Amhara National Regional State

East Gojjam Zone

Enarj Enawuga Woreda Irrigation Office

Irrigation Engineer team

Gilgel Cheye Small Scale Irrigation development project

February, 2025

D/Work

EXCUTIVE SUMMERY

In the Ethiopian government development policy, agriculture is leading sector of the economy and given prime attention and considered as source of growth. Accordingly, Amhara Nationa Regional State Bureau of Irrigation and Low Land Area Development (BOILLAD) initiated for study of several small scale irrigation projects at Woreda level of which **Gilgel Cheye** irrigation project is one among those project identified for feasibility study. The study was undertaken by Woreda Experts.

Location: Gilgel Cheye Small Scale Irrigation Project is located in Amhara region East Gojjam zone Enarji Enawuga Woreda in Felege Zachena Kebele. The proposed irrigation project is to be undertaken on Gilgel Cheye River and the headwork structure is located at an altitude of 2566m a.s.l. and UTM geographical coordinates of 11787980N & 399768E.

Hydrology/Watershed Managemen/t: Gilgel Cheye River is Perennial River. During field work in February 10, 2012 the study team measured the base flow as 18.84 l/s and as elder inhabitants confirmed us base flow would not be lower in the other seasons. Gilgel Cheye is un-gauged River and we use the dominant hydrometric station of Debre Work town. The maximum design rainfall data from 36 years is 82.2mm at 1995 EC. Accordingly the design rain fall value is 83.11mm using Gumbel EVI distribution method; i.e. largest value. And from the watershed results the total area of catchment covers about 31.411km^2, longest stream reach 12.33km. Using SCS method of runoff estimation the peak value of discharge (design discharge) is 98.05m^3/sec.

Structural Engineering: Encounter the availability of construction easily, materials, considering the river features, the height of the weir that can dissipate the energy of water and to protect its edge from crushing effect of boulder **broad crest weir type** is selected for Gilgel Cheye small scale irrigation project. So that, it is better for the structure foundation and the risk of seepage through weir can be minimized. The proposed weir is to be constructed by masonry with reinforced concrete capping on the top, U/S and D/S face of the weir body. The weir section has 16m span length and 1.6m height/1.4m masonry & 0.2m reinforced concrete capping/.

In the farm structure the gross command area covers about 24ha to be irrigated by the main canal capacity of 24.72l/sec with 350m lined main canal. And two flume structures are considered in the design document with social, environmental and economical feasibility aspects.

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- ***Name of the project:***
  - *Gilgel Cheye small Scale Irrigation Project*
- ***Location:***
  - *Amhara Region*
  - *East Gojjam Zone*
  - *Enarji Enawuga Woreda*
  - *Felege Zachena Kebele*
  - *Easting: 399768*
  - *Northing: 1187980*
  - *Altitude: 2566m a.s.l.*
- ***Hydrology:***
  - *Design Rain fall: 83.11mm*
  - *Base flow: 19.84 l/s (Measured on February 10, 2012)*
  - *Design Peak flood: 98.05m^3/s*
  - *Catchment area: 31.411Km^2*
- ***Headwork structure:***
  - *Weir type: Broad Crested Weir*
  - *Crest length: 16m*
  - *Height: 1.6m*
- ***Farm Structure:***
  - *Duty: 1.03l/sec/ha*
  - *Project Command area: 24ha.*
  - *Main canal length: 550m*
  - *Main canal design capacity: 24.72l/sec.*
- ***Number of beneficiaries' : 85 households(77male,8female)***
- ***Client: Enarj Enawuga Woreda Irrigation Office***
- ***Budget source SHIDP***

Acronyms

a.m.s.l.	-	*Above Mean Sea Level*
CN	-	*Curve Number*
CWR	-	*Crop Water Requirement*
C/C	-	*Center to Center*
D/S	-	*Downstream*
E	-	*East*
FAO	-	*Food Association Organization*
GIR	-	*Gross Irrigation Requirement*
GIS	-	*Global Information System*
HFL	-	*High Flood Level*
IDD	-	*Irrigation and Drainage Design*
N	-	*North*
NIWR	-	*Net Irrigation Water Requirement*
PC	-	*Primary Canal*
RCC	-	*Reinforced Concrete Cover*
SCS	-	*Soil Conservation Service*
SSIP	-	*Small Scale Irrigation Project*
TC	-	*Tertiary Canal*
Tc	-	*Time of concentration*
TEL	-	*Total Energy Level*
U/s	-	*Upstream*
UTM	-	*Universal Transmission Mercator*

1. INTRODUCTION

1.1 Background

Under the prevalent rain-fed agricultural production system, the progressive degradation of the natural resource base, especially in highly vulnerable areas of the highlands coupled with climate variability have aggravated the incidence of poverty and food insecurity. The major source of growth for Ethiopia is still conceived to be the agriculture sector. Hence, this sector has to be insulated from drought shocks through enhanced utilization of the water resource potential of the country through development of irrigation coupled with strengthened linkages between agriculture and industry (agro-industry).

