
53.158	0.610	455.100	462.033	462.033	468.270	6.237	
58.338	0.610	455.100	462.033	462.033	467.970	5.937	
62.476	0.610	455.100	462.033	462.033	467.731	5.698	
68.688	0.610	455.100	462.033	462.033	467.371	5.338	
73.347	0.610	455.100	462.033	462.033	467.101	5.068	
84.218	0.610	453.271	462.033	462.033	466.471	4.438	
101.612	0.610	460.923	462.033	462.033	465.464	3.431	

11.1.22 Floor thickness calculation

Hor.dist.from start of glacis,x	unbalanced head	uplift pressure	floor thickness calculated	floor thickness provided
0.000	9.316	9.316	6.654	6.700
5.600	8.992	8.992	6.423	6.500
18.969	8.217	8.217	5.870	6.000
58.338	5.937	5.937	4.241	5.000

11.1.23 Design of under sluice on rocky foundation

1. Design Input Parameters	Quantity	Units
Average deepest river bed level at weir site	461.423	m
Design discharge after Gilgel Gibe III dam will in operation		
Design dischargefor 1 in 100 years flood	7931.000	cumecs
Design dischargefor 1 in 500 years flood	13610.000	cumecs
HFL for 1 in 100 years flood before construction of Weir	470.473	m
HFL for 1 in 500 years flood before construction of Weir	472.473	m
Full supply level of R/B & L/B Main Canal at Offtake Point	476.723	m
Lacey's silt factor 'f'	3.000	
Assumed Concentration	0.200	%
Assumed river bed retrogression	0.000	m
2. Fixing of Crest Level and Waterways		
For silty part Lacey's wetted perimeter required P = 4.83*(Q)0.5	430.141	m
River bed width Abut. to Abut. available	397.000	m
Looseness Factor =	0.923	
Crest Level		
No. of offtaking canal	2.000	No.
No. of under sluices	2.000	No.
undersluice crest levels same as U/S floor	461.423	m
The height of weir portion	16.300	m
Working head to feed both sides of canals to FSL	1.000	m
The crest level of the weir portion	477.723	m
Waterway Proposed		
For silty part Lacey's wetted perimeter required P = 4.83*(Q)0.5	430.141	
eight under sluice bays of 3.5 m width each,total 8*3.5	28.000	m
The clear water way width adopted for the weir bay is	105.000	m
The embankment width adopted is	238.000	m
Width of bridge piers= 10*0.90	9.000	m
Fish ladder with 1.5 m width and a partition wall of 2 m width.	3.500	m
thickness of two divide walls =2*2.5	5.000	m
six undersluice peirs of total width = 6*2	12.000	m
The total waterway provided between Abut. To Abut.	397.000	m
Looseness Factor =	0.923	
Average dischareg intensity ,q = Q/ Total overall water way	19.977	m3/s/m
Scour Depth ,R =0.473*(Q/f)^1/3	6.540	m
Velocity of approach v = q/R =	3.055	m/s
Velocity head = v2/2g =	0.476	m
Head over the crest of weir for Q1oo flood	4.711	m
HFL with afflux at weir site	482.434	m

Head over the crest of weir for Q5oo flood	6.966	m
HFL with afflux at weir site	484.689	m
U/S TEL= (Pond level with Afflux) + Velocity head	482.910	m
D/S TEL=D/S water level (HFL)+Velocity head =	470.949	m
Head over the undersluice way crest = H1	21.487	m
Head over the ogee shaped weir crest = H2	5.187	m
Design of Under sluice portion		
1. For maximum design flood		
(a) Without concentration and retrogression of river bed		
During maximun design flood , the u/s water level with afflux	482.434	m
The HFL d/s during high flood=	470.473	m
Therefore, head causing flow ,h =	11.961	m
Let the gate opening be 'x' m ; the discharge can then be calculated		
with the help of submerged orifice formula as given below :-		
Q = Cd*A*(2*g*h)0.5 ,where Cd = 0.62 and A = 8*3.5*X		
Therefore, to pass 15% of Q100	1189.650	
0.15*Q100 = 0.62*8*3.5*X(2*9.81*h)^0.5		
0.15*7931 = 0.62*8*3.5*X (2*9.81*11.961)^0.5		
X =	4.500	m
The velocity of flow through the opening =Q15%/A	9.442	m/s
Loss of head at entry =0.5*v2/2g	2.272	m
Velocity head calculated above =	0.476	m
T.E.L at just U/S of gate =	482.910	m
T.E.L at just D/S of gate =	480.638	m
U/S T.E.L =	482.910	m
D/S T.E.L without retrogression ==	470.949	m
Head Loss =	9.689	m
Discharge intensity 'q' = Q/width of opening =	42.488	cumecs/m
(b) With 20% concentration and without retrogression of river bed		
Discharge intensity, q = 1.2*q	50.985	cumecs/m
Head causing the discharge intensity h =	17.020	m
50.985 = Cd*X*(2gh)^0.5		
Therefore, U/S water level = HFL d/s during high flood + h	487.493	m
The D/S water level without retrogression of 0m =	470.473	m
Velocity of flow through the opening = Q15%/A	11.330	m/s
Loss of head at entry =0.5*v2/2g	3.271	m
T.E.L at just U/S of gate =	487.969	m
T.E.L at just D/S of gate =	484.697	m
U/S T.E.L =	487.969	m
D/S TEL with retrogression =	470.949	m
Therefore, the head loss HL =	13.749	m

