WOLKITE UNIVERSITY

COLLEGE OF ENGINEERING AND TECHNOLOGY

DEPARTMENT OF HYDRAULIC AND WATER RESOURCE ENGINEERING

FINAL YEAR PROJECT ON GENALE DAWA SIX HYDRO POWER PROJEC

JUNE/2019

WOLKITE

ETHIOPIA

FINAL YEAR PROJECT

GROUP MEMBER

NO NAME ID NO

MENGISTU KETSELA………………………….ENGR/752/07

ADANECH WOLDE…………………………….ENGR/074/07

FEKADU HAILEMARIAM……………………..ENGR/431/07

HIWOT TESFAYE………………………………ENGR/606/07

WONDIMAGEGN DESALEGN………………..ENGR/1131/07

ABDI JEMAL……………………………………..ENGR/010/07

GENET TEKA……………………………………ENGR/969/07

ACKNOWLEDGEMEN

In advance, our indebted gratitude goes to the Almighty God, for His mercy, love and care, which enable us to be alive and stay in the campus during the course of our study.

It is our pleasure to express our deepest gratitude to our advisors Mr. Kenenisa Bulti and

Mr. Tesfaye Abdeta for their consistent guidance and supervision. And it was also impressing that they were working with the group in harmony as a friend.

We would like to express our gratitude to all those who help us for the success and accomplishment of our final project, whose support was either direct or indirect during our project progress.

Our thanks still continue for inexpressible effort of our friends and classmates for giving us encouragement, valuable discussions throughout the work of this Project.

.

ABSTRACT

The Genale Dawa hydropower project design aims at utilizing the potential of Genale river to generate electricity, to support the power demand in the country. The power plant is located approximately 80km east of Negele, in Liben Zone of the Somali National Regional State. The project area is approximately 700km by road south and east of Addis Ababa on river Genale.

The flow for hydrologic analysis is transported to the proposed dam site using a coefficient incorporating catchments area ratios. The estimated annual sediment transported to the reservoir is about 1Mm3/year. A designing flood of 1800m3/s corresponding to 1000 year return period is chosen for designing the hydraulic structures.

Taking in to consideration the topography, economical aspects and the geological characteristics of the dam site gravity dam is found to be feasible. The dam height is 95m above the river bed including 6.51m height of spillway.

The live and dead storage amount is 615 and 76.923Mm^3 respectively.

The Environmental and Social Impact Assessment (ESIA) has been prepared in compliance with Ethiopian ESIA procedures and in accordance with international standards, as reflected in the policies, safeguard procedures, and guidelines of the African Development Bank.

The analysis of hydrological data, transposing of data from given station into the dam site, reservoir planning, dam dimension fixing and stability analysis, spillway structure and energy dissipating structure design are considered in this paper.

TABLE OF CONTENT

LIST OF FIGURE

LIST OF TABLE

CHAPTER ONE
1. ITRODUCTION
1.1 General
In the World where the demand for electric energy is steadily increasing, hydropower holds a unique position as the renewable energy source with highest potential in a medium range perspective. The region of the world where the need for energy is most pressing all possesses huge hydropower resources. With a century of experience to draw on, the hydropower community has extensive bases of knowledge and skills in support of every aspect of hydropower development.

Assessment of the world's technically exploitable hydroelectric energy potential is only about 14000TWh/year. However, the present developed electric energy is about 2134TWh/year, which is about 15.2% of the available hydropower potential. Thus, there is a vast scope for exploitation of hydropower.

(Source: Haresh Khemani, 2008)

1.2 Current Hydropower Status of Ethiopia
The Ethiopian electric power corporation presently maintains two different supply systems, namely, the interconnected system (ICS), which is mainly supplied from hydropower plants, and the self-connected system (SCS), which consists on mini hydropower plants and a number of isolated diesel generating units that are widely spread over the country. The proper development of the exploitable hydropower potential in the nation is planned to satisfy the national demand and export to the neighboring countries Djibouti, Sudan, Somalia and Kenya.

An attempt to implement a hydropower project helps directly or indirectly the country's economy as well as social developments. Basically, it enhances the economic development by providing sufficient energy. On the other hand, it promotes the social developments by improving the life standard of the people.

The existing situation indicates that the supply and demand of power is not balanced. Not only the demand being unbalanced with the supply but also the rate of demand much greater than that of the supply.

Due to power supply shortage and other reasons out lined above, currently the government of Ethiopia wishes to maximize the use of hydro resources for power generation. To that end, the objective in this particular project deals with the assessment of available alternative water resource potential in the Genale Dawa river, and to verify whether the potential projects appear technically, economically and environmentally feasible or not.

1.3 Description of the Project Area

1.3.1 Location

The GD-6 Hydropower project is located on the Genale River of the Genale Dawa River Basin, approximately 80km east of Negele, in Liben Zone of the Somali National Regional State. The project area is approximately 700km by road south and east of Addis Ababa. The project forms the downstream power plant in a series of three utilizing the large reservoir of the planned hydropower project GD-3 located some 82km further upstream along the Genale River. Just upstream of the reservoir of GD-6, there is a potential Hydropower Project GD-5, which forms the middle hydropower project in the series.

The locations of the projects are shown in Figure 1-1 below.

Figure1. 1 Location of Project

(*Source: Genale GD 6 Feasibility Study Report)*

1.3.2 Climate

As in all parts of the country climate is strongly influenced by the effects of elevation, which gives rise to distinct ones and characteristics. Traditional classifications based on altitude and temperature shows the presence of five zones, of which the following three are predominant in Genale project river basin.

Kola – tropical hot and arid type, below 1500m altitude with mean temperature in the range 20 -28°c.

Woin Dega - sub-tropical warm, between 1500 - 2500 m altitude with mean temperature in the range 16 – 20°c.

Dega temperate highland climate above 2500 m altitude with mean temperature in the range 6 – 16°c.

1.3.3 Topography

The main topographic features at the dam site are as follows:

A river channel some 65m wide (incorporating a deeper dry season channel some 20m wide) with a bed elevation of some 1015m amsl and an average bed gradient of some 1:160.

At the narrowest cross-section, the valley widths at different elevations are as follows

Table 1. 1 Width to Height Ratio of Dam Site

Elevation(m asl)	Valley width(m)	Width: Height ratio
1015	0	-
1050	170	4.9
1100	340	4.0
1150	520	3.9
1200	680	3.7
1250	900	3.8
1300	1190	4.2
1350	>4000	>12

The ratios of width: height at this cross-section would not suit an arch dam but will suit, in terms of costs related to topography, an embankment dam or gravity dam.

The main topographic features at the powerhouse site are as follows:

- A main river channel some 100m wide with a bed elevation of some 840m a.s.l and an average bed gradient of some 1: 600.

- Wide and gently sloping banks above flood level.

A ridge extending, at slightly less than perpendicular to the river flow direction, from close to the river up to an elevation of over 1,400 m amsl. The ridge crest has several pronounced sections:

- a gentler lower section with an average gradient of 1: 10 from near river level up to 120m above river level,
- a steep middle section with an average gradient of 1: 4.5 extending up a further 200m,
- a very steep upper section with an average gradient of 1: 1.5 extending up a further 180m,
- a gentler uppermost section with an average gradient of 1: 20 extending up to elevation 1,400m a.s.l.

The topography of the area does not impose restrictions on either a surface or an underground powerhouse. The ridge extending to the river will provide good cover for underground openings, while the wide river banks above flood level suit the space requirements of a surface powerhouse or of a tailrace outfall structure. The topography of the middle and lower sections of the ridge crest would suit the requirements of a surface steel penstock.

The main topographic features at the reservoir area are as follows:

- A narrow gorge extending from the dam site with a river elevation of 1020m a.s.l in an upstream direction for some 5km to the station with a river elevation of 1050m a.s.l.
- An up to 5km wide and relatively flat river valley extending from the river section with elevation 1050m a.s.l some 17km further upstream to the station with a river elevation of 1100m a.s.l.

1.3.3.1 Impact of the Topographic Setting on Layout and Design

The impact of the topographic setting on layout and design are summarized as follows:

- At the proposed dam site, the width: height ratios of the valley cross-section are suitable for an embankment or gravity type of dam, but not an arch dam.
- The topographic setting will not limit dam heights for dams with crest elevations up to 1300m amsl.
- The valley upstream of the GD-6 dam site offers only minimal storage below elevation 1070m amsl. However, above that elevation, very large amounts of storage are available.
- The very limited storage at lower elevations means that floods during construction will not be greatly attenuated and water levels at the dam site will rise to relatively high levels during river diversion. A further impact is that immediately after reservoir impoundment begins, the reservoir water level will rise relatively quickly.
- The very large storages available above 1070m amsl mean that very high degrees of flow regulation, firm energy generation and flood attenuation will be possible

1.3.4 Geology

The rock formation at the dam site is fairly uniform consisting of post-tectonic pegmatoidal granite. It is composed mainly of coarse grained feldspar and quartz. The rock mass is relatively massive with only minor jointing. The major joint directions are vertical and horizontal. The rock mass is considered strong to very strong and watertight. In the dam area, weathering depths are generally shallow and of the order of 5m. There are a number of large boulders and blocks of rock embedded in the dam abutments which will require excavation. The slopes at and around the dam site are steep but stable and there is no unfavorable major joint planes. These slopes should remain largely stable during and after impoundment, although minor surface slides may occur along shallow surface parallel exfoliation planes.

The above conditions will support all dam types. The shallow depth of weathering and the uniformly strong base rock particularly suit the hard, strong and densest of dam namely a concrete gravity dam.

Downstream of the Dam Site

In the areas downstream of the dam, through which power waterways would be driven to the locations of prospective powerhouses, the geology becomes more complex. The rock formation is less uniform comprising basement gneisses and schist which have been intruded by pegmatoidal granite. The gneisses and schist are more jointed and, at higher elevations, joints are more likely to be open and depths of weathering higher. At depth, the rock mass will be strong and tight. Tunnels for waterways would probably pass through rock of generally good quality. However, open joints might be encountered in places and would require special treatment. Slopes around the powerhouse are gentle and stable with no unfavorable major joint planes. Slopes along the upper penstock route are steep, but again no unfavorable major joint planes exist.

Upstream of the Dam

The rock formation upstream of the dam comprises both the older basement gneisses and schist intruded by the younger granites. Limestone occurs on an old erosion surface at elevations of between 1400m and 1500m asl. There is more jointing within the gneisses and schist, but at lower elevations these joints are generally tight. The slopes in the reservoir area are gentle and stable ñ they will not become unstable after impoundment. The reservoir will be tight. Leakage through cavities in the limestone will be impossible since this limestone sits some 300 to 400m above the reservoir.

1.3.4.1 Impact of the Geological Setting on Layout and Design

There are few constraints on layout and design imposed by the geological setting. The strong foundations and shallow weathering suit both concrete gravity and embankment dams. The strong rock mass conditions impose no constraints on excavating underground openings (either caverns or tunnels). The largely uniform hard rock conditions between the dam and power outfall areas favor tunnels driven by Tunnel Boring machine. The impermeable rock underlying the reservoir area will impose no restrictions on the reservoir size or top water level.

1.4 Objective of the Study

General Objective

The main objective of this project is to summarize our overall courses for partial fulfillment of Bachelor of Science in Hydraulic and Water Resources Engineering.

Specific Objectives

- Determining the amount of available discharge in terms of small potential discharge Q_{85}, and the minimum or base flow Q_{100}
- Peak discharge determination
- Reservoir capacity determination by Elevation-Area-Storage method.
- Dam type selection and load analysis
- Assessing the environmental impact that will be at the area due to construction of dam and reserving water.

CHAPTER TWO

2. HYDROLOGICAL DATA ANALYSIS

2.1 General

Hydrology, which treats all faces of the earth water, is a subject of great importance for people and their environment. Practical applications of hydrology are found in such tasks as the design and operation of hydraulic structures. The role of hydrology is to help analyzing the problems involved in these tasks and to provide guidance for the planning and management of water resources. Different attributes of hydraulic structures are directly dependent on the peak flood magnitude adopted in the design process and the stream flow records available at the project site. Hence, stream flow records are the major data required in planning and operation of hydraulic structures.

2.2 Availability of Hydrological Data

In the design of a project, it is vital to collect or to obtain relevant data. For realistic and accurate design, it is essential that the collected data should be continuous, consistent, reliable and adequate. Hydrological data are historical in nature which is collected much before actual design. This is the reason why we are not required to collect data at this time of design. Planning and design of hydraulic structures requires adequate hydrologic information of the specific project area.

2.2.1 Adequacy

It refers primarily to length of records, but scarcity of data collecting stations is often a problem. The observed record is merely a sample of the total population of flow that has occurred and may occur again. If the sample is too small, the probabilities derived cannot be expected to be reliable. Estimation is said to be adequate, if it uses all the information about the population parameters contained in the sample.

A number of hydrological data that recorded in different gauging station with different gauging time length is provided to us to perform our design task.

Table 2. 1 Gauging Stations in Genale River basin

ID	Station name	Latitude	Longitude	Time length of record(year)	Drainage Area (sq. km)
1	Weib @ sofUmer	6:54:0	40:50:0	19	3792.7
2	Dawa @ Melka Guba	4:52:	39:19: 0	18	19611.0
3	Awatta Nr, Oddo-Shakiso	5:54: 0	38:56:0	19	1611.0
4	Mormora Megado	5:41: 0	38:48: 0	18	1375.0

5	Genale at halwen	4:26:0	41:50:0	19	54093.0
6	Genalie Chenemasa	5:31:0	39:41:0	20	10574.0
7	Mana nr. Goro	7:01:0	40:22:0	19	485
8	Shawe@Mrslo Project	6:26: 0	39:41:0	19	340
9	Yadot Nr Dello Mena	6:25: 0	39:51:0	19	531
10	Dimtu nr.Bore	7:10: 0	39:50:0	28	28.6
11	Togona@Shallo Village	7: 0: 0	39:58:0	19	336.2
12	Weib nr. Denbel	7: 2: 0	40:48:0	30	1215
13	Weib at Alemkerem	6:59: 0	40:58: 0	20	3576.9
14	Shaya nr.Robe	7:10: 0	39:58: 0	19	433.8
15	Tegona @ Goba	7: 0: 0	39:59: 0	19	83.1
16	Welmel @ Melka Amana	6:14: 0	39:46: 0	19	1048.0
17	Tebel nr. Guinir	7:11: 0	40:44: 0	33	79.0
18	GD-6 Dam Site	5.68:0	40.93:0		13,350

But we have to select from those all gauging station data that which is recorded on the main river and is nearest to the GD-6 dam site. To do this selection we used GIS software with Ethio river data. From this system we got that the hydrological data recorded on Genale at Chinemasa station is better than the other station data as shown in the following map. Chinemasa station is located at Latitude: 5:31:0N Longitude: 39:41: 0 E and covers Area of: 10574.0 km^2. For large hydropower project, a minimum of 30years hydrological data is mandatory. But Genale at Chinemasa gauging station has only 20 years recorded data; so data extension is required and it is presented in detail under the topic of “Extension of Recorded Flow Data”

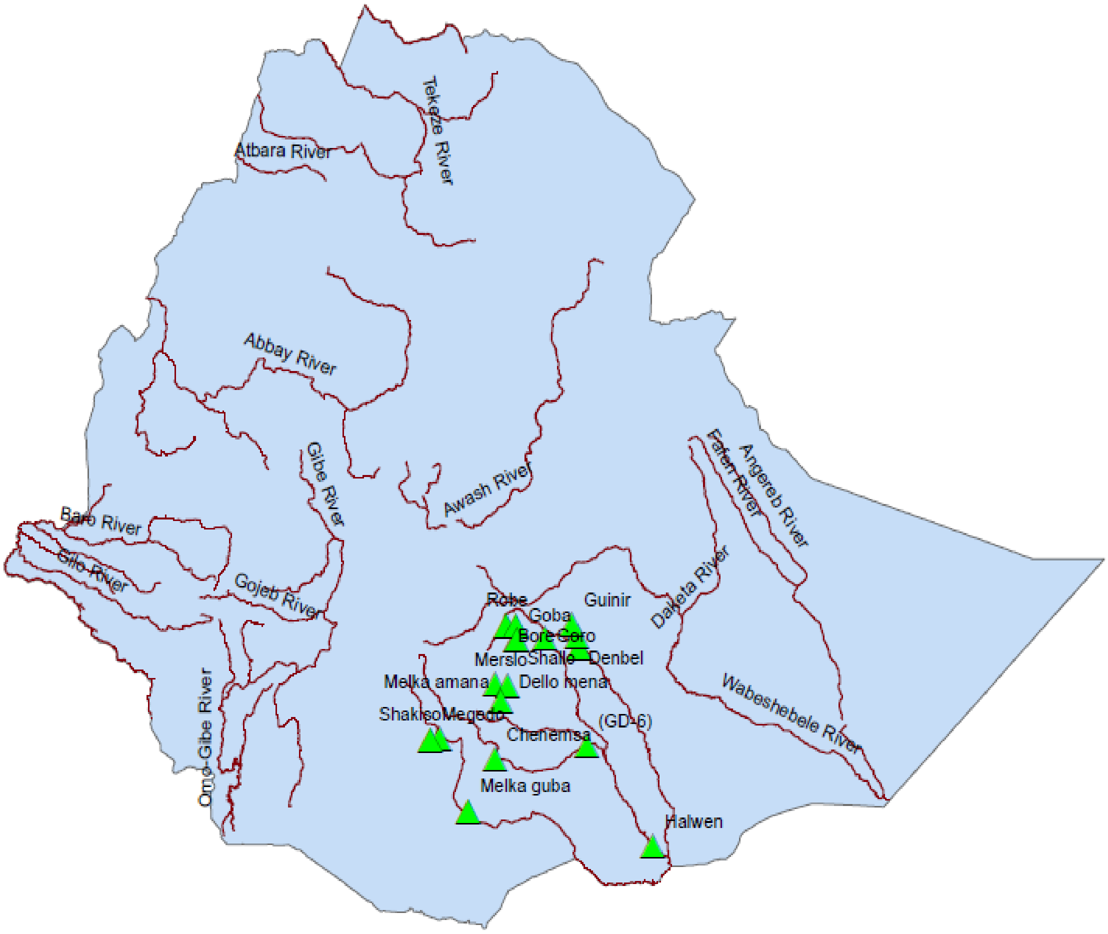

Figure 2. 1 Gauging Station Distribution on Ethiopian River

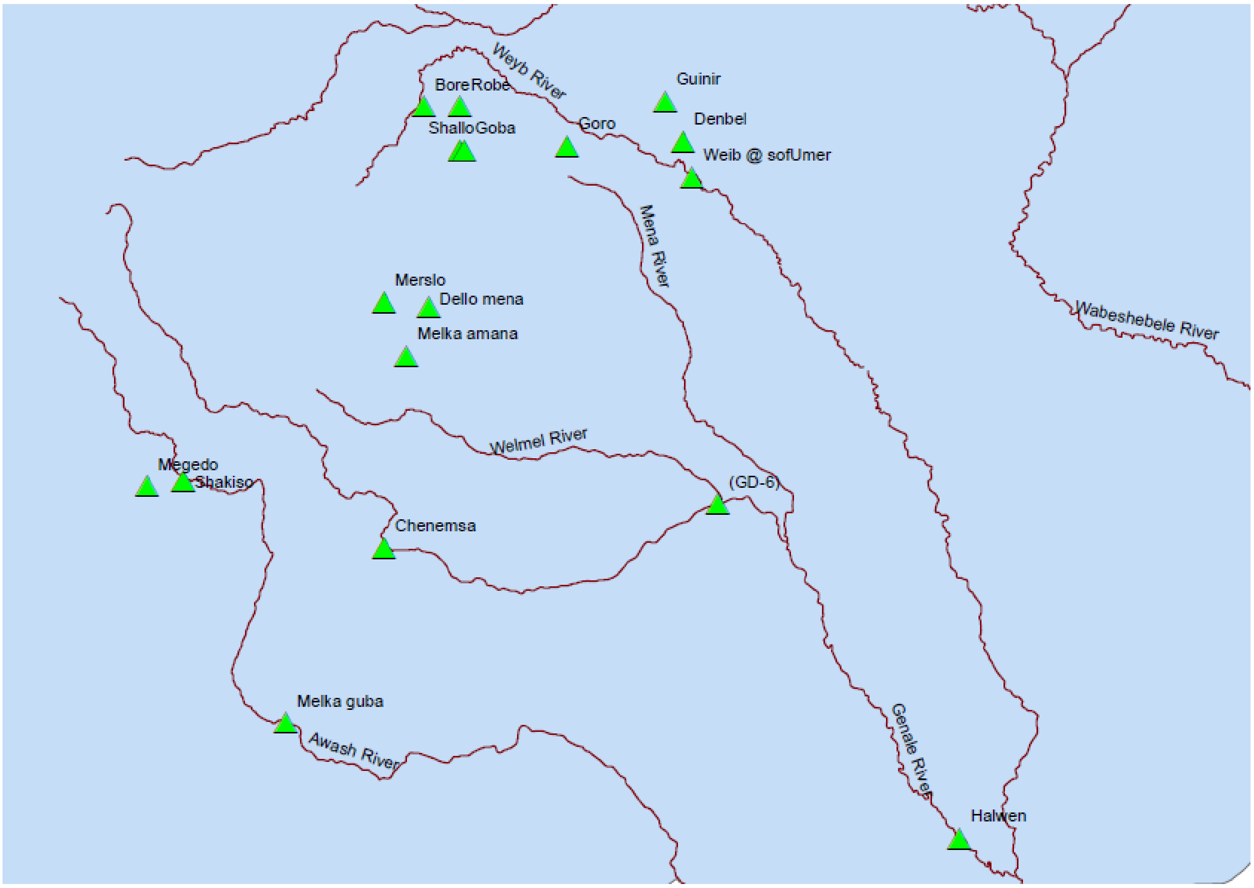

Figure 2. 2 Gauging Station Distribution of Genale river

2.2.2 Accuracy

This refers primarily to the problem of homogeneity. Most flow records are satisfactory in terms of intrinsic accuracy, and if they are not, there is little that can be done with them. If the reported flows are unreliable they are not a satisfactory basis for frequency analysis. Even though reported flows are accurate, they may be unsuitable for probability analysis if change in the catchments have caused a change in the hydrologic characteristic i.e. if the record is not internally homogenous.