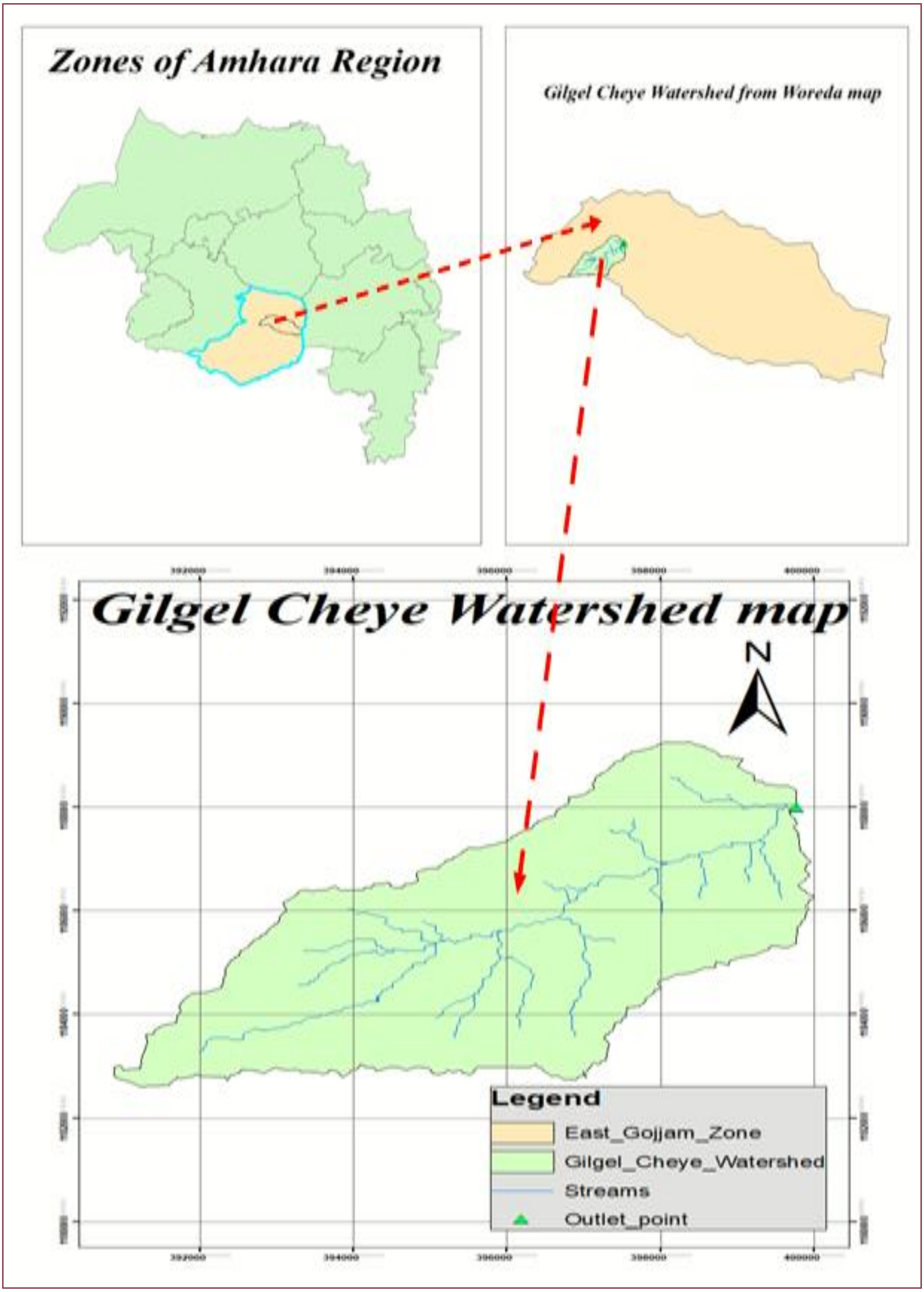
In line with the above, the government exerts its efforts to improve the country position in the area of irrigation for the development of small scale irrigation projects in our region specifically East Gojjam zone, Enarj Enawuga wereda.