2. Pond level flow condition		
(a) For no concentration and no retrogression		
pond level	477.723	m
From the rating curve at weir axis,the Q value at 477.723m is projected as	9257.000	cumecs
Average discharge intensity at the pond level to pass 15% of Q100 (q)=	49.591	cumecs/m
Scour depth R =0.473*(Q/f)1/3 (Assume f= 3)	3.659	m
Velocity of approach=q/R	13.554	m/s
Veocity head=v2/2g	9.363	m
U/S TEL=	487.086	m
From the rating curve,the D/S water level for 15 % of design discharge the le	464.923	m
D/S TEL	474.286	m
Let the gate opening be 'x' m ; the discharge can then be calculated		
with the help of submerged orifice formula as given below :-		
Q = Cd*A*(2*g*h)0.5 ,where Cd = 0.62 and A = 8*3.5*X		cumecs
Therefore, to pass 15% of Q100		
0.15*Q100 = 0.62*8*3.5*X(2*9.81*h)^0.5		
0.15*9257 = 0.62*8*3.5*X (2*9.81*11.961)^0.5		
X =	5.050	m
velocity through opening = q/x	9.820	cumecs/m
Loss of head at entry = 0.5*v2/2g	2.458	m
T.E.L at just D/S of gate =	484.629	m
Head loss =	10.342	m
Discharge intensity through under sluice =	49.591	cumecs/m
(b) Pond Level Flow With 20% concentration & without		
river bed retrogression		
Discharge intensity with 20% concentration	59.509	cumecs/m
Scour depth R =0.473*(Q/f)1/3 (Assume f=3)	3.888	m
Velocity of approach=q/R	15.306	m/s
Veocity head=v2/2g	11.940	m
From the rating curve,the D/S water level for 15 % of design discharge the le	464.923	m
D/S TEL	476.863	m
Head causing the discharge intensity h =	18.400	m
Therefore, U/S water level =	483.323	m
The D/S water level without retrogression=	464.923	m
D/S TEL with retrogression =	476.863	m
Velocity of flow through the opening =(0.15*9257*1.2)/(28*4.5)=	13.224	m/s
Loss of head at entry = 0.5*v2/2g	4.457	m
T.E.L at just U/S of gate =	483.799	m
T.E.L at just D/S of gate =	479.342	m
Therefore, the head loss HL=	2.479	m

11.1.24 Length of concrete floor required and downstream Cistern level

S.No	Item	high flood flow		pond level flow	
		without conc. and retrogn.	with concn. and retrogn.	without conc. and retrogn.	with conc. and retrogn.
1	Discharge intensity,q in cumecs/meter	42.488	50.985	49.591	59.509
2	Upsteam water level	482.434	482.434	477.723	477.723
3	Downstream water level	470.473	470.473	464.923	464.923
4	U/s TEL	480.638	484.697	484.629	479.342
5	D/s TEL	470.949	470.949	474.286	476.863
6	Head loss HL	9.689	13.749	10.342	2.479
	(8*q2*HL)/g = (-1.5*y1+((y12/4) +((2*q2)/(g*y1)))0.5)3*(0.5*y1+((y12/4)+((2*q2)/(g*y1)))0.5)				
	(8*q2*HL)/g	14263.683	29145.062	20742.117	7159.006
	let the value of y1	2.128	2.237	2.386	3.960
	(-1.5*y1+((y12/4) +((2*q2)/(g*y1)))0.5)3*	14265.944	29153.361	20753.833	7153.739
	(0.5*y1+((y12/4)+((2*q2)/(g*y1)))0.5)				
7	After ittiration we get the value of y1	2.128	2.237	2.386	3.960
8	y2 =(-0.5*y1+((y12/4)+((2*q2)/(g*y1)))0.5	12.130	14.314	13.352	11.667
9	Ef2=y2+(q2/2gy22)	12.755	14.961	14.055	12.993
10	Ef1 = H L + Ef2	22.444	28.709	24.398	15.472
11	Level at which jump will form.i.e (d/s TEL - Ef2)	458.193	455.988	460.231	463.870
12	length of concrete floor required = 5*(y2- y1)	50.009	60.385	54.830	38.535
13	Froud No. Fr = q/(gy13)0.5	4.370	4.865	4.296	2.411