2.2.3 Continuity

The continuity of a record may be broken with missing data due to many reasons such as damage or fault in recording gauges during a period. Unfortunately, the given 20 years of stream flow record of Genale Dawa River gauging station at Chinemasa was found with missing data and also it needs an extension.

2.3.4 Consistency

If the conditions relevant to the recording of a stream gauge station have undergone a significant change during the period of record, inconsistency would arise in the stream data of that station. This inconsistency can be differentiated from the time the significant change took place.

Some of the common causes for inconsistency of the records are:

- Shifting of rain gauge station to a new location
- The neighboring hood of the station under gone a marked change
- Change in the ecosystem due to calamities, such as forest fires, land slide and
- Occurrence of observational error from a certain date.

Inconsistence of the records can be checked by using different techniques like, Grubbs and Beck (1972) (G-B) test and others can be used. The detail test is presented under Outliers Test topic by using G-B test.

2.3 Estimation of Missing Data

Before using storm record of a station, it is necessary first to check the data for continuity. The continuity of record may be broken with missing data due to many reasons such as;

- Absence of the observer
- Broken or frailer of instrument
- War (political instability)

It is often necessary to estimate these missing records. The missing data can be estimated by using the data of the neighboring station. There are different methods of filling missing data such as arithmetic average method, normal ratio method or other appropriations method.

2.3.1 Arithmetic Mean Method

This method is used when:

- The normal annual flow of the missing station say x is within 10% of the normal annual storm of the surrounding stations,

- Data of at least five surrounding stations, called index stations, are available within the basin,
- The index stations should be evenly spaced around the missing station and should be as close as possible,
- The missing storm data of station *x* is computed by simple arithmetic average of the index stations in the form

$$Q_x = \frac{1}{n}(Q_1 + Q_2 + \ldots Q_n)$$

In which Q_1, Q_2. . .Q_n are the discharge of index stations and Qx is that of the missing station, n is the number of index stations. The word normal means average of N years of data, i.e., N values of the latest records. For example, for a station when the last N years of June month discharge are averaged, we call it as normal discharge for the month of June for that station.

2.3.2 Normal Ratio Method

This method is used when the normal annual discharge of the index stations differ by more than 10% of the missing station. The storm of the surrounding index stations are weighed by the ratio of normal annual discharge by using the following equation.

$$Q_x = \frac{N_x}{n}\left[\frac{Q_1}{N_1} + \frac{Q_2}{N_2} + \ldots \frac{Q_n}{N_n}\right]$$

where Q_1, Q_2, . . ., Q_n are the discharge data of index stations, N_1, N_2, . . ., N_n the normal annual discharge of index stations, Qx and N_x the corresponding values for the missing station x in question and n is the number of stations surrounding the station x. Comparing the above equation with the general equation, the weight coefficient a_i for the i^{th} station is given as;

$$a_i = \sum N_x / nN_i$$

2.3.3 Linear Regression

For this particular project a better way is to fit linear regression line of stream and

Arithmetic Mean method. Since there are a number of stations it is best method to estimate missing data by using data from D/S and U/S station. This method considers all surrounding data and does not need more criteria other than correlation so it estimates more accurate data. If the correlation coefficients is in the range $0.6<R<1.0$ indicates good correlation (Ray K.Linsley.R)

The equation for linear regression is:

$$Y = mx + b$$

And the values of the coefficients *m* and *b* are given by:

$$m = \frac{N\sum_{i=1}^{N} XY - \sum_{i=1}^{N} x * \sum_{i=1}^{N} Y}{N\sum_{i=1}^{N} X2 - (\sum_{i=1}^{N})2}$$

$$b = \frac{\sum_{i=1}^{N} y \sum_{i=1}^{N} x2 - \sum_{i=1}^{N} x \sum_{i=1}^{N} y}{N\sum_{i=1}^{N} X2 - (\sum_{i=1}^{N} X)2}$$

$$R = \frac{N\sum_{i=1}^{N} XY - \sum_{i=1}^{N} X \sum_{i=1}^{N} y}{\sqrt{((N\sum_{i=1}^{N} x2 - (\sum_{i=1}^{N} x)2(N\sum_{i=}^{N} y2 - (\sum_{i=1}^{N} y)2))}}$$

Where, a and b are constant

Y = monthly average stream to be filled

X = monthly average stream available

R = correlation coefficient

2.3.4 Extensions of Flow Records

In many cases, a stream gauge may be of short period records (y_i, i =1, 2, 3..., n) in one site and there may be a very long record in another gauging station (x_i, i=1, 2, 3--, N).

It will be assumed that n<N and those records for both stations exists for the period i=1, 2, 3... n.

Thus the correlation between x and y may be exploited to extend the length of the y-records by estimating a set of n-N flows for the period during which there was an x records but no y records. The obvious approach to this problem is to use regression analysis to estimate y as a function of x. However; it is not particular values of y_i that is important, but rather the full collection of estimates of y, these properties include the mean and variance of y and it may also contain the correct statically properties of y.

The flow records at Chinemasa station have been observed for 20years, but flow records at Weib near Denbel which we selected from stations have 30years. Thus we need to exploit the correlation between the short record y and the long record x to extend the length of y by estimating a set of 10 years flow records for the period during which there was an x record but no y record for the above noted station. The variance of the regression estimates of y_i, i=1,2,3---30 is biased down ward because regression estimates lies on the regression line and the actual data are scattered about the regression line and are thus more viable.

A better approach to this problem is a class of methods known as maintenance of variance extension (move). The principle of move technique is to drive a set of coefficients for the model that transform the x data to estimates of y in such a way that the expected values of statistics of the estimated y values are equal to the true but unknown values of the statistical properties of y. Simplest **Version Move 1**, presents the variance and mean of y using the equation

$$Y_i = a + b.x_i,$$

Where:

$$B = \sqrt{\frac{S_{YY}}{S_{XX}} * \sin r}$$

$$A = Y - BX$$

$$S_{XX} = \sum_{I=1}^{N} (X - X)^2$$

$$S_{YY} = \sum_{I=1}^{N} (Y - Y)^2$$

Where r =the correlation coefficient

Sample calculation:

Table 2. 2 Record Extension Calculation

Mont	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec

h											
Xm	1.69	13.57	1.07	1.37	4.40	1.50	5.00	10.42	3.89	6.85	5.85	1.28
Ym	68.6	131.8	85.5	88.4	245.6	89.9	76.3	133.0	118.8	211.9	121.1	51.2
Sxx	267	9832	231	36	750	16	501	1705	120	232	196	80
Syy	1406	83912	8461	6085	310195	12580	16873	71702	33390	285158	139422	2589
A	66.2	103.3	79.9	72.8	208.6	57.1	54.6	120.1	91.0	97.8	55.6	48.6
B	1.41	2.10	5.22	11.39	8.41	21.98	4.33	1.24	7.15	16.67	11.20	2.03
R	0.39	0.54	0.84	0.87	0.17	0.68	0.59	0.04	0.19	0.23	0.18	0.13

By using this method extension is performed for the selected station Chenemasa from 20 year to 30year by using the station Weib near Denbel which has 30years record. The extension is done in both forward and backward way (2012-2015 and 1993-1986) respectively.

2.3.5 Transferring of Stream Flow Data to the Proposed Site

All too often the stream flow data that are available from measured gauging stations are not from locations for which a hydropower site analysis is to be made. Methods are required to develop extrapolation of measured flow duration data, which will be representative of a given site on a stream. There are several methods to estimate flows from ungagged catchments. Regional frequency analysis, sequential flow analysis and use of parametric flow duration curve are some of them.

The catchments of G-D-6 and Genale Chinemasa gauging station have different features such as density and types of natural vegetation cover, soil type, land use, rain fall distribution, evaporation rate, slope, **catchments size** and rain fall amounts. Therefore the transposition of flow record from the gauging station is worked out by the ratio of the drainage area.

Mathematically;-

$$Qu = Wf * Qg$$

Where Q_u – parameter of ungauged site

Q_g – parameter of gauged site

W_f – weighted factor $=A_u/A_g$

A_u – area of ungauged site

A_g – area of gauged site

A_u = 13,350 km^2

A_g = 10574.0 km^2

W_f = 13,350 km^2/10574.0 km^2 = 1.263

Qu = 1.263 *Qg

Check for the transposed data

In order to check the reliability of the transposed data's at the dam site, the hydrograph for both flows of each stations was drawn and the pattern of flow versus time is compared for the same period of time with their respective flow values as shown in fig below.

Table 2. 3 Values for Checking of the Transposed Data

Time		Chinemasa station	G-D dam site
1986	Jan	61.826	78.057
1986	Feb	62.278	78.628
1986	Mar	64.032	80.842
1986	Apr	88.660	111.937
1986	May	133.708	168.810
1986	Jun	67.692	85.463
1986	Jul	80.797	102.008
1986	Aug	156.777	197.936
1986	Sep	92.904	117.294
1986	Oct	82.571	104.248
1986	Nov	67.213	84.859

1986	Dec	62.969	79.500

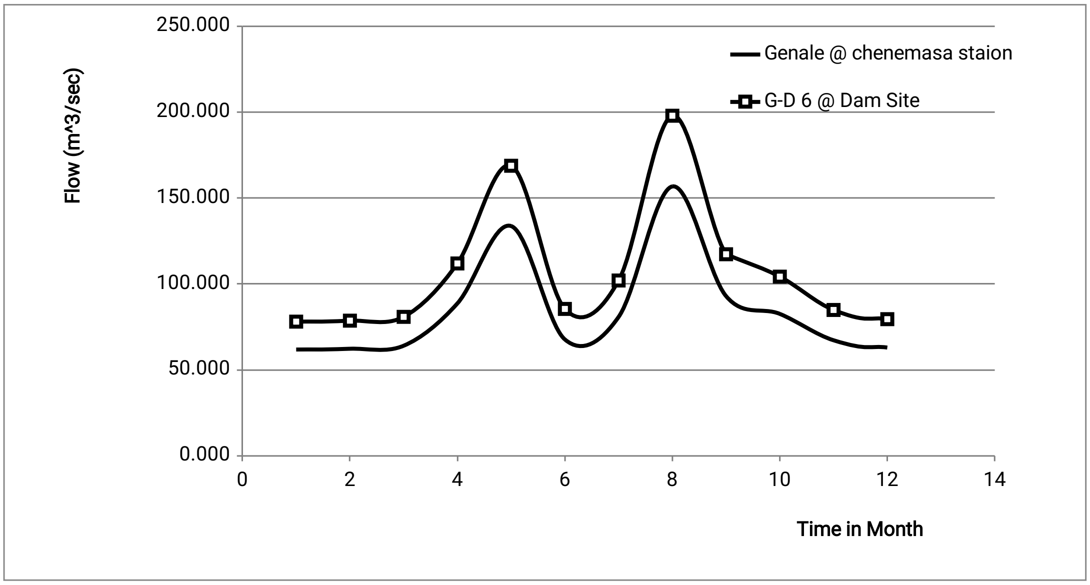

Figure 2.3 Checking of the Transposed Data

Since the pattern of flow before transposing and after transposition is the same, therefore the transposed data's are reliable.

2.4 Hydrological Data Test Methods

2.4.1 Outliers Test

Outliers are data points that depart significantly from the trend of the remaining data. According to the water resources council of US (1981), if a station skewness is greater than +0.4, then test for high outliers are considered first, if station skewness is less than -0.4, test for low outliers are considered first. Where the stations skewness is between ±0.4, test for both high and low outliers shall be applied before eliminating any outliers from the data set. Thus, both the high and low outlier threshold values should be determined in order to omit the respective outliers using the equation given below. But here for better approximation we evaluated both higher outlier and lower outlier test.

Higher outlier, $Y_{H/L} = Y_m \pm K_n * S_y$

Where: $Y_{H/L}$ are the high or low outliers' threshold in log units.

Y_m = mean of data in log unity

K_n = Coefficient from table for sample size N

$$S_y = \sqrt{\frac{\sum (Y - Y_m)^2}{N - 1}}$$

S_y=standard deviation

Table 2. 4 Data for Outlier Test

Year	Qmax	Dec. Qmax (X)	(X-Xmean)^2	Log X=Y	(Y-Ymean)^2
1986	197.94	579.809	81741.043	2.763	0.11893
1987	225.54	503.408	43891.456	2.702	0.08037
1988	266.57	449.634	24251.350	2.653	0.05496
1989	251.02	405.200	12386.585	2.608	0.03581
1990	229.94	402.242	11736.869	2.604	0.03462
1991	290.96	390.243	9280.902	2.591	0.02990
1992	390.24	351.212	3284.027	2.546	0.01616
1993	267.56	341.648	2279.403	2.534	0.01326
1994	316.19	338.787	2014.361	2.530	0.01243

1995	286.04	320.324	697.972	2.506	0.00760
1996	351.21	318.702	614.877	2.503	0.00722
1997	579.81	316.188	496.535	2.500	0.00665
1998	503.41	300.033	37.547	2.477	0.00345
1999	318.70	293.445	0.211	2.468	0.00241
2000	338.79	290.956	8.698	2.464	0.00206
2001	248.18	286.039	61.876	2.456	0.00144
2002	136.18	267.563	693.931	2.427	0.00008
2003	145.87	267.563	693.931	2.427	0.00008
2004	221.48	266.566	747.414	2.426	0.00005
2005	341.65	263.499	924.546	2.421	0.00001
2006	300.03	251.022	1839.005	2.400	0.00035
2007	320.32	248.175	2091.240	2.395	0.00056
2008	267.56	229.942	4091.244	2.362	0.00323
2009	293.45	225.537	4674.277	2.353	0.00425
2010	449.63	221.478	5245.777	2.345	0.00534
2011	402.24	197.936	9210.154	2.297	0.01486
2012	16.42	191.536	10479.422	2.282	0.01854
2013	263.50	145.869	21914.608	2.164	0.06475
2014	405.20	136.180	24877.196	2.134	0.08083
2015	191.54	16.421	76997.487	1.215	1.44727
mean		293.905		2.418	
SUM			357263.946		2.067

Y_{mean} = 2.418 (from above table)

K_n = 2.563(from table w.r.t. N, see on ANNEXES)

S_y = (2.067483197/(30-1))^0.5 = 0.267007

Then the higher and lower threshold value becomes:

Y_h = (2.418+2.563*0.267007) = 3.102765

Q_H = antilog (Yh) = 1266.966m^3/sec.

Y_L = 2.418-(2.563*0.267007) = 1.73409

Q_L = antilog (YL) = 54.21m^3/sec.

In general, sample values greater than Q_H are considered to be high outliers, while those less than Q_L are considered to be low outliers. For Genale River data record all the observations are lower than Q_H. But unfortunately there are data values which are less than Q_L values. Therefore those data values lower than Q_L should be replaced by Q_L value for further frequency analysis.

Consistency

The Grubbs and Beck (1972) (G-B) test is used to detect outliers, and hence used for checking the consistency of records of data.

$$\sigma e = (Se/Xm)(100) \; < \; =10\%$$

$$Se = Sx/\sqrt{(N)}$$

$$Sx = \sqrt{(\sum(X - Xm)^2/N - 1)}$$

Where, σ_e= relative standard error

S_x = standard deviation of monthly maximum flow

X_m = mean value monthly maximum flow

N = number of year = 30

S_x = 110.9930016

Se = 20.2644569

Xmean = 293.9053

σe =(20.2644569/293.9053)*100 =6.894894%

6.894894% < 10%

Therefore the data is reliable and consistent.

2.4.2 Flow Duration Curve

A useful way of treating the time variability of water discharge data in hydropower studies is by utilizing flow duration curves. It is a plot of flow verses the percent of time a particular flow can be expected to be exceeded. A flow duration curve merely reorders the flows in order of magnitude instead of the time ordering of flows verses time plot.

Flow duration curves find considerable use in water resources planning and development activities. Some of the important uses are:

1. In evaluating various dependable flows in the planning of water resources engineering projects.
2. In evaluating the characteristics of the hydropower potential of a river

There are two methods to construct flow duration curve;

- Calendar- year basis method
- Total period basis method

In total period method, the entire available record is used for drawing the flow duration curve. These are first tabulated in the ascending order starting from the driest month period and ending with the wettest month of the 30 years of duration or vice versa. The resulting flow duration curve would then be drawn with the help of the whole records values. The total period method gives more accurate results than extreme events.

In calendar year method, each year's average monthly flow is first arranged in ascending order. Then the average flow values corresponding to the driest month, second driest month and so on up to the wettest months are found out by taking arithmetic mean of all values of the same rank. These average values are then used for plotting flow duration curve.

In this specific project we used calendar year method to draw flow duration curve. This is due to that the total year method considers each daily data which is very long when arranged in single column and which takes large number of pages to show it in annexes. Inversely that of calendar year method is short and simple.