As a result the Agriculture sector has initiated the study, design and construction of small scale irrigation scheme project on Felege Zachena Kebele and all the study, design and construction supervision works are process by woreda Irrigation Engineer team experts.

1.2. Description of the Project Area

1.2.1. Location

Gilgel Cheye small scale irrigation project is located mainly at Felege Zachena kebele, Enarj Enawuga Wereda of East Gojjam Zone in the Amhara National Regional State. The proposed irrigation project is to be undertaken on Gilgel Cheye River and the headwork structure is located at an altitude of **2566m a.s.l.** and UTM geographical coordinates of 1187980N & 399768E.


Zones of Amhara Region
Gilgel Cheye Watershed from Woreda map
Gilgel Cheye Watershed map
N
Legend
East_Gojjam_Zone
Gilgel_Cheye_Watershed
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1.2.2. Accessibility

The project area is accessed in one route from Debre Work Town at a distance of around 21 Km. Gilgel Cheye small scale irrigation project site is accessible around 21km asphalt road from Debre work to Felege Berhan route.

1.2.3. Previous Irrigation Practices

There had traditional irrigation practice on the river D/S at a distance of 120m from the proposed headwork site. The traditional irrigation practice below the proposed headwork site is under taken by small holder farmers on the left side traditionally and using generator pump at every year on D/S side. So, the farmers in the project area are interested for the upgrading of the traditional scheme to modern scheme.

1.3. Objectives of the project

1.3.1 Main Objective

Basically, to make feasibility study and detail design by conducting an assessment and investigation of all available water and land resources along Gilgel Cheye River. This main objective is to be realized by constructing diversion structures across the River and diverting the flow in to the irrigable land.

1.3.2. Specific Objectives

The specific objectives of the project are:

- To improve efficient use of water;
- To enhance local food security gains;
- To manage scarcity of natural resources (land and water);
- For the intensification and modernization of small-holder agriculture and rural lifestyles.

➢ To analyze hydrologic requirements of the project and engineering structures;

1.4. Scope of the project

⇒ The irrigation design shall ensure reliability, equity and flexibility of water delivery to the farmers.

⇒ For reducing conflicts among water users and will lead to lower operation and maintenance costs.

⇒ Upgrading the existing traditional diversion system, computation of the crop water requirement, irrigation demand/duty using the existing and recent agronomic, climatologic and soil data using more appropriate methodologies.

⇒ Establish design criteria for irrigation structures to be approved by the client and to be used in the final design stage,

⇒ Design proper irrigation system compatible with local conditions and management capabilities,

⇒ Determination and estimation of water application conveyance and other losses and irrigation efficiencies and consideration of those parameters in design steps.

⇒ Check hydraulic and structural designs of main canal considering total demand and the required capacity and the base flow availability,

⇒ Prepare general plans and drawings for all irrigation infrastructure and irrigation systems designs,

1.5. Methodology

In the study and design procedure, have been used the following steps.

- Specific Site identification:
    - ✓ Review of the reconnaissance survey
    - ✓ GIS information
    - ✓ Local farmers interview and discussion
    - ✓ Woreda and Zone Agriculture sector expertise discussion
    - ✓ Previous studies
- Topographic survey:
    - ✓ Surveying the headwork site and the Command area with sufficient radius, using Total station
- Flow estimation
    - ✓ Physical observation on flood mark indications and local information about high flood and critical flow condition of the river
    - ✓ Analyzing the recorded river flow data and use watershed inputs for further analysis.
    - ✓ Base flow estimated 19.84 l/s during the reconnaissance field visit by floating method.
    - ✓ Irrigable area identification:

2. HYDROLOGY

2.1 Water Resources and Base Flow Measurement

It is tried to plan the size of the irrigable area that the project supports considering the water potential available, D/s utilization allowance and crop water requirement. Daily crop water requirement is estimated 1.33l/s/ha from CROPWAT 8. In critical seasons, the total water requirement for the estimated hectare is found by multiplying the total hectare with demand.

Gilgel Cheye River is Perennial River. During field work in February 10, 2012 the study team measured the base flow as 19.84 l/s and as elder inhabitants confirmed us base flow would not be lower in the other seasons. Therefore, this project is designed to use this potential of water effectively and efficiently to increase the income of the farmers living around the place.