11.1.25 Design of cut-off walls for under sluice on rocky foundations

discharge caried by undersluice	1189.650	cumecs
total water way of undersluice	28.000	m
discharge intensity	42.488	cumecs/m
scour depth , R = 1/3(q2/f)^1/3	11.397	m
Downstream cut-off:		
Provide d/s cut-off at 1.5R below the d/s water level which is 470.473	17.096	m
with out a retrogration. Hence, reduced level of bottom of downstream cut-off	453.377	m
Depth of d/s cut-off = d/s floor level - RL of d/s cutoff	8.046	m
provide 9 m cutoff wall Up to RL of	451.923	m
Providing a diaphrgm wall as cut-off wall on		
D/S as foundation is resting on silt		
upstream stream cut-off:		
provide u/s cut-off at depth of 1.25R below the u/s water level	14.247	m
Level of bottom of U/S cut-off = U/S water level -1.25R =	468.187	m
Provide level of bottom of U/S cut-off =	455.423	m
Depth of U/S cut-off =	6.000	m
Intermidiet cut-off:No need of intermidiet cutoff b/c of rocky foundation		

11.1.26 Under sluice uplift pressure calculation on rocky foundation

For U/S pile-1				
	b=	150.000		
	d=	6.000		
	α=b/d	25.000		
	λ=0.5*(1+(1+α^2)	13.010		
	ϕE=1/PI*ACOS((λ-2)/2)	0.179	17.893	
	ϕD=1/PI*ACOS((λ-1)/2)	0.126	12.568	
	ϕC1= 100 -ϕE	82.107		
	ϕD1=100 -ϕd	87.432		
	ϕE1 =	100.000		
correction for ϕC1				
	a	**for d/s pile 2 interfrance**		
		D=	8.500	
		d=	5.000	
		b=	150.000	
		b'=	148.000	
		C1=19*(D/b')^0.5*((D+d)/b)	0.410	positive
	b	**for the slope: corection is nil**		
	c	**for the floor thickness**		
		ϕD1=	87.432	
		ϕC1=	82.107	
		t=ASSUMED	1.000	
		dis.b/n CD	6.000	
		C2	0.888	positive
		hence, corrected ϕC1 =	83.404	

For d/s pile no.2				
	b=	150.000		
	d=	9.000		
	α=b/d	16.667		
	λ=0.5*(1+(1+α^2)	8.848		
	ϕE=ϕE2=1/PI*ACOS((λ-2)/2)	0.218	21.83796	
	ϕD=ϕD2=1/PI*ACOS((λ-1)/2)	0.153	15.28736	
	ϕC2=	0.000		
	correction for ϕE3			
	a	**for pile 2 interfrance**		
		D=	4	
		d=	7.5	
		b=	150	
		b'=	148	
		C=19*(D/b')^0.5*((D+d)/b)	0.239475	negative
	b	**for the slope: corection is nil**		
	c	**for the floor thickness**		
		ϕD2=	15.28736	
		ϕE2=	21.83796	
		t=assumed	1.5	
		dis.b/n E2D2	9	
		C2	1.091766	negative
	hence ,corrected ϕE2 =	20.507		

11.1.27 Corrected pressure calculation

pile -1	%	pile-2	%
ϕE1=	100	ϕE2=	20.507
ϕD1=	87.4319	ϕD2=	15.287
ϕC1=	83.4041	ϕC2=	0.000

11.1.28 Elevation of Hydraulic grade line HGL

U/S WL	d/s WL	Head,m		ϕE1(%)	ϕD1(%)	ϕC1(%)	ϕE2 (%)	ϕD2 (%)	ϕc2 (%)
477.723	462.033	15.690		100.000	87.432	83.404	20.507	15.287	0.000
			press.head	15.690	13.718	13.086	3.218	2.399	0.000
			loss	0.000	1.972	2.604	12.472	13.291	15.690
			HGL	477.723	475.751	475.119	465.251	464.432	462.033
482.434	470.473	11.961	press.head	11.961	10.458	9.976	2.453	1.829	0.000
			loss	0.000	1.503	1.985	9.508	10.132	11.961
			HGL	482.434	480.931	480.449	472.926	472.302	470.473
477.723	464.923	12.800	press.head	12.800	11.191	10.676	2.625	1.957	0.000
			loss	0.000	1.609	2.124	10.175	10.843	12.800
			HGL	477.723	476.114	475.599	467.548	466.880	464.923