Using Weibul formula i.e., $Pp = (m / (N + 1)) * 100\%$

Where; m = is the order of the discharge

P_p= percentage probability of the flow magnitude being equaled or exceeded

N= number of data points that are used in the listing

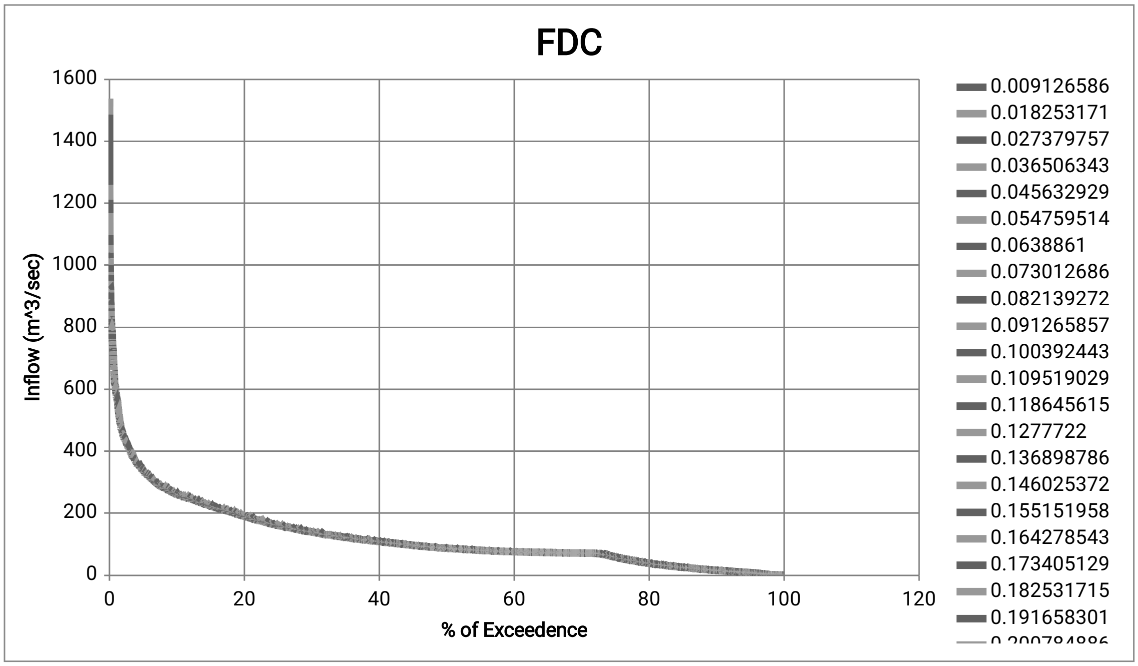

figure 2. 4 Flow Duration Curve

From the flow duration curve the conventional discharges that are obtained for some percentage of exceedence are Q_{100}, Q_{75}, and Q_{50}.

Where;.

- Q_{95}- small potential power computed from the flow available for 95% of the time
- Q_{75}- potential power is computed from the flow available for 75% of time
- Q_{50}- median potential power is computed from the flow available for 50% of time

Table 2. 5 Discharges for Specific Pp

Pp (%Exceedance)	Discharge (m^3/sec.)
95(Firm discharge)	2.5
75	58
50	86

2.5 Flood Estimation

2.5.1 General

For planning and designing of water resource development project the important parameter are

river discharges and related questions on the frequency and duration of normal flow (e.g. for

Hydro power production or for water availability) and extreme flows (for floods and droughts).

At a given location in the stream, flood peaks enable one to assign a frequency to a given flood

peak value. In the design of all hydraulic structures, the peak flows that can be expected with an assigned frequency are of primary importance to adequately proportion the structures to

accommodate its effect.

To estimate the magnitude of a flood peak the following alternative method are available:

- Rational Method
- Empirical method
- Unit hydrograph technique and
- Flood frequency studies.

Rational method is applicable to a small site (<50km^2) catchments but the G-D hydropower project has an approximate area of 13,350 km^2.

Regarding the **unit hydro graph method,** it is normally restricted to moderate size catchments with areas less than5000km^2. This method also does not fit to our data because the catchment area of G-D exceeds 5000km^2. It is also not used due to shortage of available data like rain fall resulting floods and infiltration.

2.5.2 Selection of Return Period

Return period (T) is the average interval in years between events when equal or exceed a given magnitude .It may ,however, be clearly understood that the concept of return period does not imply that the event of any given magnitude will occur at constant or even approximately constant interval of years.

Selecting higher return period means the corresponding flood magnitude is also very high. Such a very high flood may never occur during the life time of the structure on the other hand, if a very low discharge corresponding to lower return period is chosen for the design and exceeded it will result in the failure of the structure causing more damage than would have been caused in the absence of the structure .

Table 2. 6 General Guide Line for Selecting the Return Period

Types of structure	Return period
Spillways for project with storage more than 60Mm3	1000
Barrage and minor dams with storage less than 60Mm3	100
Spillway of small reservoir dam in considering not endangering urban residences	10-20
Diversion weir	50-100

Source: Subramanya (1989) and Nevec (1972)

In our case we expect the total storage greater than 60Mm3; therefore we have taken the return period as 1000 years.

2.5.3 Empirical Formula Method

But the calculated value by using General Extreme method is minimum to be as a design flood, it is tried to make the flow to be an instantaneous flow i.e. changing of the monthly flow in to a daily or hourly flow, so that the exact maximum flood value can be predicted. Therefore, for this particular project the modified version of GEV By DR. Admassu is used for the determination of the maximum design flood. Which is used for the Blue Nile basin. As to his classification GD-6 hydropower project is found in region 2 therefore for this particular project.

$$Q_p = Qav\,(1 + KCv)\ ; \qquad Q_{av} = 2.44A^{0.61}$$

$$K = -\frac{\sqrt{6}}{\pi} * (\gamma + \ln(\ln\left(\frac{T}{T-1}\right)$$

Where:

Q_p=Maximum peak flood at a given return period T(m3/s)

Q_{av}=mean annual flood (m3/s)

A=catchment area (Km2)= 13350Km^2

CV=average regional coefficient**=** 0.25

γ = Culler's constant=0.57721

T=1000

Q_{av}=2.44(13350)^0.61 = 801.555m^3/sec

K = 4.9355

Q_p = 801.555(1+4.9355*0.25) = **1790.574m^3/sec**. = 1800m^3/sec.

2.5.4 Flood Frequency Analysis

Frequency analysis is hydrologic term used to describe the probability of occurrence of particular hydrologic event (e.g. rainfall, flood, drought, etc). Water resource systems must be planed for future events for which no exact time of occurrence can be forecasted. Hence, the hydrologist must give a statement of the probability of the stream flows will equal or exceed a specified value. These probabilities are important to the economic and especial evaluation of a project.

For major projects, the failure of which seriously threatens human life, a more extreme events, the probable maximum flood, has become the standard for designing the spillway

Flood frequency analysis is determination of the frequency occurrence of extreme flood in the life of the project. This is important in water resource planning & management for economic and efficient design of structures. However, flood is essentially a random phenomenon so it is difficult to predict the exact maximum flood which can occur in the future. Towards these, specific extreme value distributions are assumed and the required statistical parameters calculated from the available data. Using these flood magnitude for a specified return period is estimated. *(Source: Chow, V.T (1988), Applied Hydrology.Mc Graw-Hill Book Company, Singapore.*

2.5.5 Selection of Probability Distribution

The observed frequency distribution which also called as empirical frequency distribution is assumed to follow any of the theoretical distribution. Some of the commonly used frequency distribution functions for prediction of extreme flood value are:

1. Gamble's extreme value distribution
2. Exponential distribution
3. Log-Normal distribution
4. Generalized Extreme Value distribution etc.
5. Pearson type-III distribution
6. Normal distribution
7. Uniform distribution

There are many simplified available methods to select and evaluate the parent distribution. From those the most commonly used L-Moment, method is employed. This is the recent method and that gives efficient result as compared with the others. That is why we choose this method for this project.

Estimation of L-Moment Method

L-Moments are linear functions of probability weighted moments. They are more convenient than the probability weighted moments because they can be interpreted as measures of scale and shape of probability weighted moments which are defined as:

$$br \ = E[X\{F(X)\}^r]$$

Where, F(X) is the cumulative density function for X.
The unbiased estimator:

$$\beta o = Xm = \text{Average} = \mathbf{129.141}$$

$$\beta 1 = \frac{\Sigma(n - j)(xj))}{n(n-1)} = \mathbf{72.3110569}$$

$$\beta 2 = \frac{\Sigma (n - j)\ (n - j - 1)\ (Xj)}{n\ (n - 1)\ (n - 2)} = \mathbf{50.45914251}$$

$$\beta 3 = \frac{\Sigma (n - j)\ (n - j - 1)\ (n - j - 2)\ (Xj)}{(n - 1)\ (n - 2)\ (n - 3)} = \mathbf{38.83786069}$$

All the above computation is shown on annex -2
The L-moments are easily calculated as follows:

$$\lambda 1 = \beta o = \mathbf{129.141}$$

$\lambda 2 = 2\ \beta 1 - \beta o = 15.48077621$

$\lambda 3 = 6\ \beta 2 - 6\ \beta 1 + \beta o = -1.970148853$

$\lambda 4 = 20\ \beta 3 - 30\ \beta 2 + 12\ \beta 1 - \beta 0 = 1.574284092$

The L-moment ratio, which are analogous to conventional moment ratios are defined by:

$Z = \lambda 2 / \lambda 1 = 0.119874991$

$Z3 = \lambda 3 / \lambda 2 = -0.127264216$

$Z4 = \lambda 4 / \lambda 3 = -0.799068603$

Where, λ1 is a measure of location
Z is a measure of scale and dispersion,
Z3 is a measure of skewness, and
Z4 is a measure of kurtosis.

Sample of L-moment ratio are calculated as follows.

Using relationship between Z3 and Z4 for different distribution, the values of Z4 are calculated for the sample Z3 in L- moment ratio diagram.

1. **Uniform**: Z3 =0, Z4=0.
2. **Exponential**: Z3 =1/3, Z4 =1/6
3. **Gamble (EV1**): Z3 =0.1699, Z4 =0.1504
4. **Normal:** Z3 =0, Z4 =0.1226
5. **Generalized Extreme Value(GEV):**
   $Z4 = 0.1070 + 0.1109 (Z3)^2 - 0.0669 (Z3)^3 + 0.60567 (Z3)^4 - 0.04208 (Z3)^5 + 0.03763 (Z3)^6$
6. **Pearson-III:**
   $Z4 = 0.1224 + 0.30115 (Z3)^2 + 0.95812 (Z3)^4 - 0.57488 (Z3)^6 + 0.19383 (Z3)^8$
7. **Log Normal:**
   $Z4 = 0.12282 + 0.77578 (Z3)^2 + 0.12279 (Z3)^4 - 0.13638 (Z3)^6 + 0.113638 (Z3)^8$

Table 2. 7 Computed Values of Kurtosis for Different Distribution

Assumed	GEV	Pearson III	Log normal
Z3	Z4	Z4	Z4
0.1	0.108103	0.1255067	0.1307
0.2	0.111867	0.1359427	0.154825
0.3	0.120051	0.1568579	0.195864

0.4	0.135801	0.1928842	0.254319
0.5	0.163576	0.2493446	0.330427
0.6	0.209043	0.3314204	0.424169
0.7	0.278931	0.443548	0.535575
0.8	0.380867	0.5893999	0.665502
0.9	0.523164	0.7728767	0.817155
1	0.71459	1.00062	0.998648

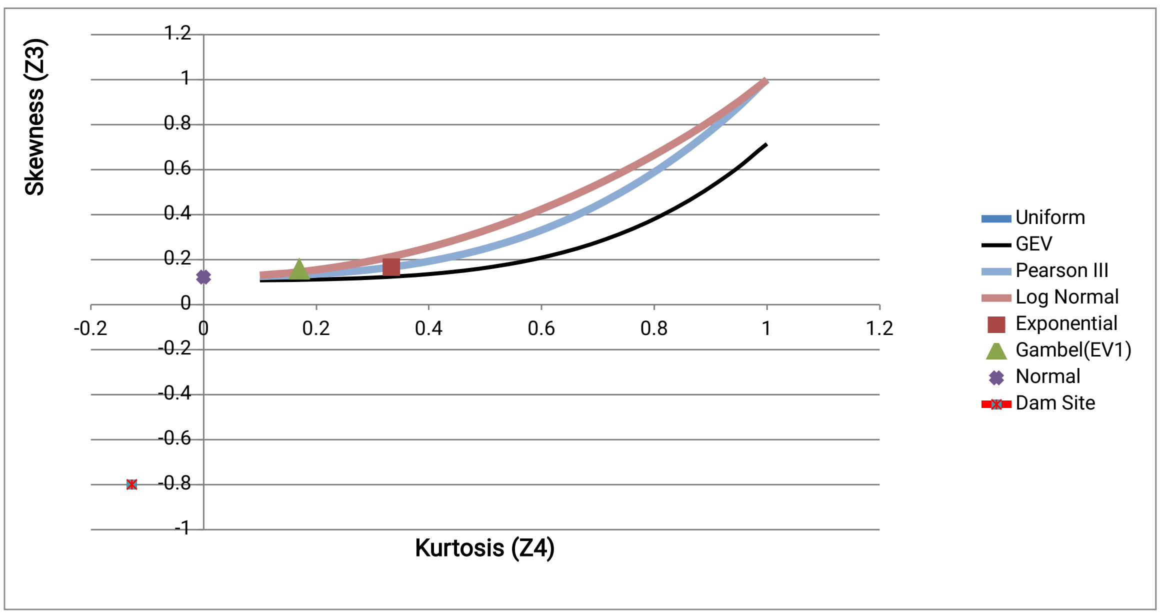

figure 2. 5 Coefficient of Skewness Vs Kurtosis

As we can see from the above Skewness Vs Kurtosis diagram, the graph of General Extreme Value more approaches to Dam site graph. Then the best probable parameter distribution for our 30 years stream flow data is the GEV method.

The flood discharge value using the general equation of General Extreme Value distribution is computed:

$$X_T = \xi + \alpha/k\,[1 - (-\ln (1 - 1/T))\text{^}\,K]$$

Where:

ξ = location parameter
α = scale parameter
k= shape parameter

$$\alpha = k\,\lambda 2 / [(1 + K)\,(1 - 2^{\wedge} - K)] = 17.46806591$$

$$\xi = \lambda 1 + \alpha/k\,[(1 + K) - 1] = 146.6090659$$

$$k = 7.859C + 2.9554C^{\wedge}2 = 0.525553344$$

$$C = \frac{2\lambda_2}{\lambda 3 + 3\lambda 2} - \frac{\ln 2}{\ln 3} = 0.065270715$$

Where: λ_1, λ_2, λ_3 are from L-moment

X_T = **178.9653m^3/sec**

Conclusions: comparing the two methods, the maximum probable flood is selected as the largest of all values. And this flood is obtained by modified General Extreme Value formula by Dr. Admasu for a return period of 1000 years .thus the peak flood Q_q is **:**

Q_p = 1800m^3/sec.

Since there are a lot of community at downstream that uses this river for different purpose like small scale irrigation, livestock and others and also obey water law we decided to release 20% of peak discharge.

Q_p = 1800 - 20%(1800m^3/sec) = **1440m^3/sec**.

2.6 Risk and Reliability

The design of a hydraulic structure always faces a nagging doubt about the risk of failure of structures. This is because of the estimation of the hydrologic design values such as design flood involves a natural or in built uncertainty and as such a hydraulic risk of failure.

Risk (Ř): is the probability of occurrence of an event ($X \geq X_T$) at least once over a period of n years where n is the useful life of the reservoir.

Reliability (Re): is the probability of non-occurrence of an event (X ≥ Xt)

Thus the risk is given by;

$$R = 1 - (1 - P)^n = 1 - \left(1 - \frac{1}{T}\right)^n$$

Where P =probability = 1/T

T=return period

n = service life of project

The reliability Re is defined as

$Re=1-R= (1-1/T)^{n}$

Service life of the project

Return period	$R = 1 - \left(1 - \frac{1}{T}\right)^{n}$ (%)		Remark
	n = 50	n = 100	
1000	4.87943718	9.520785	ok for n=50

The risk occurrence with in 1000years return period flood for 50years' service time of the project is only 4.88% therefore 50 year service time is selected for risk analysis.

$Ř= 1-(1-1/T)^{n}$

$= (1-(1-1/1000)^{50})*100\% \quad = 4.88\%$

$Re = 1- Ř = 1- 4.88\% = 95.12\%$

Conclusions: Since the value of occurrence risk is very less and the value of non-occurrence of probable flood is maximum, the probability of the dam to be failed is very low. Thus it is more reliable

CHAPTER THREE

3. RESERVOIR PLANING

3.1 General

Reservoir is a large, artificial lake created by constructing a dam across a river.

Broadly speaking, any water pool or a lake may determine a reservoir. However, the term reservoir in water resource engineering is used in a restricted sense for a comparatively large body of water stored on the upstream of a dam constructed for this purpose.

This is true, especially in Ethiopia, in which most of the rain fall occurs during the

"kiremt" (wet) season from June to October. Most of the rivers carry very little water during "bega" (dry season). During low flow periods, it is not possible to meet water demand for ultimate requirement of power production. To regulate the water supplies, a reservoir is created on the river to store water during the rainy season.

The primarily purpose of a reservoir is to regulate the flow so that the firm supply can be generated, and the principal gain to offset the cost of increased storage is in the increased proportion of firm output and the increased kilo-watt revenue which are those made available. The storage is often provided at the intersection dam, and in such cases an increase in the dam height produces an increase in head and in mean output. This gain must be allowed for in the economic analyses. If the increase is a significant proportion of the total head it may permit an increase in the installed capacity and in the kilo-watt revenue. The planning is based on the assumption that the reservoir area is geologically sound and water tight. Reservoirs justify there highest for direct benefits like water supply, irrigation and hydroelectric power generation.

Depending on the function served reservoirs may be classified as:

- Single purpose-a reservoir which is used for single purpose.
- Conservation reservoir
- Flood control reservoir
- Multipurpose reservoir-they are constructed to serve more than one purpose. For example irrigation, water power generation, domestic and industrial water supply.

3.2 Selection of Site for Reservoir

While selecting the site for reservoir, the following characteristics points should be taken into account;

a) Geological factors
b) Topographical factors
c) Land acquisition factors
d) Economic considerations

A. Geological Factors

- Geology of the catchment area:

The soil characteristic of the whole catchment areas which contribute water to the reservoir should be studied so that the losses over the catchments should be minimum and the silt deposition will be reduced.

- Geology of the dam site

The whole length of the dam should be founded on a sound water tight rock foundation so that the percolation below the dam is minimized.

B. Topographic factors

- The reservoir basin should have narrow opening in the valley. It will reduce the length of the dam to be constructed and consequently the cost of the project will be reduced.
- The side slope should be steep through the basin .It will reduce the surface area per unit volume so that undesirable shallow water depth and surface evaporation

may be reduced.

C. **Land acquisition factor**

Since the reservoir basin may cover large area, it is very essential to see that the land going to be submerged is not valuable.

D. **Economic consideration**

- The dam to be constructed should not be of great height. If the height of the dam is more, the cost of the project will increase.
- The materials for the construction of the dam should be locally available.
- The reservoir site should be close to powerhouse, so it reduces the cost of conveyance system.

General Reservoir site selection criteria

It is virtually impossible to locate a reservoir site having completely ideal characteristics. General rules for choice of reservoir sited are:

1) A suitable dam site must exist; the cost of dam is often a controlling factor in selection of a site.
2) The cost of real estate for the reservoir (including road, railroad, geometry, and dimensioning relocation) must not be excessive.
3) The reservoir site must have adequate capacity.
4) A deep reservoir is preferable than a shallow one because of lower land costs per unit of capacity, less evaporation loss, and less likelihood of weed growth.
5) Tributary areas, which are usually productive of sediment, should be avoided if possible.
6) The quality of the stored water must be satisfactory for its intended use.
7) The reservoir banks and adjacent hill slopes should be stable. Unstable banks will contribute large amounts of soil material to the reservoir.

In this particular project the major characteristics which mostly influence the selection of reservoir site is the geologic condition and overall cost of project (Dam, Power House, Intake,

Water conveyance system, Transmit ion system…)

3.3 Physical Characteristics Of Reservoirs

As the primary function of reservoirs is to provide storage, their most important physical characteristics are storage capacity. The capacity of a reservoir of irregular shape can be compared with the formulas for the volumes of solids, but the capacity of reservoirs on natural sites must usually be determined from topographic surveys.

3.3.1 Elevation- Area- Capacity Curve

The objective of an elevation area capacity curve is to obtain the capacity of reservoirs at different elevation with respect to submergence area. This curve utilizes an input data from topographic survey on natural sites for its construction .An area elevation curve is constructed by plan metering the map enclosed with each contour of the reservoir area. For this particular project elevation, area and capacity data are given.