On the U/S of the headwork site there is no modern irrigation scheme, but there is traditional diversion on D/S of the headwork site. Therefore, implementation of this modern irrigation scheme will alleviate unnecessary traditional diversions and improve water utilization of the area. There will not be deficiency of water for the D/S users. We have also checked that, the implementation of Gilgel Cheye Irrigation project has no effect on the downstream users.

⇒ Maximum duty value = 1.03 l/sec/ha

⇒ Irrigation hour, = 18hr

⇒ Over all irrigation efficiency = 56.7%

⇒ Command area = 24ha.

⇒ Water demand based on the available irrigable area = 17.86l/sec, (after releasing 10% for D/S beneficiaries. i.e. 10% of base flow, 19.84l/sec.)

Table 1: Gilgel Cheye river base flow calculation using floating method

DATA SHEET FOR CALCULATION OF GILGEL CHEYE RIVER BASE FLOW USING FLOATING METHOD

Date: February 10, 2012E.C.

	Cross Section Profile 1 (upstream cross section)		Cross Section Profile 2 (middle cross section)		Cross Section Profile 3 (downstream cross section)	
	Width (m)	1.2	**Width (m)**	1	**Width (m)**	1.7
		Water Depth (cm)		**Water Depth (cm)**		**Water Depth (cm)**
		14		17		11
		12		16		9
		13		15		10
Mean depth (m)		**0.130**		**0.160**		**0.100**
Area of Cross section profile (m2)	**0.156**		**0.160**		**0.170**	

FLOAT MEASUREMENTS	
stream length(m)	**10**
Time (seconds)	**51**
	49
	47
Average time over the reach in seconds	**49**
flow velocity (m/s)	**0.20**

CALCULATION OF DISCHARGE (m3/s)	
Average cross sectional area	**0.16200**
Average measured velocity (m/s)	**0.20408**
Velocity coefficient	0.60000
Average channel velocity (m/s)	**0.12245**
Discharge (Flow) (m3/s)	**0.019837**
Discharge (Flow) (l/s)	**19.84**

2.2. Design Rainfall Estimation

2.2.1. Available Rainfall Data

Gilgel Cheye is un-gauged River and we use the dominant hydrometric station of Debre Work town. The maximum design rainfall data from 36 years is 82.2mm after filling missed data on appropriate or nearest stations at 1995 EC.

Table 2: Debre Work Daily Maximum Rainfall in mm

No	Year	Daily MAX Rainfall(mm), X	Descending order	Rank	log value(Y0)	(Y0-Ym)^2	(Y0-Ym)^3
1	1975	53.1	82.2	1	1.915	0.055	0.013
2	1976	46.7	78.5	2	1.895	0.046	0.010
3	1977	45.8	66	3	1.820	0.020	0.003
4	1978	66	65.1	4	1.814	0.018	0.002
5	1979	59.1	61.6	5	1.790	0.012	0.001
6	1980	42.4	59.1	6	1.772	0.008	0.001
7	1981	39.6	58.1	7	1.764	0.007	0.001
8	1982	39.7	57	8	1.756	0.006	0.000
9	1983	47.7	53.4	9	1.728	0.002	0.000
10	1984	42.3	53.2	10	1.726	0.002	0.000
11	1985	46.3	53.1	11	1.725	0.002	0.000
12	1986	38	52.4	12	1.719	0.002	0.000
13	1987	39.5	51.1	13	1.708	0.001	0.000
14	1988	78.5	50.3	14	1.702	0.000	0.000
15	1989	42.4	48.5	15	1.686	0.000	0.000
16	1990	36.4	47.7	16	1.679	0.000	0.000
17	1992	47.1	47.1	17	1.673	0.000	0.000
18	1993	58.1	46.7	18	1.669	0.000	0.000
19	1994	53.4	46.3	19	1.666	0.000	0.000
20	**1995**	**82.2**	46.2	20	1.665	0.000	0.000
21	1996	51.1	45.8	21	1.661	0.000	0.000
22	1997	41	45	22	1.653	0.001	0.000
23	1998	52.4	45	23	1.653	0.001	0.000
24	**1999**	**29.1**	44	24	1.643	0.001	0.000
25	2000	65.1	42.4	25	1.627	0.003	0.000
26	2001	45	42.4	26	1.627	0.003	0.000
27	2002	45	42.3	27	1.626	0.003	0.000
28	2003	46.2	41	28	1.613	0.004	0.000
29	2004	61.6	39.7	29	1.599	0.007	-0.001
30	2005	50.3	39.6	30	1.598	0.007	-0.001
31	2006	53.2	39.5	31	1.597	0.007	-0.001
32	2007	57	38	32	1.580	0.010	-0.001
33	2008	36	36.4	33	1.561	0.014	-0.002
34	2009	35.1	36	34	1.556	0.015	-0.002
35	2010	48.5	35.1	35	1.545	0.018	-0.002
36	2011	44	29.1	36	1.464	0.047	-0.010
Sum			**1764.9**		**60.473**		
Mean			**49.025**		**1.680**		
Standard Deviation			**11.454**		**0.096**		
Skewness coefficient (Cs)					**0.412**	**2.345**	**1.409**