11.1.29 Floor thickness calculation

Floor thickness calculation				
dist.from d/s end ,m	d/s end	15.000	30.000	60.000
unbalanced pressure,m	2.413	3.153	3.893	5.374
thickness	1.724	2.252	2.781	3.838
design thickness,m	2.000	2.500	3.000	4.000

11.1.30 Pre-jump Profile Calculations

Distance from the d/s end of the crest i.e the start of glacis,m	Glacis level m	High flood flow q =50.985		Pond level flow q= 49.591	
		Ef1 =u/s TEL-Glacis level 484.697- col.(2)	y1 from plate 10.2	Ef1 =u/s TEL-Glacis 484.629- col.(2)	y1 from plate 10.2
col.1	col.2	col.3	col.4	col.5	col.6
0.000	461.423	23.274	2.932	23.206	2.990
3.000	460.423	24.274	2.857	24.206	2.910
5.600	459.556	25.141	2.800	25.073	2.850
8.500	458.590	26.107	2.735	26.039	2.790
10.230	458.013	26.684	2.700	26.616	2.750
13.500	456.923	27.774	2.641	27.706	2.690
15.500	456.256	28.441	2.600	28.373	2.650
16.569	455.900	28.797	2.583	28.729	2.630

11.1.31 Post jump profile

Frude Nr.for high flow condition, Fr =4.865, Fr^2 = 23.668

Depth y1 for high flood condition = 2.237

Frude Nr.for pond level flow condition, Fr = 4.296, Fr^2 = 18.456

Depth y1 pond level flow condition = 2.386

x/y1	high flood flow			pond level flow		
plate 10.3(a)	Fr2 =23.668,y1 = 2.237			Fr2= 18.456, y1 = 2.386		
	y/y1	y = 2.237* col.2	X = col .1 * 2.237	y/y1	y =2.386*col.5	X = col 1 * 2.386
col-1	col 2	col 3	col 4	col 5	col 6	col 7
0.500	1.101	2.463	1.119	1.100	2.625	1.193
1.000	1.200	2.684	2.237	1.200	2.863	2.386
2.500	1.510	3.378	5.593	1.500	3.579	5.965
5.000	2.700	6.040	11.185	2.700	6.442	11.930
8.500	3.602	8.058	19.015	3.700	8.828	20.281
17.506	3.850	8.612	39.161	3.850	9.186	41.769
18.672	4.080	9.127	41.769	3.920	9.353	44.551
20.000	4.200	9.395	44.740	4.000	9.544	47.720
25.600	4.520	10.111	57.267	4.150	9.902	61.082
27.890	4.800	10.738	62.390	4.230	10.093	66.546
29.000	5.150	11.521	64.873	4.650	11.095	69.194
30.350	5.300	11.856	67.893	4.830	11.524	72.415
31.500	6.101	13.648	70.466	5.800	13.839	75.159
32.378	6.200	13.869	72.430	6.020	14.364	77.254

11.1.32 Unbalanced heads in the jump trough for High flood flow condition.

Hor.dist. from start of glacis,x	Flow depth	Glacis or d/s apron level	Actual w.s el. due high flood d/s discharge	W.S. ELV. Col.(2) +Col.(4)	H.G.L elv.	Unbalanced head col 6-col.5	Remark
col-1	col-2	col-3	col-4	col-5	cpl-6	col-7	col-8
0.000	2.932	461.423	470.473	473.405	476.432	3.027	Pre-jump
3.000	2.857	460.423	470.473	473.330	476.231	2.901	
5.600	2.800	459.556	470.473	473.273	476.057	2.784	
8.500	2.735	458.590	470.473	473.208	475.863	2.655	
10.230	2.700	458.013	470.473	473.173	475.747	2.574	
13.500	2.641	456.923	470.473	473.114	475.528	2.414	
15.500	2.600	456.256	470.473	473.073	475.394	2.321	
16.569	2.583	455.900	470.473	473.056	475.322	2.266	
17.688	2.625	455.900	470.473	473.098	475.248	2.150	P0st-jump
18.806	2.863	455.900	470.473	473.336	475.173	1.837	
22.162	3.579	455.900	470.473	474.052	474.948	0.896	
27.754	6.442	455.900	470.473	476.915	474.574	-2.342	
35.584	8.828	455.900	470.473	479.301	474.049	-5.252	
55.730	9.186	455.900	470.473	479.659	472.701	-6.959	
58.338	9.353	455.900	470.473	479.826	472.526	-7.300	
61.309	9.544	455.900	470.473	480.017	472.327	-7.690	
73.836	9.902	455.900	470.473	480.375	471.488	-8.887	
78.959	10.093	455.900	470.473	480.566	471.145	-9.421	
81.442	11.095	455.900	470.473	481.568	470.979	-10.589	
84.462	11.524	455.900	470.473	481.997	470.777	-11.221	
87.035	13.839	460.831	470.473	484.312	470.605	-13.707	
88.999	14.364	460.923	470.473	484.837	470.473	-14.364	