Since the topography of the reservoir area is more of trapezoidal shape, the storage volumes of various elevations are determined by using trapezoidal formula:

$$V = \frac{A1 \ +A2}{2}(\Delta h)$$

Where: V is storage volume

A1 &A2 are planner area at each contour

Δ h=depth between consecutive contour

Table 3. 1 Elevation-Area-Storage

Elevation masl	Area km2	Com.vol
522	0	0
525	0.1	0.15
530	0.3	1.15
535	0.5	3.15
540	1	6.9
545	1.3	12.65
550	1.7	20.15
555	2.4	30.4
560	3.1	44.15
565	3.9	61.65
570	4.9	83.65

575	5.9	110.65
580	7.2	143.4
585	8.1	181.65
590	9.3	225.15
595	10.2	273.9
600	11.4	327.9
606	16.2	480.63
612	19.8	695.6
620	20.4	730.98

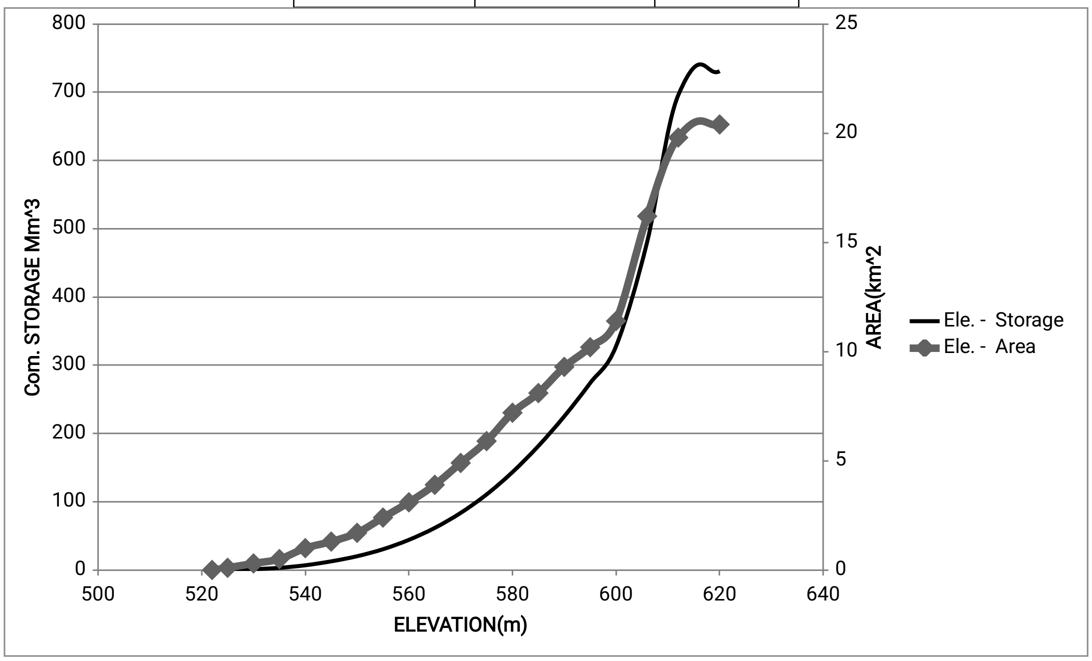

Figure 3. 1 Elevation-Area-storage curve

3.4 Reservoir Capacity Determination

The storage capacity of a reservoir is the maximum difference between the cumulative supply and demand during the period of the driest year of the available records. There can be different techniques that are used to determine the storage capacity of a reservoir, such as mass curve and sequent peak algorithm methods.

3.4.1 Mass Curve (Ripple) Method

A mass curve is a curve of cumulative net reservoir inflow against time and as such it rises continuously. Any point on the curve indicates total inflow from the beginning of the period up to the given time. The slope of the tangent to the mass-curve at any time is a measure of inflow

at that time. Demand line is a straight line, rising from the origin for a uniform rate of demand. The mass curve thus gives the relationship between the accumulated inflow and outflow and the water available for storage at any given time from the beginning of the year. Demand line drawn tangent to the high points of inflow mass curve represent rates of withdrawal from the reservoir.

The reservoir is full where ever a demand curve coincides the inflow curve. The maximum vertical ordinate between the two curves represents the reservoir storage capacity to meet the demand during the dry period.

Limitations of Mass curve method

- Doesn't consider the effect of evaporation, which is the major component in arid/semi-arid areas
- Not good for non-uniform demand conditions
- Since this method only uses short length of flow data and this can mislead the planning engineers sometimes
- It is not possible to relate the economic analysis of the project in this method

3.4.2 Sequent Peak Algorithm: -

Sequent peak algorithm is a plot between time on X-axis and cumulative inflow minus cumulative outflow on Y-axis. It is a simple and straight forward analytical procedure for computing reservoir capacity. And it is used as an excellent alternative to the mass curve method of determining reservoir capacity.

Advantages of SPA Method Over Mass Curve Method

- SPA is a modification of the Mass Curve analysis for lengthy time series and particularly suited to computer coding.
- In this method it's possible to include the effect of evaporation and to use lengthy time series flow data, unlike that of mass-curve method.
- An excellent alternative to the mass-curve method

Due to those advantages, for this specific project we selected sequent peak algorithm method.

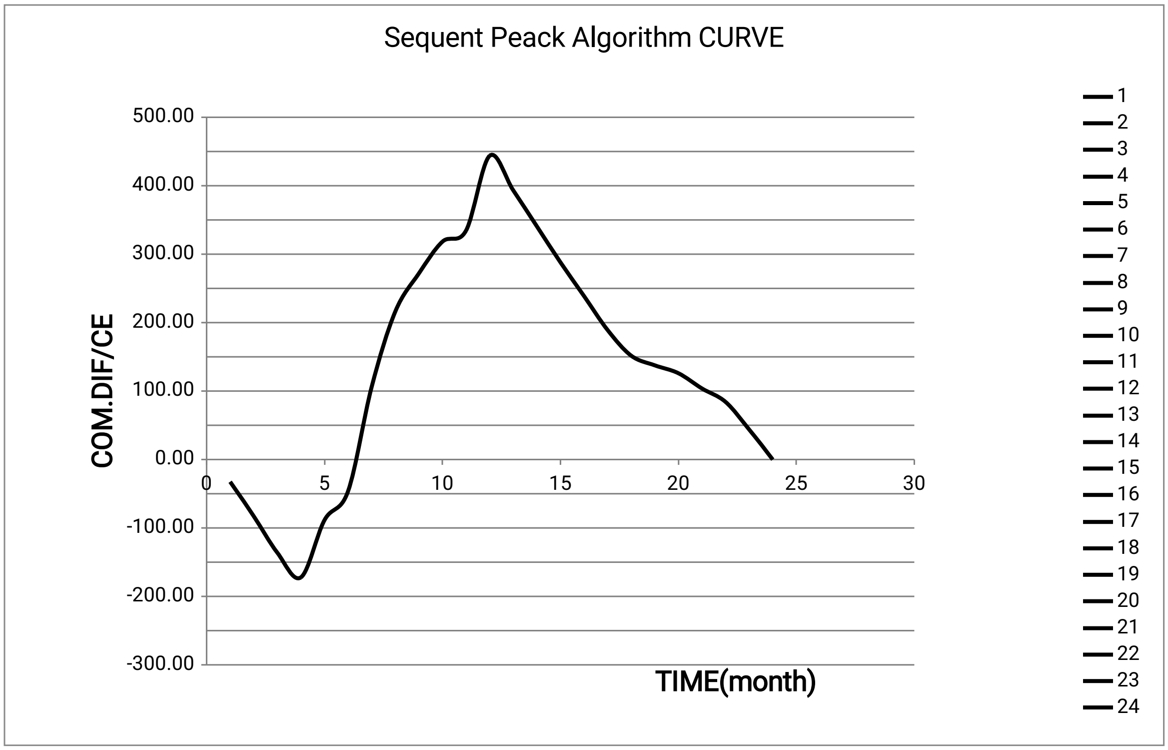

Figure 3. 2 Sequent Peak Algorithm curve

To draw the sequent peak algorithm two driest years (i.e.2000 &2012) are considered from the recorded data. And the volume of 615.76Mm^3 is determined as live storage by using sequent peak algorithm curve.

Use full life of a reservoir

The life of a reservoir can be expressed under various concepts such as the following

- **Use full life** - It is usually taken as the period through which the capacity occupied by sediment does not prevent the reservoir from serving its intended primary purpose.
- **Design life** – this is either useful life or shorter of the expected economic life or fixed span of life of 50/100 years keeping in view of various criteria.
- **Full life** – it is the number of years required for the reservoir capacity to be fully depleted by sedimentation.

For this particular project the use full life is taken to be 100yrs as the storage capacity is more than60Mm3.

(Source:Subramanya,K(1984),Engineering Hydrology.Mc Graw-Hill Publishers.Company Ltd, New Delhi.)

3.5 Reservoir Sedimentation

3.5.1 General

All rivers carry certain amount of sediment which is produced due to erosion in their catchments areas. The amount of sediment in a river depends on the extent of erosion in its catchments area which depends on the following factor

- Nature of soil in the catchments area
- Topographic of the catchments area
- Vegetation covers
- Intensity of rainfall

The sediment load carried by a river may be divided in to suspended load and bed load .the suspended load is the part of the sediment load which is held in suspension against gravity by the vertical component of eddies in the turbulent flow. The bed load on the other hand is that part of the sediment load that moves in contact with the bed of the river. It consists relatively coarser material.

When the sediment laden water of the river approaches the reservoir the velocity and turbulence are greatly reduced due to which bed load are deposited as dead storage in the reservoir. This deposition of the sediment in the reservoir is known as reservoir sedimentation or reservoir silting. In order to allow for such deposition of sediment certain percentage of the total storage is usually left unutilized which is defined as dead storage.

Sediments Earlier studies indicate that the sediment load in the Genale River is approximately 1 mill tons annually. Sediment composition and particle distribution 13% sand, 22% silt and 55% clay. The remaining 10% is bed load.

(Source:- GD6 feasibility study report)

3.5.2 Dead Storage

The rate at which the capacity of reservoir is reduced by sediment depends on:

- The quantity of sediment in flow
- The percentage of inflow sediment trapped in the reservoir
- The density of the deposited sediment

Total load = suspended load + bed load

Suspended load = 90% (1M tone/year) = 0.9*1M tone/year = 900,000 tone/yr

Bed load = 10% (1M tone/year) = 0.1*1M tone/year = 100,000 tone/year

The dead load storage in the reservoir taking the life of the reservoir as 100years is;

$$V_{Dead\ storage} = \frac{\text{Total Load IN Reservoir life}}{\text{Density of load}}$$

$$V_{\text{Dead storage}} = \frac{1000000 \ *10^3 \frac{Kg}{year}}{1300 \frac{Kg}{m^3}} *100year = 76923076.923m^3$$

$V_{\text{Dead storage}}$ = 76.923Mm^3

3.6 Reservoir Water Losses

Some quantities of water stored in a reservoir are always lost and hence it is necessary to account for this lost amount in the planning and operation of reservoir. The loss of water from a reservoir may be due to:

- Evaporation loss
- Absorption
- Percolation or reservoir leakage

As it is stated in ESIA (March, 2009), the soil in the reservoir is anticipated to be residual silt with varying content of sand and clay. So absorption and percolation losses are not considered for this particular case.

3.6.1 Evaporation Losses

The evaporation loss mainly depends on the water surface area of the reservoir.

The other factors influencing evaporation loss vary depending upon certain meteorological factors and the nature of the evaporating surface.

Meteorological factors which influence the rate of evaporation are temperature, wind speed, relative humidity, water quality, solar radiation, atmospheric pressure and surface area etc.

The loss of water due to evaporation is expressed in terms of depth of water measured in cm or mm which this represents the volume of water loss per unit area of the water surface of a reservoir.

Methods to estimate open water evaporation are:

- Empirical formula
- Pan evaporation
- Penman formula

$E_r = K*E_p$ (S.K. Garg)

Where, K= pan evaporation coefficient that vary from 0.6 to 0.8. So, pan coefficient of 0.7 is mostly recommended. K. Subramanya (1994)

Ep=Evaporation from pan

Er = Evaporation from reservoir

Monthly evaporation and rainfall recorded data from Dam site are given:

Table 3. 2 Evaporation and Precipitation at the Dam Site

Month	Evaporation(mm) Ep	Evaporation (mm),Er	PPt(mm)	Net Evapo (mm)
Jan	190	133	6	127
Feb	181	126.7	5	121.7
Mar	198	138.6	37	101.6
Apr	161	112.7	146	-33.3
May	163	114.1	98	16.1
Jun	167	116.9	7	109.9
Jul	170	119	1	118
Aug	192	134.4	0	134.4
Sep	188	131.6	17	114.6
Oct	163	114.1	84	30.1
Nov	149	104.3	53	51.3
Dec	175	122.5	17	105.5
Year	**2097**	**1467.9**	**471**	**996.9**

To calculate the net evaporation the following steps are done.

First, the reservoir at the normal pool level is calculated as

Reservoir capacity =dead storage + live storage

$= 615.76Mm^3 + 76.923Mm^3$

$= 692.683Mm^3$

From an area elevation capacity curve for a volume of $V=692.683Mm^3$ the reservoir surface area is $18Km^2$. And elevation 608m amsl.

So, from the given data of annual average evaporation depth (E_r) = **1467.9**mm/yr and annual average depth of precipitation (P mean) =**471**mm/yr.

Total volume of evaporation $=A*E_r$

Where A is Top reservoir surface area

Evaporation = $18*10^6 km^2*\left(\frac{\mathbf{1467.9}mm}{1000}\right)$ =**26.422Mm³**

In addition to evaporation loss, the effect of direct precipitation over the reservoir surface is also considered, and the mean annual rainfall record at the dam site is

471mm/yr.

Therefore, the total annual volume of water from direct rain fall over the reservoir surface is:

Ppt = $18*10^6 km^2*\left(\frac{\mathbf{471}mm}{1000}\right) = 8.478Mm^3$

Thus, Net Evaporation= Evaporation – Rainfall

$= 26.422Mm^3 - 8.478Mm^3$ **= 17.944Mm³**

Therefore, total storage capacity =dead storage +live storage + net evaporation volume

Total storage capacity = $76.923Mm^3 + 615.76Mm^3 + 17.944Mm^3$ = **710.627Mm³**

Therefore, from area elevation capacity curve for a storage capacity of V=**710.627Mm³**, the elevation of reservoir = 613m. amsl. (Spillway crest level)

3.7 Computation of Probable Life of Reservoir

To justify the 100 year useful life of GD-6 reservoir we used the relationship between trap efficiency and capacity-inflow ratio developed by Brune, and the table is shown below.

Trap Efficiency of Reservoir

The sedimentation of reservoir is measured in terms of trap efficiency. It is defined as the percent of inflowing sediment that is retained in a reservoir. Trap efficiency a function of the ratio of reservoir capacity to total inflow, i.e.

Trap Efficiency = $f\left[\frac{\text{Capacity}}{\text{Inflow}}\right]$

Trap efficiency is low for reservoirs having small capacity in flow ratio and it gradually increase as the capacity-inflow ratio increase. Further the trap efficiency of the reservoir will be more in the beginning but it will decrease as the capacity of reservoir is reduce due to sedimentation.

Tap efficiency depends mainly on the:

- Sediment characteristics,
- Detention time of the water in the reservoir and
- Degree of reservoir siltation.

Table 3. 3 Trap Efficiency Value for Different Values of Capacity-Inflow Ratio

Capacity-inflow ratio	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
Trap Efficiency (%)	87	93	95	95.5	96	96.5	97	97.3	94.4	97.5

Total reservoir capacity = **710.627Mm³**

Annual inflow = **120.3m³/sec. =** 120.3*365*24*3600 = **3700.235Mm³**

Annual sediment inflow =0.769 Mm3

$$N_o\,\text{year} = \frac{\text{Storage capacity}(Mm^3)}{\text{anual sediment }(Mm^3)\ \text{*Average trap efficiency}}$$

Table 3. 4 Computation Useful Life of Reservoir Using Trap Efficiency

%	Capacity in Mm3	Capacity-inflow ratio	Trap ef.	Avge.trap effi	Annual trapped Sediment in Mm3	Year
100	615	0.3	0.95	0.94	0.769	170.15
80	492	0.2	0.93	0.915		146.35
60	369	0.15	0.9	0.885		141.55
40	246	0.1	0.87	0.835		133.55
20	123	0.07	0.8			**591.62**

3.8 Determination of Height of Dam

To determine the dam height, we have two options:

i. From the area-elevation curve
ii. From the site conditions

1. From the Site Condition

From the topographical map, by using bed level of the dam and the maximum crest level of our dam it is possible to determine the height of dam. But in our project topographical map is not provided. So the only option we have is that using Area-Elevation curve.

2. From the Area-Elevation Curve

By calculating the live storage, dead storage and the evaporation volume, we can compute the total capacity of the reservoir corresponding to this volume. Then, by using elevation corresponding to total storage we can fix the dam height.

Bed level Elevation of Dam site = 522m amsl

The maximum storage capacity level = 613m amsl

Dam height (H) = 613-522 = 91m

3.9 Silting Control in Reservoirs

In order to increase the life of the reservoir, it is necessary to control the deposition of sediment. The various methods which are adopted to control the deposition of sediment in reservoir are as follows:

a) Selection of dam site: - The silting depends upon the amount of erosion from the catchments. If the catchment is less eroding able, the silting will less. Then the silting can be reduced by choosing the reservoir site in such a way as to exclude the runoff from the easily erodible catchments.

b) Construction of check dams:-The sediment inflow can be controlled by building check dams across the river stream contributing much sediment load.

c) Construction of under sluice in the dam:-The dam is provided with openings in its bed, so as to remove the more silted water on downstream side.

d) Reservoir design:-The rate of sedimentation is less for reservoir of small capacity having large inflow rate than that of reservoir of large capacity. Because, even on the same area, capacity inflow rate is small for small capacity reservoir.

e) Erosion control in the catchments:-The erosion in the catchments area may be controlled by the use of methods of soil conservation such as: afforestation, controlling of grazing, and provision of control bunds. The methods of soil conservation are quite effective in the control of sedimentation of reservoir but, they are costly and show appreciable result only after considerable time.

CHAPTER FOUR

4. DAM WORK

4.1 General

A dam is an obstruction built across a stream or a river. At the back of this barrier, water gets collected, forming a lake of water. The lake of water which is formed upstream is called a reservoir.

In most of the high head and medium head hydro-electric projects, a dam across the river is an important component.

The dam fulfills two fundamental functions in hydro-electric projects, namely

- It develops a reservoir which has a capacity to store water
- It builds up head and thus, potential for the river water

4.2 Classification of Dams

Dams may be classified in various ways. The classifications based on their function, shape, material of construction and hydraulic & structural design.

Based on their function dams may be classified as storage, diversion and detention dams. Based on the shape, they are classified as trapezoidal section, multiple arch dams, etc.

According to construction materials the dam may be classified as rock fill, earth, masonry gravity dams etc. Based on hydraulic design the dams may be classified as overflow, non-overflow, rigid and non-rigid dams.

4.3 Selection of Dam Site

The selection of a site for a dam depends upon the function of the dam. From cost consideration, smaller length of the dam requires less cost.

During dam site selection the following criteria are to be considered.

- Suitable foundations must be available
- The length of the dam should be small as possible and it should store the maximum volume of water.
- The general bed level at dam site should preferably be higher than that of the river basin.
- A suitable site for the spillway should be available in the near vicinity
- Materials required for the construction should be easily available
- The reservoir basin should be reasonably water-tight. The stored water should not escape out through its side walls and bed.
- The value of land and property submerged by the proposed dam should low
- The dam site should be easily accessible, so that it can be economically connected to important towns and cities by rails, roads, etc
- Site for establishing labor colonies and a healthy environment should be available in the near vicinity.