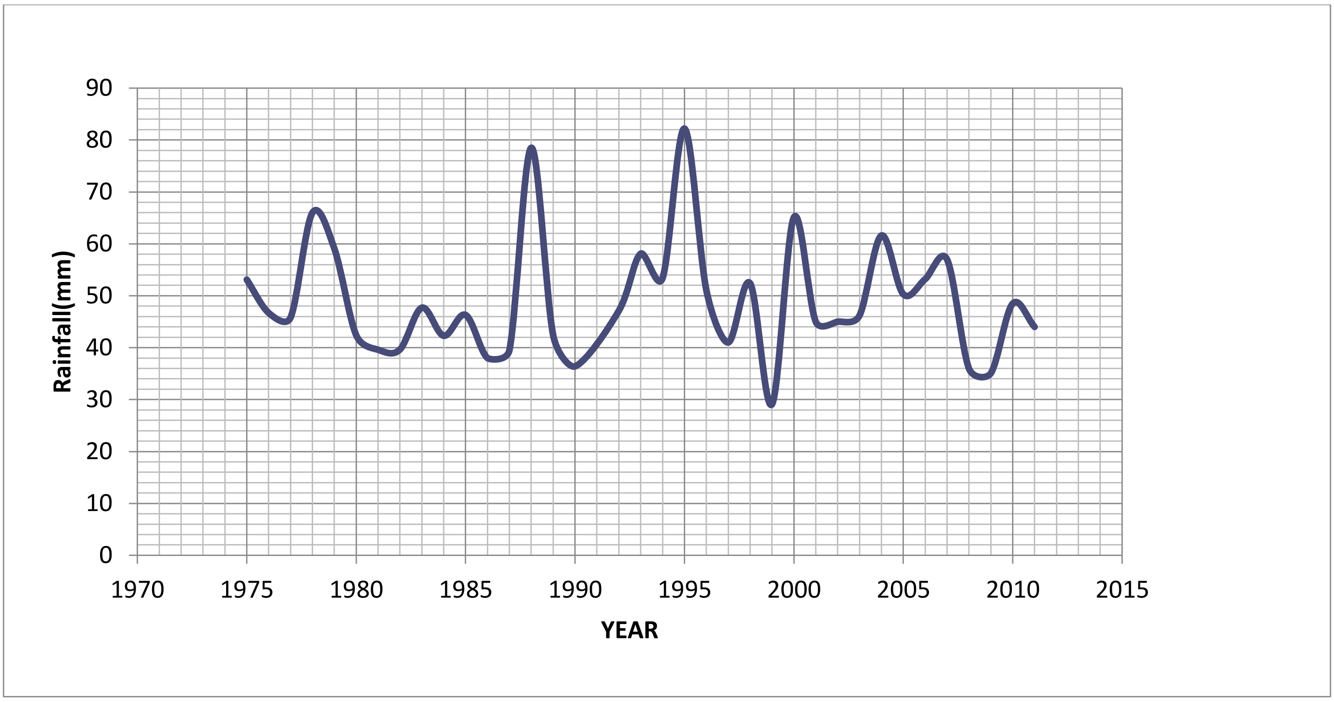


Figure 1: Daily Maximum Rainfall Graph

The following are the sample statistics that are commonly used in fitting distributions for hydrological analysis.

(1) **Sample mean: -** is generally used to represent measures of central tendency.

$$\overline{X} = \frac{\sum_{i=1}^{n} X_i}{n} = \mathbf{49.025}$$ Where; n is the number of sample size

(2) Sample standard deviation: - is the measure of distribution around the central value or mean.

$$S = \left[\frac{\sum_{i=1}^{n}(X_i - \overline{x})^2}{n-1}\right]^{1/2} = \mathbf{11.454}$$

(3) Sample Skew coefficient: - Skew measure the symmetry of a distribution. The lack of symmetry of a distribution is called skewness or asymmetry. The degree of the skewness of the distribution is usually measured by the coefficient of skewness(Cs) and is given by:

$$C_S = \left[\frac{n\sum_{ii=1}^{n}(X_i - \overline{x})^3}{(n-1)(n-2)S^3}\right] = \mathbf{0.412}$$

2.2.2. Rainfall Data quality test

The consistency of estimated 36 years of average areal rainfall depth has to be checked before using the data for any hydrological analysis.