11.1.33 Unbalanced heads in the jump trough for pond level flow condition

Hor.dist. from start of glacis,x	Flow depth	glacis or d/s apron level	actual w.s e due pond level flow	W.S. ELV. Col.(2) +Col.(4)	H.G.L elv.	unbalanced head, col 6-col.5	remark
col-1	col-2	col-3	col-4	col-5	cpl-6	col-7	col-8
0.000	2.990	461.423	464.923	467.913	471.300	3.387	Pre-jump
3.000	2.910	460.423	464.923	467.833	471.085	3.252	
5.600	2.850	459.556	464.923	467.773	470.899	3.126	
8.500	2.790	458.590	464.923	467.713	470.691	2.978	
10.230	2.750	458.013	464.923	467.673	470.567	2.894	
13.500	2.690	456.923	464.923	467.613	470.333	2.720	
15.500	2.650	456.256	464.923	467.573	470.189	2.616	
16.569	2.630	455.900	464.923	467.553	470.113	2.560	
17.762	2.625	455.900	464.923	467.548	470.027	2.480	P0st-jump
18.955	2.863	455.900	464.923	467.786	469.942	2.156	
22.534	3.579	455.900	464.923	468.502	469.685	1.183	
28.499	6.442	455.900	464.923	471.365	469.258	-2.107	
36.850	8.828	455.900	464.923	473.751	468.660	-5.092	
58.338	9.186	455.900	464.923	474.109	467.120	-6.989	
61.120	9.353	455.900	464.923	474.276	466.921	-7.356	
64.289	9.544	455.900	464.923	474.467	466.694	-7.773	
77.651	9.902	455.900	464.923	474.825	465.736	-9.089	
83.115	10.093	455.900	464.923	475.016	465.345	-9.671	
85.763	11.095	455.900	464.923	476.018	465.155	-10.863	
88.984	11.524	455.900	464.923	476.447	464.924	-11.523	
91.728	13.839	460.923	464.923	478.762	464.728	-14.034	

11.1.34 Unbalanced heads Maximum Static Head Condition

Hor.dist. from start of glacis,x	Flow depth	glacis or d/s apron level	actual w.s el. due minimum flow	W.S. ELV. Col.(2) +Col.(4)	H.G.L elv.	unbalanced head, col 6-col.5	remark
col-1	col-2	col-3	col-4	col-5	cpl-6	col-7	col-8
0.000	0.610	461.423	462.033	462.033	469.700	7.667	Pre-jump
3.000	0.610	460.423	462.033	462.033	469.442	7.409	
5.600	0.610	459.556	462.033	462.033	469.218	7.185	
8.500	0.610	458.590	462.033	462.033	468.968	6.935	
10.230	0.610	458.013	462.033	462.033	468.819	6.786	
13.500	0.610	456.923	462.033	462.033	468.537	6.504	
15.500	0.610	456.256	462.033	462.033	468.365	6.332	
16.569	0.610	455.900	462.033	462.033	468.273	6.240	
23.651	0.610	455.900	462.033	462.033	467.663	5.630	P0st-jump
25.204	0.610	455.900	462.033	462.033	467.529	5.496	
29.863	0.610	455.900	462.033	462.033	467.128	5.095	
37.628	0.610	455.900	462.033	462.033	466.459	4.426	
48.499	0.610	455.900	462.033	462.033	465.522	3.489	
53.158	0.610	455.900	462.033	462.033	465.121	3.088	
58.338	0.610	455.900	462.033	462.033	464.674	2.641	
62.476	0.610	455.900	462.033	462.033	464.318	2.285	
68.688	0.610	455.900	462.033	462.033	463.783	1.750	
73.347	0.610	455.900	462.033	462.033	463.381	1.348	
89.000	0.610	460.923	462.033	462.033	462.033	0.000	

11.1.35 Floor thickness provided

Hor.dist.from start of glacis,x	unbalanced head	75% of uplift pressure	floor thickness calculated	floor thickness provided
0.000	7.667	5.750	4.107	4.200
5.600	7.185	5.389	3.849	4.000
16.569	6.240	4.680	3.343	3.500
58.338	2.641	1.981	1.415	2.000