4.4 Selection of Dam Type

The various factors which must be thoroughly considered before selecting one particular type of dam as follows:

- Topography,
- Geology and foundation conditions
- Earthquake zone
- Height of the dam
- Available materials

1. **Topography**: - Topography dictates the first choice of the type of dam. The dam site on the GD 6 is nearly u-shaped narrow valley. So, the gravity dam could be chosen.
2. **Geology and foundation conditions**: - The rock formation at the dam site is fairly uniform consisting of post-tectonic pegmatoidal granite. It is composed mainly of coarse grained feldspar and quartz. The rock mass is relatively massive with only minor jointing. The major joint directions are vertical and horizontal. The rock mass is considered strong to very strong and watertight. In the dam area, weathering depths are generally shallow ñ of the order of 5m. The slopes at and around the dam

site are steep but stable ñ there or no unfavorable major joint planes. The shallow depth of weathering and the uniformly strong base rock particularly suit the hard, strong and densest of dam namely a concrete gravity dam.

3. **Earthquake zone:-** If the dam is to be situated in an earthquake zone, its design must include the earthquake forces. Most of the epicenters of observed earthquakes in Ethiopia are related to major rift structures.
4. **Height of the dam:-** Earthen dams are usually provided for low heads. Hence, for high heads, gravity dams are generally preferred.
5. **Available materials:-** Availability of suitable materials is one main criterion for selection of dam type. In the Genale GD-6 project area the rock is of very good quality, easily available close to the dam site at a location that will remain under lowest regulated water levels. The tunnels will also be excavated in rock type well suited for dam construction and for concrete aggregates. Clayey or other types of natural materials suitable for use as low permeability core material are not available in sufficient quantities. The scarce deposits of clayey materials are decomposed from partly gypsiferous materials and are not suited for dam construction because of its swelling and soluble properties. Consequently, an earth fill dam with clay core is out of question for GD-6 dam.

Based on all above criteria the dam should be constructed using rock materials, either as rock fill dam or as concrete dam.

Three different gravity dam types have been evaluated for GD-6 dam:

- Roller Compacted Concrete Dam (RCCD)
- Concrete Faced Rock Fill Dam (CFRD)
- Asphaltic Concrete Core Rock Fill Dam (ACCRD)

Roller Compacted Concrete Dam (RCCD)

The RCC dam type has several advantages related to layout and to volume of construction materials. The type is furthermore not very sensitive to weather condition during construction.

A major advantage is that the dam crest may be used as spillway. The water diversion structures may be less comprehensive during construction, and the consequences of an accidental overtopping during construction are not as disastrous as for an embankment dam.

The disadvantages are the cost and the need of large quantities of cement and slag for cement replacements, which are not available in the area.

Concrete Faced Rock Fill Dam (CFRD)

Rock fill dams must contain an impervious zone. As no suitable clay materials are available in the area, one option is to construct a concrete slab on the upstream slope of the dam. The concrete face has the advantage that it requires relatively small quantities of materials that need to be transported long distances. The only material not available locally is the cement for the relatively thin front concrete slab. At GD-6 the rock materials are of good quality suitable for dam construction. The cost is favorable.

A disadvantage is the need of spillway structure constructed at the side of the dam, or constructed as a separate concrete structure somewhere within the dam body.

Asphaltic Concrete Core Rock Fill Dam (ACCRD)

An alternative to use a concrete slab as upstream impervious zone in the rock fill dam is to construct an impervious central core of asphaltic concrete. Asphaltic concrete has a special mix of binder in an aggregate mix instead of cement. The core is impervious and flexible, which is of advantage with regard to settlements in the supporting rock fill.

Another advantage is that the construction method is less vulnerable to adverse weather conditions than the other dam types.

The cost estimates and also prefeasibility study of GD6 hydro power project indicates that an asphaltic concrete core would be the most favourable dam type for GD-6. The Consultant has selected ACCRD as basis for the feasibility study. However, unit construction cost as well as cost of materials changes continuously. In connection with final design the cost estimates should be reviewed based on up- dated unit rates.

(Source Garg, S.K (1979), Irrigation Engineering and Hydraulic Structures. Khanan publishers, New Delhi.. and Feasibility study report of GD-6)

4.5 Design of Gravity Dam

4.5.1 General

Gravity dam is most durable, solid and requires very little maintenance, if it designed properly. They can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the dam.

Such a dam is generally straight in plan, although sometimes, it may be slightly curve. The line of the upstream face of the dam, or the line of crown of the dam if the upstream face in sloping, taken as the reference line for layout purposes, etc and is known as the base line of the dam or the axis of the dam. This type of the dam can be constructed up to great heights.

4.5.2 Determination of Section of the Dam

Depth of water at the upstream of the dam (H):-

H= normal pool level (NPL) – river bed level (RBL)

H = 608-522=86m

Height of free board

Free board is the vertical distance between the top of the dam and the maximum water level in the reservoir. The water may overtop the dam when the waves develop in the reservoir and cause splashing over the dam. Now a day, for high head dams' free board is provided as 4 up to 5% of the dam height.

Taking 4.5% , **FB** = **3.87m, provide 4m**

Maximum height due to flood above normal pool level

MFL – NPL= 613-908 =5m

Where: MFL – Maximum flood level

NPL – Normal pool level

Total height of the dam above the river bed level, HD

H_D = U/S water depth + FB + height due to flood above normal pool level

H_D = 86+4+5 = 95m

Top width, **B**

$$B = 0.552\sqrt{H'}$$

Where H' = MFL – RBL

=613m-522m = 91m

B $= 0.552\sqrt{91}$ = **5.266m, provide 6m**

Base width of the dam, BD

$$B_D = H' * \tan \varnothing_d + H \tan \varnothing_u$$

Where: $\tan \varnothing_d$ is downstream slope

$\tan \varnothing_u$ is up stream slope

An approximate definition of the downstream slope in terms of its angle to the vertical, $\varnothing_d$ required for no tension to occur at a vertical upstream face is given by

$$\tan \varnothing_d = \sqrt{\frac{1}{\gamma_c/\gamma_w - \vartheta}} \qquad \text{taking} \quad \vartheta = 1$$

$\gamma_c = 24KN/m^3$ $\gamma_w = 10KN/m^3$

$$\tan \varnothing_d = \sqrt{\frac{1}{\frac{24}{10} - 1}} = 0.85$$

B_D = 91*0.85+86tan90^0 = **77.35m take 78m**

Crest length of the dam

According to Lecy's equation the perimeter of the river cross section is given by

$p = 4.75\sqrt{Q}$ $Q = Q_p = 1800\ m^3/s$

P = 201.525 m

For wide channel perimeter is equal to or slightly greater than the length of the channel. Therefore the crest length of the dam is fixed to be 200 m.

4.5.3 Determination of Type of Dam (Low or High)

A distinction between low and high gravity dam is usually made on basis of the limiting height. The limiting height is the critical height by which the maximum values of the principal stress should not exceed the allowable stress for the material of the dam.

The dam height above foundation level up to the level of the crest is 86m

$$H_{cr} = \frac{\sigma_{allow.concrete}}{\gamma_w \frac{\gamma_c}{\gamma_w} + 1}$$

Where . $\sigma_{allow.concrete}$ = allowable stress of concrete

γ_c = Unit weight of concrete = $24KN/m^3$

γ_w = Unit weight of water = $10\ KN/m^3$

Taking δallow.concrete = 3000 KN/m2

$$\mathbf{H_{cr}} = \frac{3000}{10\frac{24}{10} + 1} = \mathbf{88.23m}$$

From the above result H=86m < Hcr =88.23m, so the dam is low gravity dam

4.5.4 Forces Acting on Gravity Dam

A. primary loads

1. **Water pressure**: is the major external force acting on gravity dam. The horizontal water pressure, exerted by the weight of the water stored both on the upstream and downstream side of the dam can be estimated from rule of hydrostatic pressure distribution. The resultant force due to water = $\frac{\gamma_{wH^2}}{2}$ acting at H/3 from the base.

H = the depth of water

When the upstream is, inclined or partly vertical and partly inclined, the resulting water force can be resolved in to horizontal component (p_h) and vertical component (p_v).

$P_h = \frac{\gamma_{wH^2}}{2}$ Acts at H/3 from the base.

Vertical water load from the reservoir, Pwv

There is no vertical water load on dam that by considering vertical up-stream dam.

Vertical water load from the tail water:

$$P'_{wv} = \gamma_w A_2$$

A_2 is the cross-sectional area of the water carried by the downstream inclined surface of the dam.

2. **Weight of the dam, Pm**

Weight of the dam is major resisting force to keep the stability of gravity dam. For analysis purpose, generally, unit length of the dam is considered. The cross section of the dam may be divided into several triangles and rectangles, and the weights w_1, w_2, w_3 etc of each of these may be computed conveniently, along with determination of their lines of action.

The total weight w of the dam acts at the center of gravity of its section.

$$P_m = \gamma_c A_p$$

Where:

Ap = Area of dam profile, at which the force of the dam is acting on.

3. **Uplift pressure, Pu**

The uplift pressure is the upward pressure of water as it flows or seeps through the body of the dam or its foundation. The uplift pressures can be controlled by constructing cut-off between the dam and its foundation, and by pressure grouting of the foundation.

$$P_u = \gamma_w H' + \frac{1}{3}\gamma_w [H - H']$$

Where, H' is the depth of tail water

H is the depth of water at the upstream side.

B. Secondary loads

1. Silt Pressure:

The sediment particles try to settle down to the river bottom due to the gravitational force, but may keep in suspension due to the upward currents in the turbulent flow which may overcome the gravity force.

The force exerted by this silt can be represented by:

$$P_{silt} = \frac{\gamma_{sub} * h_s^2 * k_a}{2}$$

$$k_a = \frac{1 - \sin \varnothing}{1 + \sin \varnothing}$$

Where ka = the coefficient of active earth pressure of silt

$\varnothing$ = the angle of internal friction of soil

γ_{sub} = submerged unit weight of silt material

hs = height of silt deposited

This pressure acts at **hs/3** from the base of the dam.

For computing the silt pressure by this formula, it is essential to know the values of γsub and ∅ for the soil. It is difficult to estimate the values of γsub and ∅ because the characteristics of the deposited silt vary considerably. Therefore, the computed silt pressure may not be reliable.

ARORA Dr.K.R, Irrigation, Water Power and Water Resource Engineering, Standard Publishers distributors, Delhi, 2002. recommends an empirical formula method to estimate the force due to silt pressure.

For horizontal component of force, P_{hs}, $P_{hs} = \frac{\omega_{sh^2}}{2}$

Where:

w_s is the specific weight of the equivalent liquid.

For horizontal component, w_s = 13.34 KN/m^3

For vertical component of force, Pvs $= \omega_s * A_s$

Where As is the area of an inclined surface above the base up to the height of silt deposited. For vertical component, ω_s= 18.88 KN/m^3

For this particular project the up-stream face of dam is vertical, so no vertical silt loads on dam.

For this particular project the empirical method is used to calculate horizontal silt loads.

2. **Wave pressure**: When the wind blows over the water surface of the reservoir, it generates wave pressure depends upon the wave height. It is given by the equation.

$$h_w = 0.032\sqrt{VF} + 0.76 - 0.271 * F^{1/4}, \quad \text{for } F \leq 32$$

$$h_w = 0.032\sqrt{VF}, \quad \text{for } F > 32$$

Where: hw = height of water from top of crest to bottom of trough (wave height), m

V = Wind velocity, km/hr

F = Fetch or straight length of water expanse,

Also it is possible to evaluate wave load by using the formula:

$$P_{wave} = 2 * \gamma_w * H_s^2$$

$$H_{s} = 0.75*H$$

Where: H is total dam height

Since we are not provided with V & F we used the later formula

H_s = 0.75*95 = 71.25m

P_{wave} = 2*10*71025^2 = 101531.25KN/m^2

C. **Exceptional loads:** are so designed on the basis of limited general applicability of occurrence.

Earthquake forces

Earthquake wave may travel in any direction. However, it is customary to resolve the general direction of acceleration into horizontal and vertical direction.

A vertical acceleration may either act downward or upward. When it is acting in the upward direction, the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move down ward away from the dam body; thus reducing the effective weight and the stability of the dam, and hence is the worst case for design.

Horizontal acceleration may cause the following two forces:

- Hydrodynamic pressure
- Horizontal inertial force

But in our case, Since GD-6 project site is located on zone zero (from feasibility study report), (Earth quick occurrence is negligible) we neglect earth quick effect.

All load calculation and moment analysis is presented on (Annexes-3)

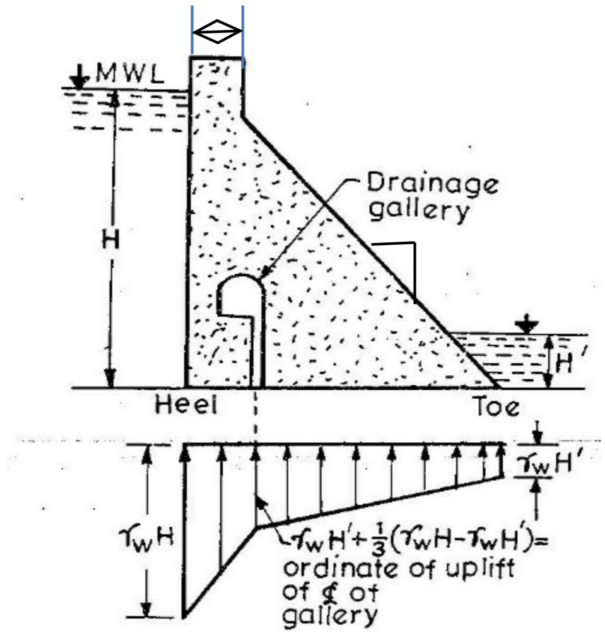

Figure 4. 1 Dam section

4.6 Stability Analysis Requirements of the Dam

The gravity dam must be in overall equilibrium. It should not move in any direction or rotate about any point. In addition to the overall stability, the internal stresses induced anywhere in the dam must be within the safe limits. The foundation should be able to withstand the imposed loads.

Assessed in relation to all probable conditions of loading including the reservoir empty condition, the profile must demonstrate an acceptable margin of safety with regard to:

- Rotation and overturning
- Translation and sliding
- Overstress and material failure

4.6.1 Overturning Stability

Factor of safety against overturning, Fo can be expressed in terms of moments about the downstream toe of any horizontal plane and is defined as:

$$F_o = \frac{\sum M_{+ve}}{\sum M_{-ve}}$$

Where: $\Sigma M + ve$= the total stabilizing moment about the toe

$\Sigma M - ve$= the total overturning moment about the toe.

Values of FO greater than 1.25 may generally be acceptable, but $Fo \geq 1.5$ is desirable.

4.6.2 Sliding Stability

The sliding stability is a measure of determining the resistance of the structure against sliding on any horizontal failure plane.

Factor of safety against sliding is estimated using one of the three definitions:

- Sliding factor, FSS
- Shear friction factor, FSF
- Limit equilibrium factor, FLE

Sliding Factor, FSS: In this case, the resistance against sliding is purely frictional, and no shear strength or cohesion is involved.

- For a horizontal plane:

$$Fss = \frac{\sum H}{\sum V}$$

Where: ΣH= Summation of all horizontal load components

ΣV= Summation of all vertical load components.

- If the plane is inclined at a small angle "α" then FSS is modified to

$$Fss = \frac{\frac{\sum H}{\sum V} - \tan\alpha}{1 + \left(\frac{\sum H}{\sum V}\right)\tan\alpha}$$

Fss on a horizontal plane should not be permitted to exceed 0.75 for the specified normal load combination, but it may be permitted to rise to 0.9 under the extreme load combination.

(Source: P.Novak, A.I.B Maffat,C. Nalluri, R.Narayanan, 1996 "Hydraulic structure "(2nd))

Shear friction factor, FSF: It is the ratio of the total resistance to shear and sliding which can be mobilized on a plane to the total horizontal load.

$$FSF = \frac{S}{\sum H}$$

Where: S = maximum shear resistance which can be mobilized.

$$S = \frac{CA_h}{\cos\alpha(1 - \tan\varnothing \tan\alpha)} + \sum V\tan(\varnothing + \alpha)$$

Where: Ah = area of plane of contact or sliding

C = Cohesion of the material at the plane of contact (which is varies from 1400kN/m2 for poor rocks to about 4000KN/m2 for good rocks)

∅ = internal friction angle of the material

To be on the safe side; for GD-6 hydropower project sliding factor, FSS is computed since the foundation base is horizontal.

4.6.3 Stress Analysis

Straight gravity dams are generally analyzed by the gravity method of stress analysis.

The gravity stress analysis is derived from elastic theory and is applied to two dimensional vertical cantilever sections on the basis of the assumptions listed below.

i. The concrete is homogenous, isotropic and uniformly elastic. All loads are carried by gravity action of vertical parallel sided cantilevers with no mutual support between adjacent cantilevers (Monoliths).

ii. No differential movements affecting the dam or foundation occur as a result of the water load from the reservoir.

iii. Vertical stresses on horizontal planes vary uniformly between upstream and downstream face.

iv. Variation in horizontal shear stress across horizontal planes is parabolic.

a. Vertical normal stresses

The analysis is based on modified beam theory which is acted up on by combined axial and bending load.

$$\delta_z = \frac{\sum V}{A_n} \pm \frac{\sum M + Y'}{I}$$

Where: $\sum V$ = the resultant vertical load above the plane considered, exclusive of

up lift.

$\sum M$= the summation of moments determined with respect to the centroid of the plane.

Y' is the distance from the neutral axis of the plane to the point where δ_z is being determined

I= the second moment of area of the plane with respect to its centroid.

For the reservoir full condition

At the upstream face $\delta_{zu} = \frac{\sum V}{B_D}(1 - \frac{6e}{B_D})$

At the downstream face $\delta_{zd} = \frac{\sum V}{B_D}(1 + \frac{6e}{B_D})$

Where "e" is the eccentricity of the resultant load, R

b. Horizontal shear stresses

If the angles between the face slopes and the vertical are respectively $\varnothing_u$ upstream and $\varnothing_d$. downstream, and if an external hydrostatic pressure, Pw is assumed to operate at the upstream face, then

$$\tau_u = (P_w - \delta_{zu})\tan \varnothing u$$

$$\tau_d = \delta_{zd}\tan \varnothing d$$

c. Horizontal Normal stresses

The horizontal normal stresses on vertical planes .δ_y can be determined by consideration of the equilibrium of the horizontal shear forces operating above and below a hypothetical element within the dam. The difference in the shear forces is balanced by the normal stresses on vertical planes and it is expressed as

- For the upstream face: $\delta_{yu} = P_w + (\delta_{zu} - P_w)\tan \varnothing u^2$
- For the downstream face: $\delta_{td} = \delta_{zd} * \tan \varnothing u^2$

d. Principal stresses

The vertical stresses are not the maximum direct stresses. The maximum direct stress in a complex stress system is the principal stress. The principal stresses are acting on the principal plane passing through the points.

The boundary values for major principal stresses,δ_1 and the minor principal stresses, δ_3 are given by the equation:

For the down steam section:

$$\delta_{id} = \delta_{zd} * \sec \varnothing d^2 - P' \tan \varnothing d^2$$

Where δ_{id}=is one of the principal stresses on the downstream section of the dam.

P'_w =is tail water hydrostatic pressure

For upstream section:

For reservoir full condition is calculated as:

$\delta_{3u} = \delta_{zu} * (1 + \cot \varnothing u^2)(P_w + P_{ew})\tan \varnothing u^2$, $\delta_{1u} = P_w$

Pew=is hydrodynamic force due to reservoir water action for seismic acceleration operating downstream

Tension: - concrete gravity dams are designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses.

4.7 Load Combination for the Design of Gravity Dam

All the forces may not act simultaneously on the dam. The design of the dam is based on the most adverse combination of loads which have a reasonable probability of simultaneous occurrence. The following load combinations should be considered for the design of gravity dams.

i. Load combination A: Normal operating condition

- Reservoir full condition
- Normal uplift force (with drainage)
- Tail water normal condition
- Silt force

ii. Load combination B: High flood condition.