2.2.2.1. Checking the existence of outliers

Outliers are the data points that depart from the trend of the remaining data. The detention or retention of these outliers can significantly affect the magnitude.

Case 1 if Cs<-0.4 Check lower outlier test

Case 2 if Cs >+ 0.4 Check higher outlier

Case 3 if -0.4<Cs<+0.4 Check both higher and lower outlier test

In our case CS = 0.412 Check the higher outlier

The higher and lower outlier was checked by using the given equations below. As shown from the above calculation the station Skew is greater than 0.4, test for high outlier is considered first.

Test for higher outlier

Higher outlier; $Yh = \bar{Y} + KN * S$

$$where; K_N = -3.62201 + 6.28446N^{\frac{1}{4}} - 2.49835N^{\frac{1}{2}} + 0.491436N^{\frac{3}{4}} - 0.037911N = \mathbf{2.639}$$

$$\text{N} = 36, \bar{Y} = \mathbf{1.68}\text{mm}, \text{Sy} = \mathbf{0.096}, C_s = \mathbf{0.412}$$

Higher outlier; $Yh = 1.68 + 2.639 * 0.096 = 1.933$

$$Higheroutlier = (10)^{1.933} = 85.73mm$$

The highest recorded value is (82.22mm) is less than high outlier (85.73mm). Therefore, there is no higher outlier.

Test for lower outlier

Lower outlier $Y_l = \bar{Y} - K_n S_y$

$$Y_h = \bar{Y} - K_n S_y = 1.68 - 2.639 * 0.096 = 1.43$$

$$Loweroutlier = (10)^{1.43} = 26.73mm$$

The Lowest recorded value is (29.1mm) which is greater than lower outlier (26.73mm). Hence, no lower outlier. Therefore, the recorded data is consistent for both outliers and it is possible to use it for analysis.

2.2.2.2. *Checking Data Reliability* and adequacy

Number of data=36

Standard deviation, δ_{n-1} = 11.4543

Mean, $\bar{\boldsymbol{X}}$=49.025mm

Standard error of mean, $\delta_n = \frac{\delta_{n-1}}{\sqrt{n}}$ = 11.4543/6 = 1.909

Relative standard, δ_n / x *100 = (1.909/49.025)*100 = **3.894 % < 10%**

Hence the data series could be regarded as reliable and adequate.

2.2.2.3. Data Consistency Test

The yearly heaviest rainfall data of Debre Work Town metrological recorded data from 1975 to 2011 were taken for the design. These data are not fully recorded, abandoned and only these data fully recorded are taken for computation. These data should be checked for its consistency by outlier test.

2. 2.3. Return period (T)

The average interval in year between events when the occurrence of peak or excess for a given magnitude. The average frequency occurs in an event for a long period of time of years. Selecting higher return period means the corresponding flood magnitude is also very high. In our case going to be construct a diversion weir structure on Gilgel Cheye River, 50 years return periods has been adopted.

Table 3: Guideline for selection of the return period

Type of structure	Return period (year)
Spillways for project with storage more than 60Mm^3	1000
Barrage and minor dams with storage less than 60Mm^3	100
Spillway of small reservoir dam in considering not endangering urban residences	10-20
Diversion weir	50-100

Source: Subermanya (1989) and Nevec (1972).

2.3. Design Flood Computation

Maximum probable flood is a hypothetical flood at a selected location, whose magnitude is such that there is no chance of its being exceeded. It is estimated by combining the most hydrological and meteorological conditions considered reasonably possible at the particular location under consideration.

As we have described earlier 36 years 24 hr (daily) heaviest Rainfall data obtained from Debre Work Meteorological station, which are useful for the determination of the maximum probable flood. Based on the available data, the following methods are used to estimate the design flood.

2.3.1. Design Rainfall distributions

After checking the consistency of the data for higher and lower outlier, the 36 years data is obtained as representative for the analysis. The probability of occurrence of maximum probable rainfall is estimated by the following methods as shown below.

Selection of Distribution

The observed data was tested using different statistical distributions. The most commonly distributions used to fit extreme rainfall events are: Gumbell, Log Normal, Pearson Type III and Log Pearson Type III. The results of the analysis are shown in the following table.