11.2 Appendix to Chapter Six

11.2.1 Engineering properties of foundation fill and filter materials

Shell earth fill		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	$1.55*10^{-6}$	WWDSE (2013). "Geological, Geophysical, Geotechnical and Seismic Hazard Study ."
Shell rock fill		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	10^{-1}	Dr.K.R.Arora(2003). "Soil mechanics and foundation engineering ." Delhi.
Core		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	$1* 10^{-10}$	Dr.K.R.Arora(2003). "Soil mechanics and foundation engineering ." Delhi.
Foundation Layer-1		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	5.77*10^-4	WWDSE (2013). "Geological, Geophysical, Geotechnical and Seismic Hazard Study."
Foundation Layer-2		
Engineering Property	Magnitude	Source

Coefficient of horizontal permeability (m/s)	$5.05*10^{-7}$	WWDSE (2013). “Kuraz headwork and appurtenance structure detail design.”
Foundation Layer-3		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	$5.05*10^{-5}$	WWDSE (2013). “Kuraz headwork and appurtenance structure detail design.”
Fine filter		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	$1*10^{-5}$	WWDSE (2013). “ Kuraz headwork and appurtenance structure detail design.”
Coarse filter		
Engineering Property	Magnitude	Source
Coefficient of horizontal permeability (m/s)	$1*10^{-4}$	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”

Shell earth fill		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	16.95	WWDSE (2013). “Geological, Geophysical, Geotechnical and Seismic Hazard Study .”
Cohesion C‟ (kN/m^2)	11.4	WWDSE (2013). “Geological, Geophysical, Geotechnical and Seismic Hazard Study .”
Cohesion C (kN/m^2)		
Angle of Friction ø‟ (degrees)	39.9 (35-45)	WWDSE (2013). “Geological, Geophysical, Geotechnical and Seismic Hazard Study .”
Angle of Friction ø		

(degrees)	35	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Coefficient of compressibility, m_v	0.102	WWDSE (2013). “Geological, Geophysical, Geotechnical and Seismic Hazard Study .”
Young‟s modulus E (kN/m^2)	$1*10^5$	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Poisson‟s ratio ν	0.35	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Shell rock fill		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	26.5	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Cohesion C‟ (kN/m^2)	0	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Cohesion C (kN/m^2)	0	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Angle of Friction ø‟ (degrees)	45	ENS-080312-EN-JZ. “Properties of Rock Materials.”
Angle of Friction ø (degrees)	40	ENS-080312-EN-JZ. “Properties of Rock Materials.”
Coefficient of compressibility, m_v	0.346	WWDSE (2013). “Geological, Geophysical, Geotechnical and Seismic Hazard Study .”
Young‟s modulus E (kN/m^2)	$3.5*10^4$	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Poisson‟s ratio ν	0.4	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”

Core		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	14.76	WWDSE (2013). "Geological, Geophysical, Geotechnical and Seismic Hazard Study ."
Cohesion C‟ (kN/m^2)	65	Dr.K.R.Arora(2003). "Soil mechanics and foundation engineering ." Delhi.
Angle of Friction ø‟ (degrees)	20	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition."
Coefficient of compressibility, m_v	0.2	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition."
Young‟s modulus E (kN/m^2)	7.5*10^4	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition."
Poisson‟s ratio ν	0.45	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition."
Foundation Layer-1		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	20.5	WWDSE(2013)."Kuraz headwork and appurtenance structure detail design."
Cohesion C‟ (kN/m^2)	0	Dr.K.R.Arora(2003). "Soil mechanics and foundation engineering ." Delhi.
Angle of Friction ø‟ (degrees)	35	WWDSE(2013)."Kuraz headwork and appurtenance structure detail design."
Coefficient of compressibility, m_v	0.134	WWDSE (2013). "Geological, Geophysical, Geotechnical and Seismic Hazard Study ."
Young‟s modulus E		WWDSE (2013). "Geological, Geophysical,

(kN/m^2)	7*10^4	Geotechnical and Seismic Hazard Study ."
Poisson‟s ratio ν	0.3	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition."
Foundation Layer-2		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	28.5	WWDSE(2013)."Kuraz headwork and appurtenance structure detail design."
Cohesion C‟ (kN/m^2)	35000	TecMec-L6."Hitched plug design."
Angle of Friction ø‟ (degrees)	42	TecMec-L6."Hitched plug design."
Coefficient of compressibility, m_v	0.346	WWDSE (2013). "Geological, Geophysical,Geotechnical and Seismic Hazard Study ."
Young‟s modulus E (kN/m^2)	2.5*10^7	WWDSE(2013)."Kuraz headwork and appurtenance structure detail design."
Poisson‟s ratio ν	0.2	ENS-080312-EN-JZ. "Properties of Rock Materials."
Foundation Layer-3		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	22.5	WWDSE(2013)."Kuraz headwork and appurtenance structure detail design."
Cohesion C‟ (kN/m^2)	33	WWDSE (2013). "Geological, Geophysical, Geotechnical and Seismic Hazard Study ."