- Maximum reservoir level
- Normal uplift force
- Tail water at flood elevation
- Silt force

iii. Load combination C:

- Load combination B with extreme uplift (Drainage inoperative)

Stability Analysis of the Dam

The dam as a whole should be structurally safe and stable. It should be able to withstand the stresses developed due to imposed loads. The foundations should be strong enough to carry the loads.

Stability Analysis of Dam for Different Load Combinations

For load combination A

$$X = \frac{\sum M}{\sum V} = \frac{83530282.59}{1629333.21} = 51.26654393$$

$$e = 0.5B_D - X = 0.5*78 - 51.26654393 = -12.266$$

Check for Tension

$$\frac{B_D}{6} = \frac{78}{6} = 13$$

$e < \frac{B_D}{6}$, tension will not occur

Check for sliding

$\sum M^{+} = 24312671.8$ KN/m2 $\quad \sum V^{+} = 416604$

$\sum M^{-} = 12941476.88 \text{KN/m}^2 \quad \sum V^{-} = 254800.43$

$\sum H = 19378.91 \quad \sum V = 796125.21$

a. Sliding factor, Fss

$$Fss = \frac{\sum H}{\sum V} = \frac{193780.91}{796125.21} = 0.243$$

$F_{ss} = 0.243 < 0.75$. So it is safe!

b. Shear friction factor, FSF

$$FSF = \frac{CAh + \sum V \tan \varnothing}{\sum H}$$

C =Cohesion of the material at the plane of contact which varies from 1400kN/m2 for poor rocks to about 4000KN/m2 for good rocks) let us take C=3000KN/m2

$$FSF = \frac{300 * 78 + 796125.21 * 0.7}{19378.91} = 4.5$$

FSF > 3 it is OK !

Analysis of stresses

i. Vertical normal stresses

At the upstream face (Heel) $\quad \delta_{zu} = \frac{\sum V}{B_D}\left(1 - \frac{6e}{B_D}\right)$

$$\frac{96125.21}{78}(1 - \frac{6(-12.266)}{78}) = 2395.17$$

At the Downstream face (Toe) $\quad \delta_{zd} = \frac{\sum V}{B_D}\left(1 + \frac{6e}{B_D}\right)$

$$\frac{96125.21}{78}(1 + \frac{6(-12.266)}{78}) = 69.58$$

ii. Principal Stresses

At Downstream $\delta_d = \delta_{zd} \sec \varnothing d^2$

$= 69.58*1.7225 = 119.85$

At Upstream $\delta_u = \delta_{zu}(1 + \tan \varnothing u^2) - P_w \tan \varnothing u^2$

$$2395.17(1+0)-0 = 2395.17$$

iii. Shear Stresses

At Downstream $\tau_d = \delta_{zd} \tan \varnothing d$

$$=2395.17*0.85 = 2035.89$$

At Heel $\tau_u = -[\delta_{yu} - p_w] \tan \varnothing u$

There is no shear stress at upstream since ø = 0

For Load combination B: High flood condition.

$\sum M^+ = 3913783$ KN/m2 $\sum V^+ = 416604$

$\sum M^- = 9367541.131$ KN/m^2 $\sum V^- = 197428.78$

$\sum H = 19378.9$ $\sum V = 796125.21$

$\sum M = 35069612.56$

$$X = \frac{\sum M}{\sum V} = \frac{35069612.56}{796125.21} = 44.05$$

$$e = 0.5B_D - X = 0.5 * 78 - 44.05 = -5.05$$

Check for Tension

$$\frac{B_D}{6} = \frac{78}{6} = 13$$

$e < \frac{B_D}{6}$, tension will not occur **OK !**

Check for sliding

Sliding factor, Fss

$$Fss = \frac{\sum H}{\sum V} = \frac{193780.91}{796125.21} = 0.243$$

$F_{ss} = 0.243 < 0.75$. So it is **safe !**

Shear friction factor, FSF

$$FSF = \frac{CAh + \sum V \tan \varnothing}{\sum H}$$

$$FSF = \frac{300 * 78 + 796125.21 * 0.7}{19378.91} = 30$$

FSF > 3 it is **OK !**

Check for overturning, F0

$F_O = \frac{\sum M^+}{\sum M^-} = \frac{9367541.131}{3913783} = 2.393474718 \quad F_O > 1.5$ it is **ok !**

Analysis of stresses

I. **Vertical normal stresses**

At the upstream face (Heel) $\delta_{zu} = \frac{\sum V}{B_D}\left(1 - \frac{6e}{B_D}\right)$

$$\frac{796125.21}{78}(1 - \frac{6(-5.05)}{78}) = 14171.653$$

At the Downstream face (Toe) $\delta_{zd} = \frac{\sum V}{B_D}\left(1 + \frac{6e}{B_D}\right)$

$$\frac{796125.21}{78}(1 + \frac{6(-5.05)}{78}) = 6241.81$$

II. **Principal Stresses**

At Downstream $\delta_d = \delta_{zd} \sec \varnothing d^2$

$= 6241.81*1.7225 = 10751.52$

At Upstream $\delta_u = \delta_{zu}(1 + \tan \varnothing u^2) - P_w \tan \varnothing u^2$

$14171.653\ (1+0)-0 = 14171.65$

III. **Shear Stresses**

At Downstream $\tau_d = \delta_{zd} \tan \varnothing d$

$=6241.81*0.85 = 5305.54$

At Heel $\tau_u = -[\delta_{yu} - p_w] \tan \varnothing u$

There is no shear stress at upstream since ø = 0

Load combination C

$\sum M^+ = 3913783.19$ KN/m2 $\qquad \sum V^+ = 416604$

$\sum M^- = 20280060\ KN/m^2 \qquad \sum V^- = 401700$

$\sum H = 19378.91$

$\sum V = 796125.21 \qquad \sum M = 35069612.56$

$$X = \frac{\sum M}{\sum V} = \frac{35069612.56}{796125.21} = 44.05$$

$$e = 0.5B_D - X = 0.5 * 78 - 44.05 = -5.05$$

Check for Tension

$$\frac{B_D}{6} = \frac{78}{6} = 13$$

$e < \frac{B_D}{6}$, tension will not occur **OK !**

Check for sliding

Sliding factor, Fss

$Fss = \frac{\sum H}{\sum V} = \frac{193780.91}{796125.21} = \mathit{0.243}$ F_{ss} = 0.243 < 0.75. So it is **safe !**

Shear friction factor, FSF

$$FSF = \frac{CAh + \sum V \tan \varnothing}{\sum H}$$

$FSF = \frac{300 * 78 + 796125.21 * 0.7}{19378.91} = 30$ FSF > 3 it is **OK !**

Shear friction factor, FSF

$$FSF = \frac{CAh + \sum V \tan \varnothing}{\sum H}$$

$FSF = \frac{300 * 78 + 796125.21 * 0.7}{19378.91} = 30$ FSF > 3 it is **OK !**

Check for overturning, F0

$F_O = \frac{\sum M^+}{\sum M^-} = \frac{9367541.131}{3913783} = 2.393474718$ F_O >1.5 it is **ok !**

Analysis of stresses

I. Vertical normal stresses

At the upstream face (Heel) $\delta_{zu} = \frac{\sum V}{B_D}\left(1 - \frac{6e}{B_D}\right)$

$$\frac{796125.21}{78}\left(1 - \frac{6(-5.05)}{78}\right) = 14171.653$$

At the Downstream face (Toe) $\delta_{zd} = \frac{\sum V}{B_D}\left(1 + \frac{6e}{B_D}\right)$

$\frac{796125.21}{78}(1 + \frac{6(-5.05)}{78}) = 6241.81$

II. **Principal Stresses**

At Downstream $\delta_d = \delta_{zd} \sec \varnothing d^2$

$= 6241.81*1.7225 = 10751.52$

At Upstream $\delta_u = \delta_{zu}(1 + \tan \varnothing u^2) - P_w \tan \varnothing u^2$

$14171.653\ (1+0)-0 = 14171.65$

III. **Shear Stresses**

At Downstream $\tau_d = \delta_{zd} \tan \varnothing d$

$=6241.81*0.85 = 5305.54$

At Heel $\tau_u = -[\delta_{yu} - p_w] \tan \varnothing u$

There is no shear stress at upstream since $\varnothing_u = 0$

Our dam is checked for different load combination for stability and it is safe for all cases. So it can be constructed without any doubt failure in its design life.

4.8 GALLERY IN DAM

For GD-6 hydropower project, the foundation gallery is provided at 2m above the foundation level of the dam. The gallery extends from one abutment to the other for the full length of the dam. It is used both for the drainage of the water percolating through the dam and to provide access to the interior of the dam for inspection of the dam from inside. It is located 23m away from the upstream of the dam. Since it is used for dual purpose, the size of gallery has 5.0m width and 4.0 m height.

CHAPTER FIVE

5. SPILLWAYS AND ENERGY DISSIPATOR

5.1 General

A spillway is a structure constructed at a dam site, for effectively disposing of the surplus water from upstream side of the dam to downstream side.

Spillways are provided for storage dams to release surplus or flood water, which cannot be contained in the allotted storage space.

There are several spillway designs. The choice of design is a function of the nature of the site, the type of dam and the overall economics of the scheme.

The importance of a safe spillway cannot be over emphasized and a spillway of insufficient capacity has caused many failures of dams. So, proper design of spillway is important before the implementation of the structure.

5.2 Essential Requirements of a Spillway

The essential requirements of a spillway are:

- The spillway must have sufficient capacity
- It must be hydraulically and structurally adequate
- It must be so located that it provides safe disposal of water, i.e.spillway discharge will not erode or undermine the downstream of the dam.
- The bounding surfaces of the spillway must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir velocities created by the reservoir surface to the tail water.
- The spillway discharge should not exceed the safe discharge of the downstream channel to avoid its flooding.

5.3 Flood Routing

5.3.1 General

The extent by which the inflow hydrograph gets modified, due to the reservoir storage can be computed by a process known as reservoir routing.

Reservoir routing is a process of computing water level in the reservoir and out flow rates corresponding to a particular inflow hydrograph at various instant of time. It is carried out to determine the maximum water level and the corresponding out flow rates when the maximum flow passes over the spillway. The maximum water level is required for fixing the height of the dam while the maximum out flow rate is required for the design of spillway.

5.3.2 Inflow Hydrograph

It is a graph of inflow versus time. In order to develop an inflow hydrograph

- Hourly measured stream flow data or
- UH (unit hydrograph) development for the basin

However such information is not available for the GD-6 dam site locations. In order to construct a unit hydrograph for this project empirical equations of a regional validity, which relate the salient hydrograph, cross section to the basin are available.

The unit hydrograph derived from such relationships are known as synthetic unit hydrographs.

Synthetic unit hydrograph is one of the Snyder's methods that are based on the study of large catchments in united states. The basin characteristics considered by Snyder's synthetic unit hydrograph are the area and shapes of the catchments.

Note: - the base period is taken to be five times the time to peak from kriptch's equation:

$Tc = 0.01947L^{0.77} S^{-0.385}$

Where;

Tc= time of concentration(min)

L =the length of main stream (m) =10000m

S=Average slope of the catchments=0.3%

Thus;

$Tc=0.01947L^{0.77} S^{-0.385}$

$= 365.14min$

And study of unit hydrograph of many large and small rural water sheds indicates that the basin lags;

And from the soil conservation service (SCS);

$t_p = 0.7*Tc$

$=4.26h$

where;

t_p =lag time from the midpoint of the rainfall e -excess duration tr to the peak of a unite hydrograph (hr).

tr=tp/5.5

= 0.775hr

tr=standard duration of the rainfall excess (hr)

t_R=3hr

t_R=duration of rainfall excess other than the standard duration tr (hr).

tp'=tp+(tR-tr)/4

= 4.82hr

t'_p= the lag time for t_R (hr)

t'pk = $t_R/2+t_p'$

= 6.316hr

t'_{PK} = time to peak from the beginning of the rainfall excess t_R (hr)

The design flood including regulated flow is,1440m^3/sec.

Table5. 1 Coordinate of Dimensionless Unit Hydrograph and the Corresponding Time

t/tpk	Q/Qp	T	Q	Qbase	Qi
0	0	0	0	35.6	35.6
0.1	0.015	0.6	17.841	35.6	53.441
0.2	0.075	1.2	89.205	35.6	124.805
0.3	0.16	1.8	190.304	35.6	225.904
0.4	0.28	2.4	333.032	35.6	368.632
0.5	0.43	3	511.442	35.6	547.042
0.6	0.6	3.6	713.64	35.6	749.24
0.7	0.77	4.2	1120.23	35.6	1258.96
0.8	0.89	4.8	1351.36	35.6	1400.65
0.9	0.97	5.4	1524.56	35.6	1712.35
1	0.99	6	1725.98	35.6	1800
1.1	0.98	6.6	1523.01	35.6	1652.45
1.2	0.92	7.2	1354.65	35.6	1426.42
1.3	0.84	7.8	1125.94	35.6	1258.35
1.4	0.74	8.4	977.45	35.6	1065.78
1.5	0.66	9	785.00	35.6	820.60

1.6	0.56	9.6	666.06	35.6	701.66
1.8	0.42	10.8	499.55	35.6	535.15
2	0.32	12	380.61	35.6	416.21
2.2	0.24	13.2	285.46	35.6	321.06
2.4	0.18	15.16	214.09	35.6	249.69
2.6	0.13	16.42	154.62	35.6	190.22
2.8	0.098	17.69	116.56	35.6	152.16
3	0.075	18.95	89.21	35.6	124.81
3.5	0.036	22.11	42.82	35.6	78.42
4	0.018	25.26	21.41	35.6	57.01
4.5	0.009	28.42	10.70	35.6	46.30
5	0.004	31.58	4.76	35.6	40.36

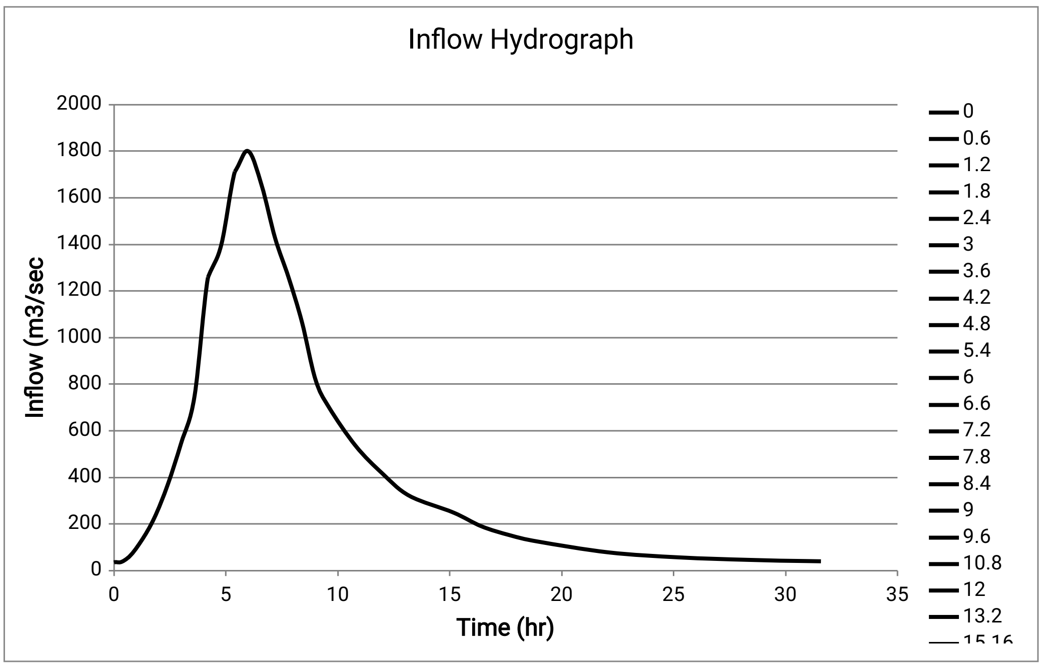

Figure5. 1 Inflow Hydrograph

5.3.3 Out Flow Hydrograph

The hydrograph of flood entering a reservoir will change in shape as it emerged out of the reservoir, because of temporary storage. This hydrograph which is modified by reservoir when a particular flood passes through it and used to determine the maximum rise in the water discharge in the river on the downstream is called outflow hydrograph.

Therefore, the outflow discharge is computed by considering the overall spill out of water.

This means

$Q = CLH^{1.5}$

Where; C=coefficient of discharge

L=effective length of the spillway crest (m)

H=Measuring head above crest in meter.

Table 5. 2Outflow and Head

Time(hrs)	Inflow(m3/s)	(I1+I2)/2	Out flow	S1/t-0.5Q1	S2/t+0.5Q2	H
0	35.6	260.3	0	0	260.3	0
3	485	850	120.32	229.972	1029.972	0.5
6	1215	1056.5	510.78	809.553	1866.053	1.5
9	898	682	952.36	1355.266	2037.266	3.0
12	466	363	1289.56	1459.567	1822.567	38.0
15	260	202.5	1420.5	1328.417	1530.917	4.7
18	145	120	1235.56	1144.339	1264.339	3.5

21	95	79	894.78	969.397	1048.397	3.0
24	63	56.5	610.32	822.344	878.844	2.2
27	50	48	403.64	703.066	751.066	15.0
30	46	43.1788	230.58	610.677	653.856	1.0
31.59	40.3576	40.3576	115.06	538.787	558.966	0.4

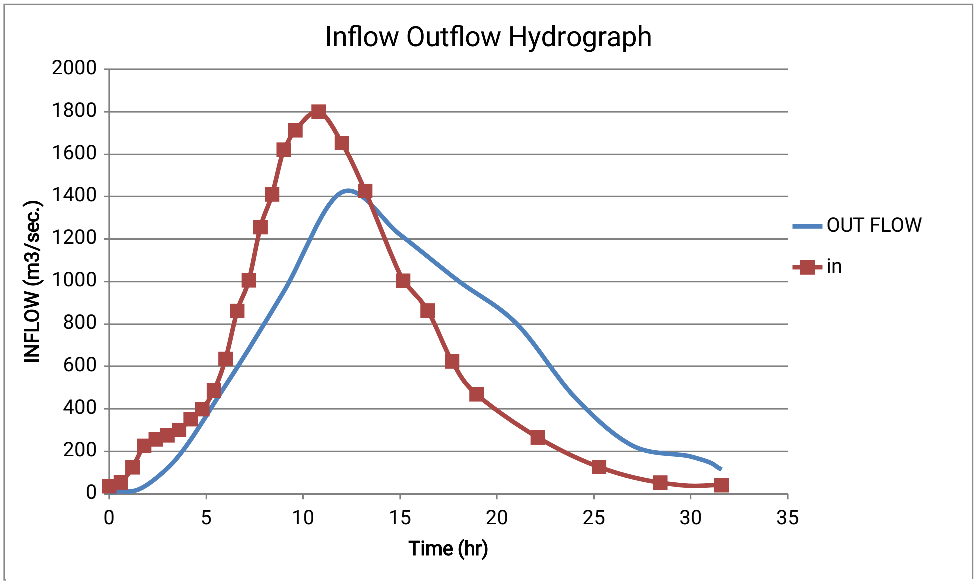

Figure5. 2 Inflow-Outflow hydrograph

5.4 Types of Spillways

The spillways can be classified into different types based on the various criteria as below.

A- Based on purposes

B- Based on control

C- Based on the pertinent features (or hydraulic criteria)

a. Free over fall or straight drop spillway
b. Over flow or ogee spillway
c. Chute or open channel or trough spillway
d. Side channel spillway
e. Siphon spillway
f. Shaft or morning glory spillway
g. Conduit or tunnel spillway

5.4.1 Selection of Spillway Site and Type

The selection of the most suitable type of spillway for a particular reservoir is mainly depends upon the type of dam, topography which surrounds the reservoir and economical considerations of the scheme.

The safe discharge of flood water through dam is very essential, and for that suitable site for spillway should be available.