Table 4: Rainfall Distribution methods

1. ***Log Normal Method***	
Design Period, T	50
Probability, P	0.02
K=(Cs/6)	0.000
$W=(Ln(1/P^2))^{0.5}$	2.797
Frequency Factor, KT=(w-((2.51557+0.802853*w+0.010328 w^2)/(1+1.432788*w+0.1899269*w^2+0.001308*w^3)))	2.054
Standared Normal Variance, Z=KT+(KT2-1)*K+1/3*(KT3-6*KT)*K2-(KT2-1)*K3+KT*K4+1/3*K5	2.054
$Y=Y_{mean} + Z*\sigma y$	1.877
Design Rainfall, $X_{50} = 10^Y$	**75.33**

2. Log Pearson Type 3 Distribution	
Design Period, T	50
Probability, P	0.02
K=(Cs/6)	0.069
$W=(Ln(1/P^2))^{0.5}$	2.80
Frequency Factor, KT=(w((2.515517+0.802853*w+0.010328*w^2)/(1+1.432788*w+0.189269*w^2+0.001308*w^3)))	2.054
Standared Normal Variance, Z=KT+(KT2-1)*K+1/3*(KT3-6*KT)*K2-(KT2-1)*K3+KT*K4+1/3*K5	2.2685
$Y=Y_{mean} + Z*Y_{Standared\ deviation}$	1.898
Design Rainfall, $X_{50} = 10^Y$	**78.98**

3. Pearson Type 3 Distribution	
Design Period, T	50
Probability, P	0.02
K=(Cs/6)	0.069
$W=(Ln(1/P^2))^{0.5}$	2.797
Frequency Factor, Z=(w((2.515517+0.802853*w+0.010328*w^2)/(1+1.432788*w+0.189269*w^2+0.001308*w^3)))	2.054
Standard Normal Variance, Kt=Z+(Z2-1)*K+1/3*(Z3-6*Z)*K2-(Z2-1)*K3+Z*K4+1/3*K5	**2.268**
$X_{50}=X_{mean} + K_T*\sigma_x$	**75.01**

4. Gumbel's EVI Type I distribution	
Design Period	50
K_T=-[(ln(ln(T/(T-1))))-Y_n]/Sn	2.976
Design Rainfall, $X_{50} = X_{mean} + K_T*\sigma_x$	**83.11**

Summary of the Design RF as below

Methods	XT	remark
Log Normal	75.34	-
Log Pearson type -3	79.03	-
Pearson type -3	75.01	_
Gumble EVI Methods	**83.11**	**Largest**
The Design rainfall value is 83.11mm		

D - Index test

The D-Index test is believed to be the better fitness test in many literatures. Hence in this study it was used to determine the best statistical distribution to estimate the peak rainfall. The D-index for the comparison of various distributions in upper tail is given as;

D-index = (1/Xm)* $\sum_{i=1}^{6} Abs(Xi - Xi')$

Here xi and xi' are the i^{th} highest observed and computed values for the distribution. The distribution giving the least D-index is considered to be the best-fit distribution.

Table 5: D-Index values

Rank	X	Log normal	Log Pearson type-3	Pearson type-3	Gumble EVI Type -I	Remark
		x-xi	x-xi	x-xi	x-xi	
1	82.2	8.92	5.97	9.060	2.18	
2	78.5	10.22	8.70	9.926	5.65	
3	66	0.80	0.01	0.315	2.59	
4	65.1	2.19	1.80	1.593	0.43	
5	61.6	0.53	0.44	0.127	1.51	
6	59.1	0.41	0.29	1.103	2.01	
Sum		23.08	17.22	22.124	14.36	
D-Index		0.47	0.35	0.45	0.29	

The smallest D index value was found to be for the Gumble EVI Type I Distribution which is 0.29. Accordingly the design rain fall was found to be 83.11 mm for the Gumble EVI Type I Distribution