Angle of Friction ø‟ (degrees)	40	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Coefficient of compressibility, m_v	0.28	WWDSE(2013).“Geological,Geophysical,Geotechnical and Seismic Hazard Study .”
Young‟s modulus E (kN/m^2)	$5*10^7$	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Poisson‟s ratio ν	0.35	WWDSE(2013).“Geological,Geophysical,Geotechnical and Seismic Hazard Study .”
Fine filter		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	17.92	Review of compaction principles and OMC formula
Cohesion C‟ (kN/m^2)	0	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Angle of Friction ø‟ (degrees)	35	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Coefficient of compressibility, m_v	$5.5*10^{-5}$	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Young‟s modulus E (kN/m^2)	25000	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Poisson‟s ratio ν	0.4	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Coarse filter		

Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	20.16	Review of compaction principles and OMC formula
Cohesion C‟ (kN/m^2)	0	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Angle of Friction ø‟ (degrees)	40	WWDSE(2013).“Kuraz headwork and appurtenance structure detail design.”
Coefficient of compressibility, m_v	$5.5*10^{-5}$	Dr.K.R.Arora(2003). “Soil mechanics and foundation engineering .” Delhi.
Young‟s modulus E (kN/m^2)	50000	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Poisson‟s ratio ν	0.35	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
concrete		
Engineering Property	Magnitude	Source
Dry unit weight $\gamma_{d\ max}$ (kN/m^3)	23	Review of compaction principles and OMC formula
Cohesion C‟ (kN/m^2)	40000	Dr.K.R. Arora (2003). “Soil mechanics and foundation engineering .” Delhi.
Angle of Friction ø‟ (degrees)	45	WWDSE (2013).“Kuraz headwork and appurtenance structure detail design.”
Young‟s modulus E (kN/m^2)	70*10^9	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”
Poisson‟s ratio ν	0.15	J.E. Bowles, McGraw-Hill (1996).Foundation Analysis and Design (5th Edition.”

11.3 Appendix to Chapter Eight

11.3.1 Estimated cost of alternative-1

Item	Description	Unit	Quantity	Rate (Birr)	Amount (Birr)
1	**Clearing and Grubbing**				
1.1	Site Clearance Dam and Weir	m^2	150000.000	6.660	999000.000
2	**Excavation**				
2.1	Excavate to diversion weir,scouring sluice abutment walls and wing walls to formation level including backfilling of selected granular material and disposal of surplus as fill in embankments or in spoil tips up to 500m away "soft"	m^3	560000.000	107.800	60368000.000
2.2	Extra over item 2.1 excavation in rock provisional	m^3	45000.000	379.621	17082937.500
2.3	Excavate in open cut to line and profiles to lay upstream dry stone pitching and cemented pitching and dispose in embankments or spoil tips to 500m away "soft"	m^3	24836.540	85.750	2129733.305
2.4	Excavate in open cut to line and profiles to lay downstream stone riprap and dispose in embankments or spoil tips up to 500m away "soft"	m^3	23263.500	85.750	1994845.125
3	Fill				
3.1	Common Earth Fill (Shell Fill)	m^3	623883.700	100.000	62388370.000
3.2	Select Earth Fill (Core Fill)	m^3	89845.000	140.000	12578300.000
3.3	Fine Drain Fill	m^3	130062.200	1100.000	143068420.000
3.4	Coarse Drain Fill	m^3	59590.440	1000.000	59590440.000
4	**Masonry Work**				
4.1	Dry stone pitching 1500mm thick on 150mm thick graded filter to bed	m^2	468.000	786.417	368043.369
4.2	Stone riprap 700 mm thick on earthfill Dam	m^2	16557.660	213.093	3528316.649
5	**Concrete work**				
5.2	Mass concrete class C-15 (1:3:6) to ogee part of the weir, Scouring sluice, upstream apron, downstream stilling basin and divide walls	m^3	84625.588	1967.350	166488189.339
5.3	7 rows of C.C. blocks (1.5mx 1.5mx 0.9m) with 10cm gap, for the main weir and undersluice (upsteam protection)	m^2	80.000	5817.176	465374.079
7	General Provision				5041365.000
8	Construction Costs Subtotal				545745398.805
9	Indirect costs				54574539.881
10	Total Construction and Indirect costs				600319938.686
11	Contingency (%): 20%				120063987.737
12	VAT at 15%				90047990.803
13	**Estimated Total Construction and Engineering Costs Plus Contingency:**				**1410751855.912**