A concrete dam requires large spillway capacity and there is no suitable site to construct the spillway away from the dam, an over flow spillway type would be the best choice. Therefore, an over flow spillway which should be constructed at the middle of the concrete gravity dam for GD-6 hydroelectric power should be selected.

5.4.2 Ogee or Over Flow Spillway

Over flow spillway is by far the most widely adopted.

It is mainly used on masonry or concrete gravity dam, and if used with earth fill need a separate concrete structure.

An over flow spillway is an improvement up on the free over fall spillway. The essential difference between the free over fall spillway and the over flow spillway is that in the case the former the water flowing over the crest of the spill way drops as free jet clearly away from the downstream face of the spillway, while in the case of later the water is guided smoothly over the crest of the spillway and is made to glide over the downstream face of the spillway.

5.4.3. Crest Shape of Overflow Spillway

The shape of the crest or the upper curve of the ogee profile of this spillway is made to conform closely to the profile of the lower surface of the nape (or lower nape) or sheet of water flowing over a ventilated sharp crested weir when discharging at head equal to the design head of the spillway.

At the design head $H = H_d$, the water flowing over the crest of the spillway will remain in contact with the surface of spillway as it glides over it and optimum discharge will occur.

At a head greater than the design head ($H > Hd$) the nape trajectory is higher than the crest profile, and the over flowing water tends to break contact with spill way surface and zone of separation will be formed in which negative or suction pressure will be produced.

5.4.4 Design of Crest Profile of Ogee Spillway

The shape of the nape shaped profile depends upon the head over the crest, the inclination of the upstream face of the spillway and the height of the spillway above the stream bed or the bed of the entrance channel.

Several standard ogee shapes have been developed by U.S Army Corps of Engineers at their Waterways Experimental Station. Such shapes are known as WES standard spillway shapes.

5.4.4.1 Discharge of Over Flow Spillway

The discharge over and over flow spillway is given by

$Q = CL_eH^{1.5}$ Where; Q = design discharge of an ogee spillway, m3/s

C = is dimension less coefficient of discharge which depends on shape of the ogee profile depth of approach head differing from design head u/s face slope d/s apron interference and d/s submergence

Le = is the effective length of crest, m

He is the actual total head over the spillway crest including that due to velocity of approach,(m)

He = Hd + Ha

Hd = design head, m

Ha = head due to velocity of approach, m

When the length of spillway decreases, height of out flow above the spillway increases. This results an increase of dam height which in turn increases the dam cost. Moreover, it will increase the submergence area by the reservoir and also increases the energy which is going to dissipated. On the contrary when the length of the spillway increases it will make the design of spillway costly. Taking into account the aforementioned merits and demerits, a spillway length (L) of 50m is selected for the design.

Peak flow, Q_d =379.5m^3/s (from flood routing)

By assuming:

Thickness of pier (t) = 1.m

L=50m

Number of piers (N) = 4

Number of span=5

Assuming that 90% cut water nose piers and rounded abutments shall be provided.

Kp =0.01 (coefficient of pier)

Ka=0.1 (coefficient of apartment)

Let L=Le=50m Where, Le = L'-2(KP*N+Ka)*He

$Q = C*Le*He^{3/2}$

Let us first work the appropriate value of He for a value of Le=L=clear water way =50m

Therefore $379.5 m^3/s = 2.2*50*He^{1.5}$

Hd = He = 6.41m (first trial)

The height of spillway above river bed level=95m

Since $\frac{H}{Hd} = \frac{95}{6.41}$ = 14.82> 1.33 it is a high spillway, the effect of velocity head can be neglected.

Since $\frac{H + Hd}{He} = \frac{95 + 6.41}{6.41}$ = 15.82 > 1.7, the discharge coefficient is not affected by tail water condition, and the spillway remains a high spillway.

Effective length of spillway (Le) can now be worked as:

Le = L'-2(KP*N+Ka)*He

Le=50 -2(0.01*4+0.1)*6.41=49.47

$379.5=2.2*49.47*He^{1.5}$

He=6.511 (Second trial)

Le=50-2(0.01*4+0.1)* 6.51 = 49.466 m

$379.5=2.2*49.466 *He^{1.5}$

He=6.512m (Third trial)

From the above trials; the second and third trials of He value is almost the same (6.511≈6.512m). Therefore H_e value is taken as 6.51m

5.4.4.2 Head Due to Velocity of Approach (Ha)

The velocity head (Ha) can be calculated as follows:

Velocity approach (Va)

$$Va = \frac{Q}{(Le + N * t) * (H + Hd)} = \frac{379.5}{(49.466 + 4 * 1) * (95 + 6.51)} = \frac{379.5}{5427.33} = 0.0699 m/s$$

$$\text{Ha = velocity head Ha} = \frac{Va\hat{}2}{2 * g} == \frac{0.0699^2}{2 * 9.81} = 0.00025m$$

This is very small and therefore neglected.

He=Hd =6.51m

$Q_{CAL}=C*Le*He^{1.5}$

$Q_{CAL}=2.2*49.466*6.51^{1.5}=1807.5\ m^3/s > Q_{peak}$ ...OK.

5.4.4.3 Downstream Profile

According to the WES (water ways experimental station) standards spillway shapes, the downstream crest profile of an ogee spillway is given by the equation:

$X^n=k*Hd^{(n-1)}*Y$

Where, (x, y) are the coordinate of points on crest profile with the origin at highest point of the crest, is called apex.

Hd is the design head including velocity head.

K and n are constants depending upon the slope of the upstream face, it is tabulated as follow. The values of X and Y are taken as positive to words the downstream and in the down direction respectively. Hence the equation of downstream profile is applicable only for positive values from the region of other coordinate.

The profile of the spillway is reverse S-shaped and the curved profile of the crest section is continued till it meet tangentially the straight sloping surface of the downstream face of the over flow dam.

Table 5. 3 Upstream Slop of Spillway

Slope of u/s face of spillway	K	N
Vertical	2	1.85
1H:3V	1.936	1.836
1H:1.5V	1.936	1.81

According to the latest study of U.S Army corps, we select a vertical slope u/s face spillway. The d/s crest profile is given by:

K = 2 and n = 1.85

$X^{1.85}=2H_d^{0.85}*Y$ OR $Y=\dfrac{X^{1.85}}{2\ *6.51}=\dfrac{X^{1.85}}{13.02}$

$Y=0.077x^{1.85}$

This curved profile of the crest section is continued till it meets tangentially the straight sloping surface of the downstream face of an overflow dam. Before determining the various coordinates of the downstream profile, the point of tangency should be determined. So that, by adopting the slope which conform to the profile of the downstream face of an overflow dam as 0.85H: 1v, the point of tangency is determined by differentiating equation with respect to x, i.e.

Hence, $\frac{dy}{dx} = \frac{1}{0.85}$

Differentiating the equation of the d/s profile w.r.t x, we get

$$\frac{dy}{dx} = \frac{1.85X^{0.85}}{13.02} = \frac{1}{0.85}$$

$$= X^{0.85} = \frac{13.02}{0.85 \; {*}1.85} = 8.28$$

x=12m

Therefore, $y=\frac{(12)^{1.85}}{9.198}$ =7.65m

The curved profile of the downstream determined by the equation

$Y=0.077x^{1.85}$

Table 5. 4 Downstream Profile of Ogee Spillway

X	1	2	3	4	5	6	7	8	9	10	11	12
Y	0.08	0.28	0.59	1.00	1.51	2.12	2.82	3.61	4.49	5.45	6.50	7.64

5.4.4.4 Upstream Profile

The upstream profile for an ogee spillway, having downstream and upstream slope can be determined on the basis of its WES profile in terms of the design head H *d*

That is for vertical upstream face (assuming it is vertical)

R1=0.5Hd

R2=0.2Hd

a=0.175Hd

b=0.282Hd

by taking $H_{d} =$ 6.51m

R1=0.5*6.51=3.255m

R2=0.2*6.51=1.302m

a=0.175*6.51=1.139m

b=0.282*6.51=1.836m

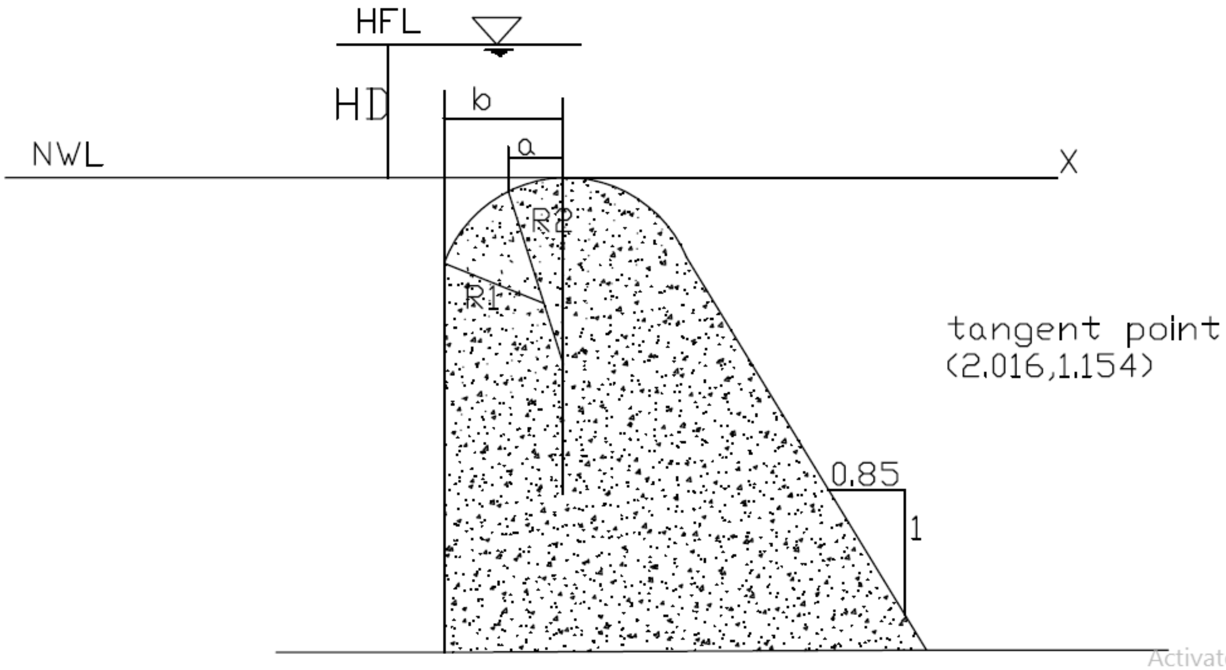

Figure5. 3 Upstream Profile of Spillway

5.5 Energy Dissipater

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway due to the conversion of potential energy into kinetic energy. If arrangements are not made to dissipate this huge kinetic energy of water, if the velocity of the water is not reduced, large scale scour can take place on the downstream side near the toe of the dam and away from it. If the scour is not properly controlled, it may extend backward and may endanger the spillway and the dam.

In order to protect the channel bed against scour the kinetic energy of the water should be dissipated using energy dissipating devices before it is discharged into the downstream channel.

For the dissipation of the excess kinetic energy possessed by the water two common methods are adopted. These are:

- By converting the supercritical flow into sub critical flow by hydraulic jump and
- By using different types of buckets, i.e. by directing the flow of water into air and then making it falls away from the toe of the structure.

The choice of energy dissipating device at a particular spillway is governed by the tail water depth and the characteristics of the hydraulic jumps, if formed at the toe.

If the tail water depth at the site is not approximately equal to that required for a perfect hydraulic jump, a bucket type energy dissipating device is usually provided.

5.5.1 Hydraulic Jump Production

The hydraulic jump that occurs at the stilling basin depends upon

- discharge entering to the basin
- critical depth of flow

Basically, hydraulic jump can form in a horizontal rectangular channel when the following relation is satisfied between pre jump depth (y_1) and post jump depth (y_2)

$$Y_2 = \frac{Y_1}{2}\left(-1 + \sqrt{1 + 8Fr^2}\right)$$

Here, the point is, neglecting any loss of energy and the head due to velocity approach and applying Bernoulli's theorem between upstream water surface and at toe section, respectively, it is possible to determine the total kinetic energy at downstream for section due to potential energy at upstream water surface.

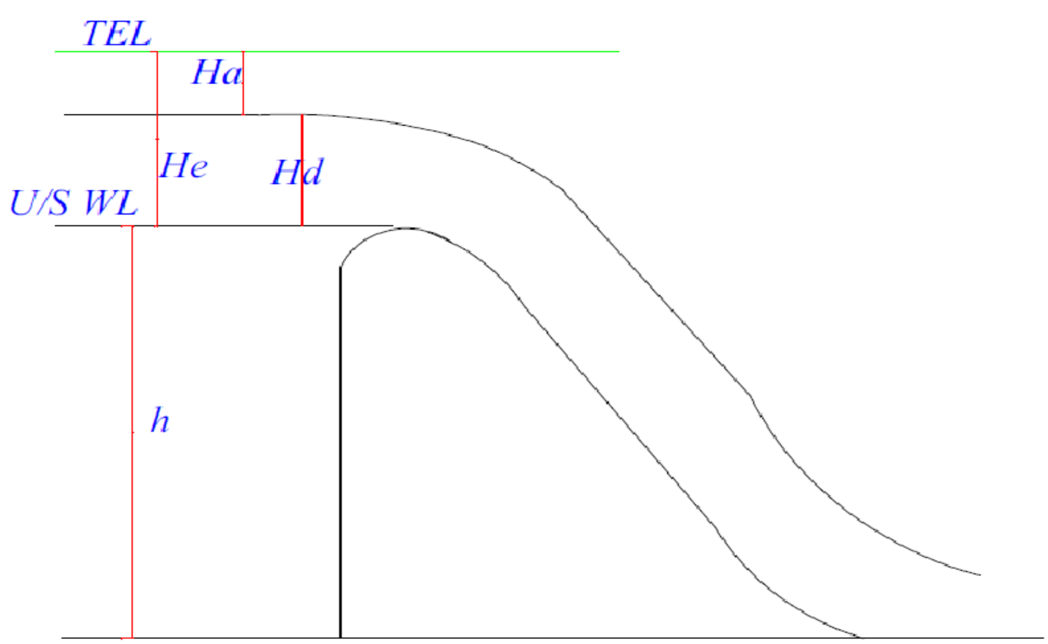

Figure5. 4 Spillway Profile

The total energy level, TEL at point A on the upstream side is calculated as:

TELA=h + Hd + Ha, h = 95m,

Hd= 6.51m, Ha = 0

TELA=h + Hd + Ha = 95+6.52 = 101.51m

Where: $h + H_d = \frac{V1^2}{2g} + y_1$

$$95 + 6.51 = \frac{V1^2}{2g} + y_1 \ldots\ldots\ldots\ldots\ldots * $$

$Q = 379.5 m^3/s.$

Le = 50m

$H_d = 6.51m$

Then, $q = \frac{Q}{L_e} = \frac{379.5}{50} = 7.59$

$q = v_1 * y_1 = v_2 y_2$

$v_1 = \frac{q}{y_1} = \frac{7.59}{y_1}$

From.......*

$$101.51 = \frac{\left(\frac{7.59}{y_1}\right)^2}{2g} + y1$$

$101.51 = \frac{2.936}{y1^2} + y_1$

by trial and error

$y_1 = 0.54m$

$V1 = \frac{q}{y_1} = \frac{7.59}{0.54} = 14.055\ m^3/sec.$

$Fr = \frac{V_1}{\sqrt{g * y_1}} = \frac{14.055}{\sqrt{9.81 * 0.54}} = 12.9$

$y_2 = \frac{y_1}{2}[-1 + \sqrt{1 + 8Fr^2}]$

$y_2 = \frac{0.54}{2}[-1 + \sqrt{1 + 8 * 12.9^2}]$

= 9.58m

- y_1 = 0.54m
- y_2 = 9.58
- Fr = 12.9
- q = 7.59m^3/s/m
- Q = 379.59m^3/s

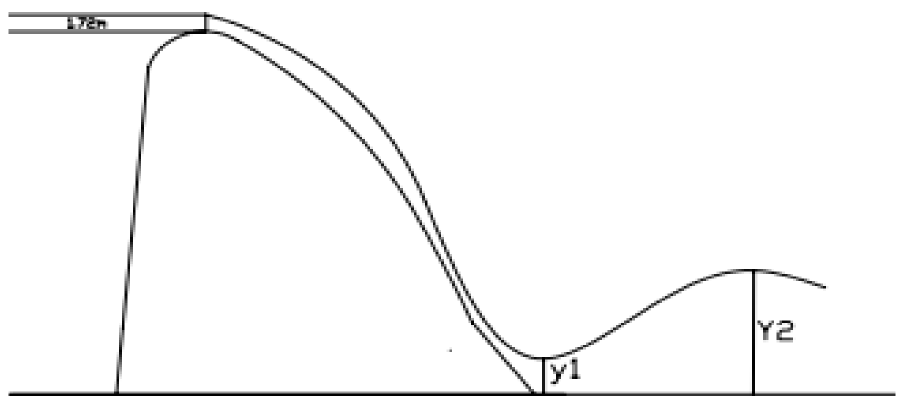

Figure5. 5 Hydraulic Jump Formation at Toe

Bucket type energy dissipaters are usually of small size and more economical than the conventional hydraulic jump stilling basins especially when the Froude number Fr1 exceeds 10, because in that case the difference between the initial depth and the sequent depth is quite large and a very long and deep stilling basin is required. Bucket type energy dissipater can also be adopted for all tail water conditions and are commonly used for dissipation of energy below the over flow spillway. Thus bucket type is selected for Genale Dawa hydropower project.

The bucket type energy dissipaters are usually of the following three types

1. Solid roller bucket
2. Slotted roller bucket
3. Sky jump bucket

For Genale Dawa hydropower project a solid rolled bucket type is selected because it has a better performance than the rest types of buckets when deeply submerged in tail water.

The radius of the bucket is taken as,

$$R = 0.6 * \sqrt{H' * H_d}$$

Where H'=the height of the fall from the crest of the spillway to the bucket invert = 95m

Hd= 6.51m

$R = 0.6 * \sqrt{95 * 6.51} = 22.21$

The bucket lip usually has an exit angle of 45^0 and the height of the lip is about 0.6R=13.325.

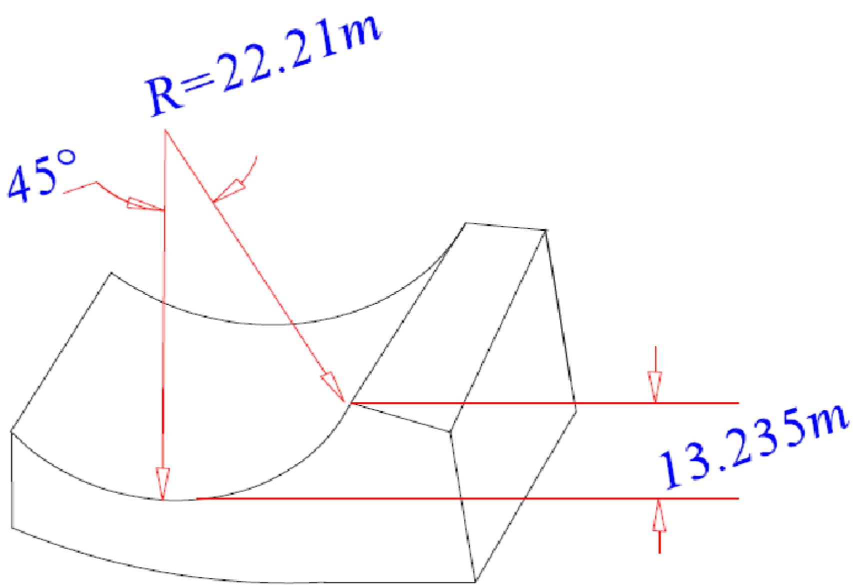

Figure5. 6 Solid Roller Buckets

At the downstream of the dam in order to prevent scouring due to kinetic energy of water from ogee spillway, we provided bucket type energy dissipater with radius 22.21m 13.235m lip height and exit angle 45°.