11.3.2 Estimated cost of alternative-2

Item	Description	Unit	Quantity	Rate (Birr)	Amount (Birr)
1	**Clearing and Grubbing**				
1.1	Site Clearance embankment Dam and Weir	m^2	150000.000	6.660	999000.000
2	**Excavation**				
2.1	Excavate to diversion weir, scouring sluice abutment walls and wing walls to formation level including backfilling of selected granular material and disposal of surplus as fill in embankments or in spoil tips up to 500m away "soft"	m^3	560000.000	107.800	60368000.000
2.2	Extra over item 2.1 excavation in rock provisional	m^3	45000.000	379.621	17082937.500
2.3	Excavate in open cut to line and profiles to lay upstream dry stone pitching and cemented pitching and dispose in embankments or spoil tips to 500m away "soft"	m^3	20816.500	85.750	1785014.875
2.4	Excavate in open cut to line and profiles to lay downstream stone riprap and dispose in embankments or spoil tips up to 500m away "soft"	m^3	18082.350	85.750	1550561.513
3	**Fill**				
3.1	Common Earth Fill (Shell Fill)	m^3	224529.200	100.000	22452920.000
3.2	Common Rock Fill (Shell Fill)	m^3	570700.200	379.621	216649685.508
3.3	Select Earth Fill (Core Fill)	m^3	89845.000	140.000	12578300.000
3.4	Fine Drain Fill	m^3	130062.200	1100.000	143068420.000
3.5	Coarse Drain Fill	m^3	59590.440	1000.000	59590440.000
4	**Masonry Work**				
4.1	Dry stone pitching 1500mm thick on 150mm thick graded filter to bed	m^2	468.000	786.417	368043.369
4.2	Stone riprap 700 mm thick on 200mm thick graded filter to bed and sides	m^2	13877.780	213.093	2957253.756
5	**Concrete work**				
5.2	Mass concrete class C-15 (1:3:6) to ogee part of the weir, Scouring sluice, upstream apron,downstream stilling basin and divide walls	m^3	84625.588	1967.350	166488189.339
5.3	6 rows of C.C. blocks (1.5mx 1.5mx 0.9m) with 10cm gap, for the main weir and undersluice (upsteam protection)	m^2	80.000	5817.176	465374.079
5.4	8 rows of C.C. blocks (1.5mx 1.5mx 0.9m) with 10cm gap, for the main weir and undersluice (down steam protection)	m^2	522.000	5817.176	3036565.865
5.5	Concrete Class C-15 in blinding layer	m^3	22.860	1967.350	44973.631
5.6	Sand Bedding 600mm thick under weir bay structure	m^3	382.440	451.116	172524.944
6	Provide for the manufacturing and supervision of two intake and two scouring sluice slide gates including hoisting device. Gate Size for Undersluice = 8 nos. of 3.5m Wide x 4.5m high	No.	8.000	800000.000	6400000.000
7	General Provision				5041365.000
8	Construction Costs Subtotal				721099569.377
9	Indirect costs				72109956.938
10	Total Construction andIndirect costs				793209526.315
11	Contingency (%): 20%				158641905.263
12	VAT at 15%				118981428.947
13	**Estimated Total Construction and Engineering Costs Plus Contingency:**				**1864042386.840**

11.3.3 Estimated cost of alternative-3

Item	Description	Unit	Quantity	Rate (Birr)	Amount (Birr)
1	**Clearing and Grubbing**				
1.1	Site Clearance embankment Dam and Weir	m^2	150000.000	6.660	999000.000
2	**Excavation**				
2.1	Excavate to diversion weir, intake, scouring sluice abutment walls and wing walls to formation level including backfilling of selected granular material and disposal of surplus as fill in embankments or in spoil tips up to 500m away "soft"	m^3	560000.000	107.800	60368000.000
2.2	Extra over item 2.1 excavation in rock provisional	m^3	45000.000	379.621	17082937.500
2.3	Excavate in open cut to line and profiles to lay upstream dry stone pitching and cemented pitching and dispose in embankments or spoil tips to 500m away "soft"	m^3	9714.366	85.750	833006.885
2.4	Excavate in open cut to line and profiles to lay downstream stone riprap and dispose in embankments or spoil tips up to 500m away "soft"	m^3	8438.430	85.750	723595.373
4	**Masonry Work**				
4.1	Dry stone pitching 1500mm thick on 150mm thick graded filter to bed	m^2	9307.830	786.417	7319839.983
5	**Concrete work**				
5.2	Mass concrete class C-15 (1:3:6) to ogee part of the weir, Scouring sluice, upstream apron ,downstream stilling basin and divide walls	m^3	216765.070	1967.350	426452859.815
6	General Provision				400041365.000
7	Construction Costs Subtotal				924397010.602
8	Indirect costs				92439701.060
9	Total Construction andIndirect costs				1016836711.662
10	Contingency (%): 20%				203367342.332
11	VAT at 15%				152525506.749
12	**Estimated Total Construction and Engineering Costs Plus Contingency:**				**2389566272.405**

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