CHAPTER SIX

6. ENVIRONMENTAL IMPACT ASSESSMENT

6.1 General

The implementation of a hydropower project will bring about many changes (impacts) to the natural and socio-economic environment in which it is set. Some changes will be negative and others positive. In addition each negative impact will vary in the extent to which it can be enhanced.

This section sets out to give an insight to how environmental issues relate in broad terms to hydropower projects. Attempts are made to sort out elements and dimensions of hydropower projects as they relate to an environmental effect and to look at mitigation factors.

The objective of Environmental Impact Assessment is to bring hydropower planning more inline with today's sustainable development concepts and to contribute to improve over all resources utilization in hydropower and water resource development through protective environmental planning ideas.

The principal source of environmental impacts in a hydroelectric project is the construction and operation of a dam and reservoir. In the environmental point of view, hydropower projects by far represent the most appropriate source of power when properly planed and managed, it is relatively clean and renewable source of energy in terms of effects on the environmental(no extraction or burning of fuels) and doesn't use up foreign exchange reserves.

The construction and operation of a dam and hydropower scheme on the Genale River will result in impacts in both the upstream and the downstream. Some of these impacts will be significant from an eco- logical or socio-economic perspective, and some will not be. The de- limitation of impact zones encompassing the project area and the study area is explained following.

Direct Impact Zone (DIZ)

Inundation of the valley behind the dams and the associated works would create a direct impact zone with the following elements:

A core area always under water;

The drawdown zone, around the permanently flooded area;

A short altered river section between dam and tailrace outlet;

The shore and periphery of the reservoir; and

Sites of ancillary works, access roads and transmission lines

Secondary Impact Zone

Secondary Impact Zone (SIZ) encompasses the areas adjacent to the DIZ/reservoir, along access routes and power transmission corridor, which will be effected by the project and people immediately down- stream of the development. This zone includes communities which, although not physically displaced, come into direct contact with the development activities and staff. These communities may rely on resources within the DIZ.

Tertiary Impact Zone

Tertiary Impact Zone (TIZ), which encompasses possible issues upstream and downstream due to the changes in river flow regime and dam wall barrier effects.

Given the extensive study area significant use of satellite imagery and GIS was made in the EIA, to both describe existing baseline conditions and to analyses potential change.

6.2 Baseline Condition

From the perspective of the EIA, critical issues are the land take for the dam, reservoir and access routes and the changes to the downstream river flow regime that will occur in the reservoir filling and operational phase.

The project area straddles the Somali and Oromia Regions with the Oromia Zone of Bale (Mede Welabu Wereda) on the left bank and the Somali Zone of Liben (Liben Wereda) on the right bank. Upstream much of the Genale catchment between GD-3 and GD-6 falls within the Oromia Zone of Bore (Liben Werede). The two Zone Capitals are Negele in Bore Zone and Filtu in the Liben Zone. Road access to site will start at Siru which is located on the main road between Negele and Filtu.

The Genale River falls into the Somali-Masai biome and is influenced by both the Somali-Masai and Afro Tropical Highland Biomes. In a regional context, the vegetation of the study area falls under White's (1983) Somalia-Masai regional Centre of endemism. The length and severity of drought and its effect on the natural vegetation and pas- total populations is different in various parts of this vegetation type. Acacia – Commiphora deciduous bush land is dominant.

The main road between Negele and Filtu is largely settled and wildlife populations are minimal. The river, though, is largely unaffected by development and hosts substantial animal diversity. The project area does not fall into any of the formal protected zones in Ethiopia nor is there informal protection although there is an understanding at the Kebele level that communities are responsible for conservation of wildlife.

The GD-6 Genale River basin is approximately 2,920 km2. The land use varies considerably from intense agriculture to open live- stock production systems. An assessment of the present land use in the basin indicates that approximately 62.5% of the basin is used for open livestock production a further 19.7% has a mixture of arable and livestock with emphasis on livestock and 17.8 is intense arable agriculture. Due to the low and erratic rainfall the primary form of farming is pastoralism (cattle, sheep, goats, camels) with smaller areas of arable agriculture. The most common crops are; maize, wheat, lowland teff, haricot-beans and sorghum.

There are two kebeles (of the Liben Werede) that are affected by the project (by flooding of the reservoir and reduced flows in the section between the dam wall and the tailrace outlet),

These are;-

1 Siru (upstream of the dam wall), and

2 Haydimtu (mainly in the area below the dam wall)

In general land use activities of the project area along the Genale River relate to response to drought conditions in which grazing and water resources in the Dawa river area are depleted. Both kebeles focus their arable agriculture and pastoralism around the Dawa River to the south.

Somali Liben Zone Socio-economic Characteristics

The main socio-economic linkages relevant to the GD-6 project are with the Somali National Regional State (SNRS) and its administrative units on the right bank (taken as looking downstream) of the Genale River, focusing on the reservoir area. On the left bank in Oromia there is an absence of settlement in and around the project area and land use appears to be limited and as an extension of activities originating from the right bank.

Somalis are mainly pastoralists and agro-pastoralists. The nomadic or transhumance way of life of pastoralists responds to highly unstable natural environment and to a future-oriented perception of time and space. However, although with arid and semi-arid rangelands pastor- alism may be thought of as the only sustainable economic system in the Somali region, pastoral households are recently becoming more and more attached to semi-permanent settlements in the vicinity of watering points. This may be due to different factors, such as intensive water development, constraints to mobility, commercialization and migration of labor.

Diode is the domination clan in the area and is strongly linked by lineage to the clans of Hawiya, the Hawazle of Hiran, and the Ra- hawein of Baay, Minor ethnic groups such as the Gerri from Hudet and Moyale, and the Maherran from Chisimaio populate the surrounding of Filtu. Clan uniformity and homogeneity are evident signs of a rigid and impermeable social structure, with strict forms of allegiance and intra/inter-group solidarity. In times of crisis, wealth redistribution, sharing of food resources, and assets concession are all coping mechanisms involving relatives and neighbors.

Patriarchal rules govern social and gender relations, according to a hierarchical power structure. Among the Diode, land is usually ad- ministered by the elders of the village, who formulate rules about re- source use and distribution according to the Koranic law. Enclosure is however expanding, and many farmers cultivate their own land, family-inherited plot of land irrespective of the possessions of their clan.

Generally speaking, land tenure rules and access to and control over resources vary from clan to clan, but land is usually a possession of the head of the household, commonly a man. Social

And gender division of labor is also defined in relation to the economic needs of the family and its practices, whether pastoralist or agro-pastoralism .

6.3 Positive Impacts

a) Generation of electric power

The main objective of the development of a hydropower project is production of electric power which can support economic development and improve the quality of life in the service area. Moreover, the generation of hydropower may also help to prevent deforestation to an extent and replace the unsustainable offtake of woody biomass with electricity and also provides an alternative to the burning of fossil fuels which allow the power demand without producing heated water, air emission and ash.

b) Employment

During the construction and implementation of the project there will be job opportunity for the local population and other skilled labors.

Other benefits are river regulation, flood control and other infrastructures for the local population either that of rural or for Woreda's town alter the economic, social and political life of the population in the area by encouraging the establishment of government offices and associated services, health and other services, commerce other Agro-industrial centers and enable individual house hold to have access to facilities which was formerly not existing in the area.

6.4 Negative Impacts

6.4.1 Physical Impacts

The implementation of a hydropower project affects some environmental conditions such as water quality, ground water; microclimate, dam breach and transport of nutrients etc. impacts on these conditions are termed as physical impacts. Under this topic the first four will be presented.

i) Landscape Impact:-Due to the increment of civilization most rural countries land loss a considerable amount of land which have a long term historical importance to the public. Most of the time aesthetical aspects of the environment have a remarkable significance to the public. Most of the components of hydroelectric plants such as penstocks, tunnels, power house, dam spillway, intake, tail race, transmission and distribution lines have a capacity to alter the visual impact of the site by introducing constructing forms, colors, lines, and textures etc. on top of hilly areas, both the mountains and valleys are necessarily be affected in the vicinity. There are people who eagerly appreciate the topography and don't allow a change in the environment such as landscape. The design location and appearance of any one feature may well determine the level of public acceptance for the entire scheme. But most of the time the landscape impacts are likely to be positive. For GD-6 Hydropower project the project area project site including the dam and pond area, which has no any significant historic importance to the public as we have seen from the topographic map. Therefore, the project has no that much physical impact.

ii) Water quality:-the quality of natural water varies considerably and this contributes to the diversity of species and habitats found in fresh water. When human activities modify the chemical environment to the extent that it strays outside the natural range for the river damage to a fresh water community is most likely to occur. Physical changes such as, vegetation clearance, leads to greater erosion which will cause a change in water quality by increasing the sediment in flow and turbidity.

iii) Ground water: - the change and variation of the water level by the reservoir operation will change the ground water in the surrounding area.

The changed flow on the downstream water course may also lead to ground water changes. The change in ground water may influence the supply of drinking water, affect vulnerable wet lands, or lead to water logging of arable land. Therefore special attention should pay to changes in the ground water level.

iv) Socio-Economic Environment:-Since the land is fertile and rain is relatively abundant, people in the study area take farming as the main means of their economy. The main crops produced in the area are Teff, Sorghum, Maize, Wheat, Barley and different perennial crops.

6.4.2 Biological Impact

i) Vegetation:-the flat impervious layer of basalt creates saturated vegetation which contains woody plant species. The project area is surrounded by evergreen rain forest

ii) Wild life:-mammals, reptiles and birds are animals that live in the forest around the project site.

6.5 Potential Environmental Impact and Corresponding Mitigation measures.

6.5.1 Bio-Physical Environment

i) Terrestrial Ecology:-there will be significant deforestation for the reservoir areas during the construction of access roads, dams, the power plant and camp, due to the destruction of forest around. This adverse clearance of forest disturbed the natural habitat of wild life and leads the larger smaller mammals to displace to adjacent area of similar or identical habitat. This impact can be reduced to an acceptable level through a combination of best practice during constructing sensitive micro settings of project components and holistic environmental planning. This includes measures taken to make the design optimum, afforestation practice on some other area.

ii) Aquatic ecology:-The aquatic ecology faces some significant adverse impacts. During the construction phase the increased rate of soil erosion which increase the inevitable sediment load, disturbs the quality of water of water. The risk of chemical and organic pollution from accidental spillages may be occurring. As a mitigation measure appreciable minimum water release measures are implemented and it is likely that negative impacts can be held to an acceptable level

6.5.2 Human Environment

The socio-economic environmental impacts associated with the displacement and involuntary resettlement of people and the loss of agricultural land. There may be displacement of people along the transmission line alignment, which results in loss of properties such as land and vegetation. Mitigation is possible through resettlement gain measures, all of which must be developed through community consultation and participation. The other impact is during the construction phase associated with a large migrant labor force. This will lead to an increase of pressure on the infrastructures such as water supply, sanitation, transport etc and a huge amount of ecosystem exploitation and biodiversity may occur in the area. this may be mitigated either by providing the same or equivalent development project for the adjacent population so as to reduce migration of people to seek for new facilities or up grading the capacity existing infrastructures in the developing project area so as to satisfy the future population demand.

6.6 Impact of Construction Activity

In order to select solutions that minimize latter mitigation actions and maximize the ability of permanent constructions to meet uncertainties in the environmental situation, the designer must identify the potential problems at the early possible stage. Unique cultural or ecosystem features should be threatened by preferred dam, spillway or diversion arrangements. During the construction of the dam lots of activities such as undesirable noises, air pollution due to the vehicle and machineries and some others should be properly considered and adequately responded by the design and construction engineers. Also it affects the flora and fauna, causes soil and water pollution and health problem.

Figure 6. 1 Impacts and Mitigation Measure

Potential negative impacts	Mitigation measure
1.negative environmental effects of constructions - air and water pollution from construction and waste disposal - soil erosion - destruction of vegetation's - Sanitary and health problems from construction camps.	- Air and water pollution control. - Careful location of comps, buildings, - Borrow pits, quarries and soil and disposal sites. -Precautions to minimize erosion. - Land reclamation.
2. Dislocation of people living in inundation zone.	Relocation of people to suitable area. - Provision of compensation in kind for resource lost. - Provision of adequate health services, infrastructure and employment opportunities.
3. Lose of land (agricultural, forest, range, wet lands) by inundation to form reservoir.	-Sitting of dam to decrease losses. -Decrease of dam and reservoir size -Protection of equal areas in region to offset losses -Creation of usable land in previously unusable areas to offset losses.
4. Loss of wild lands and wild life	Setting of dam or decease of reservoir size to avoid / minimize loss.

habitat.	Establishments of compensatory parks or reservoir areas. Animals rescue and relocation
5. Proliferation of aquatic weeds in reservoir and dam stream impairing dam discharge and fisheries and increasing water loss through transpiration.	-clearance of wood vegetation from inundation zone prior to flood eng (nutrient removal) -weed control measure Harvesting of weeds for compost, folder of biogas. -Regulation of water discharge and manipulation weed growth.

CHAPTER SEVEN

7. CONCLUSION AND RECOMMENDATION

7.1 Conclusion

It is evident that power is a crucial factor for the overall development of a country.

For, developing countries like Ethiopia which has a high potential of waterpower, more priority should be given to the implementation of the national economy.

These and the likes can support the government objectives to enhance economic development and improve the living standard of the people.

In order to alleviate the increasing demand of electric energy, the development of hydropower projects like GD-6 is important in addition to the existing ones.

The dam is RCC dam type designed to a height of 95m having a reservoir capacity of 710.627Mm^3.

Implementation of monetary and mitigation programs are suggested to avoid the environmental impact of the project area.

7.2 Recommendation

Based on what we have during our work, we have recommended the following points for the concerned bodies.

During construction of such a huge project, care should be given for the resettlement of the people from the dam area. And an equivalent compensation should be given for them.

As the people living around the dam site are going to face problems of water borne diseases such as malaria, which may or may not be there before, immediate health centers should have to be built.

Dam monitoring should be planned properly. Instruments must be placed at appropriate points around the dam and the reservoir for checking the behavior of the dam and the reservoir area.

Senior and experienced personnel with in a responsible organization should handle, planning and commissioning of the instrumentations.

We have faced lots of problem in searching of relevant data; therefore we would like to recommend the department to prepare the necessary data for the future that helps the next batches to perform their projects effectively within the given time.

The beginning time for the project should be within the correct time and available data and the department should prepare the site visit.

REFERANCES

1. ARORA Dr.K.R, Irrigation, Water Power and Water Resource Engineering, Standard Publishers distributors, Delhi, 2002.
2. Chow Vent. Hand book of applied hydrology, Nework: Mc Graw hill 1984.
3. Dandekar, M.M and K.N Sharma .Water power engeenering.New Delhi Vikas publishing, 1997
4. Dr.P.K.Bhattacharya, Water Power Engineering, Khanna Publishers, Delhi, 2003.
5. GARG.S.K. Irrigation Engineering and Hydraulic Structure, Khanna Publishers, Delhi, 2003.
6. Linsley Ray K, Max A.Kohler, and Joseph L.H Paulhus Hydrology for engineer's .Tokyo; Mc Graw Hill; 1982
7. Mutreja, K.N.Applied hydrology, New Delhi: Tata Mc Graw Hill; 1986
8. Novak, P.A.I.B Moffit, C.Nallury,and R.Narayanan, Hydraulic structure
9. Punumia Dr. B.C, Irrigation and Water Power Engineering, Laximi, Publication LTD, New Delhi, 2001.
10. Subramanya K. Engineering Hydrology; New Delhi, Tata Mc Graw Hill, 2000

ANNEXES

ANNEX- 1 Table of Kn Value for Different Sample Size (Chow, 1983)

sample size n	Kn	sample size n	Kn
10	2.036	37	2.65
11	2.088	38	2.661
12	2.134	39	2.671
13	2.175	40	2.682
14	2.213	41	2.692
15	2.247	42	2.7
16	2.279	43	2.71
17	2.309	44	2.719
18	2.335	45	2.727
19	2.361	46	2.736
20	2.385	47	2.744
21	2.408	48	2.753
22	2.429	49	2.76
23	2.448	50	2.768
24	2.467	55	2.804
25	2.486	60	2.837
26	2.502	65	2.866
27	2.519	70	2.893
28	2.534	75	2.917
29	2.549	80	2.94
30	**2.563**	85	2.961
31	2.577	90	2.981
32	2.591	95	3
33	2.604	100	3.017
34	2.616	110	3.049
35	2.628	120	3.078

36	2.639	130	3.104
		140	3.129

ANNEX-2 L-Moment Parameter Calculation

S.No. (J)	Qmean (Xj)	$\frac{((n-j)*X_j)}{n-1}$	$\frac{(n-j)(n-j-1)*X_j)}{(n(n-1)(n-2)}$	$\frac{((n-j)(n-j-1)(n-j-2)*X_j)}{(n(n-1)(n-2)(n-3)}$
1	174.473	5.816	5.816	5.816
2	163.398	5.259	5.071	4.883
3	158.493	4.919	4.567	4.229
4	158.322	4.731	4.225	3.755
5	158.114	4.544	3.894	3.317
6	156.851	4.327	3.554	2.896
7	153.012	4.045	3.178	2.472
8	152.018	3.844	2.883	2.136
9	150.890	3.642	2.602	1.831
10	149.447	3.436	2.331	1.554
11	141.917	3.099	1.992	1.254
12	139.390	2.884	1.751	1.038
13	139.117	2.718	1.553	0.863
14	135.959	2.500	1.339	0.695
15	134.200	2.314	1.157	0.557
16	131.706	2.119	0.984	0.437
17	125.168	1.870	0.802	0.327
18	124.376	1.716	0.674	0.250
19	124.143	1.570	0.561	0.187
20	114.460	1.316	0.423	0.125

21	114.216	1.182	0.338	0.088
22	113.543	1.044	0.261	0.058
23	112.245	0.903	0.194	0.036
24	111.966	0.772	0.138	0.020
25	107.465	0.618	0.088	0.010
26	102.697	0.472	0.051	0.004
27	101.693	0.351	0.025	0.001
28	91.222	0.210	0.007	0.000
29	79.529	0.091	0.000	0.000
30	54.210	0.000	0.000	0.000
Bo=129.141		*b1= 72.311*	50.459 = *b2*	38.838 = *b3*

ANNEX-3 Load and Moment Analysis of Dam

s.No	Force designaatio n	Description	Magnitude of force				Moment arm about toe (m)	Moment about the toe	
			Vertical		Horiontal				
		↑ −	↓ +	← −	→ +			−	+
1		self wt							
	pm1	6*95*24		13680			75		1026000
	pm2	0.5*72*46.63*24		402889			48		1933867 2
2		Water load							
		i) at maximum							
		flood level							
	Ph1	10*91^2)/2				41405	30.33		1255814
		ii) at normal pool							
		level							
	Ph2	10*86^2)/2				36980	28.67		1060217

		Tailwater							
	P"h1	0.5*10*6			30		2	60	
	P'v1	0.5*10*7		35			2.333333		81.66667
3		uplift load							
		muximum water level							
		i) without drainage							
	Pu1	10*78*60	46800				39	1825200	
	Pu2	10*0.5*78*910	354900				52	18454800	
		ii) with drainage							
	Pu1	10*78*60	46800				39	1825200	
	Pu2	10*283.33*26	73666.7				65	4788336	
	Pu3	0.5*52*283.33*10	73665.8				34.67	2553993	
	Pu4	0.5*626.67*26*10	3296.28				60.66	199952.3	
		Normal pool level							
		i) without drainage							
	Pu1	10*78*60	46800				39	1825200	
	Pu2	10*0.5*800*78	312000				52	16224000	
		ii) with drainage							
	Pu1	10*78*60	46800				39	1825200	
	Pu2	10*266.67*26	69333.33				65	4506666	
	Pu3	10*0.5*266.67*52	69334.2				34.67	2403817	
	Pu4	10*0.5*533.33*26	69332.9				60.66	4205734	
4		Silt Load							
	Phs	(13.34*45.64^2)/2				13894.7	15.21		211337.8
5		Wave pressure							

	Pw	2*10*5076.625				101531.3	26.36		2676363.8