2.3.2. Catchment characteristics

Total catchment area – 31.411km^2

Longest stream length – 12.33km

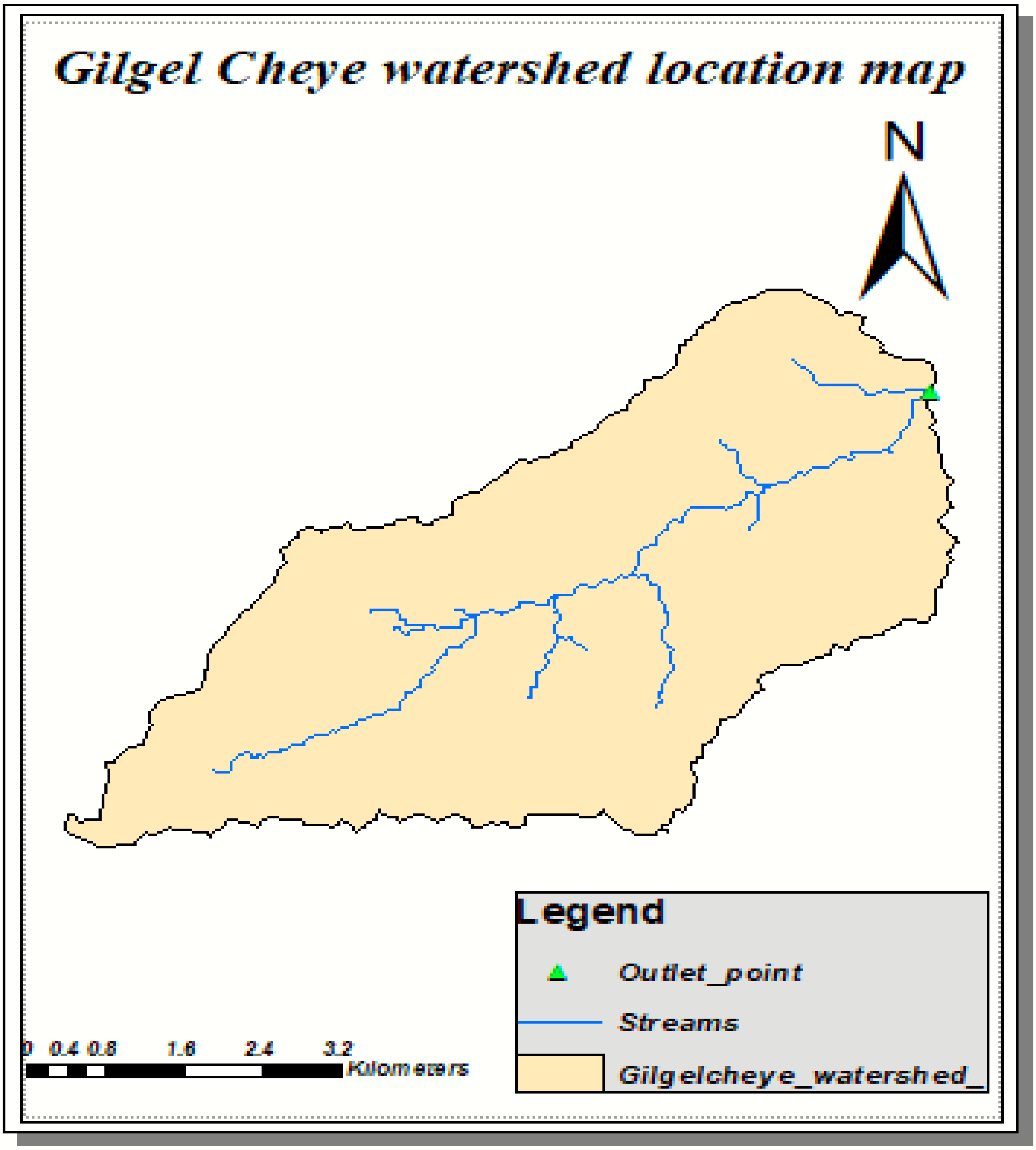


Figure 2: Gilgel Cheye river watershed

2.3.2.1. Slope Classification

The Watershed has marked topographic variation. Six types of slope classes are present as Flat or almost flat (0-3%) covers 2.4%, Gently sloping (3-8%) covers 9.2%, Sloping (8-15%) covers 18.73%, Moderately steep (15-30%) covers 39.03%, Steep slopes; 30-50% & >50% covers 20.86% and 9.8% respectively of the overall watershed area.

The dominant slope class in Gilgel Cheye watershed is moderately steep slope (39.03%) and steep terrain (20.86%); which covers from 15-50% slope classes.

OBJECTID *	Shape *	grid_code	Shape_Length	Shape_Area	Slope_class	Area_ha
1	Polygon	1	101820.531777	749193.028583	0-3	74.919303
2	Polygon	2	358313.073442	2880030.879003	3-8	288.003088
3	Polygon	3	693547.261733	5882645.658725	8-15	588.264566
4	Polygon	4	995726.3201	12261152.021713	15-30	1226.11520
5	Polygon	5	660207.716167	6552468.451882	30-50	655.246845
6	Polygon	6	242337.590387	3085484.829448	>50	308.548483

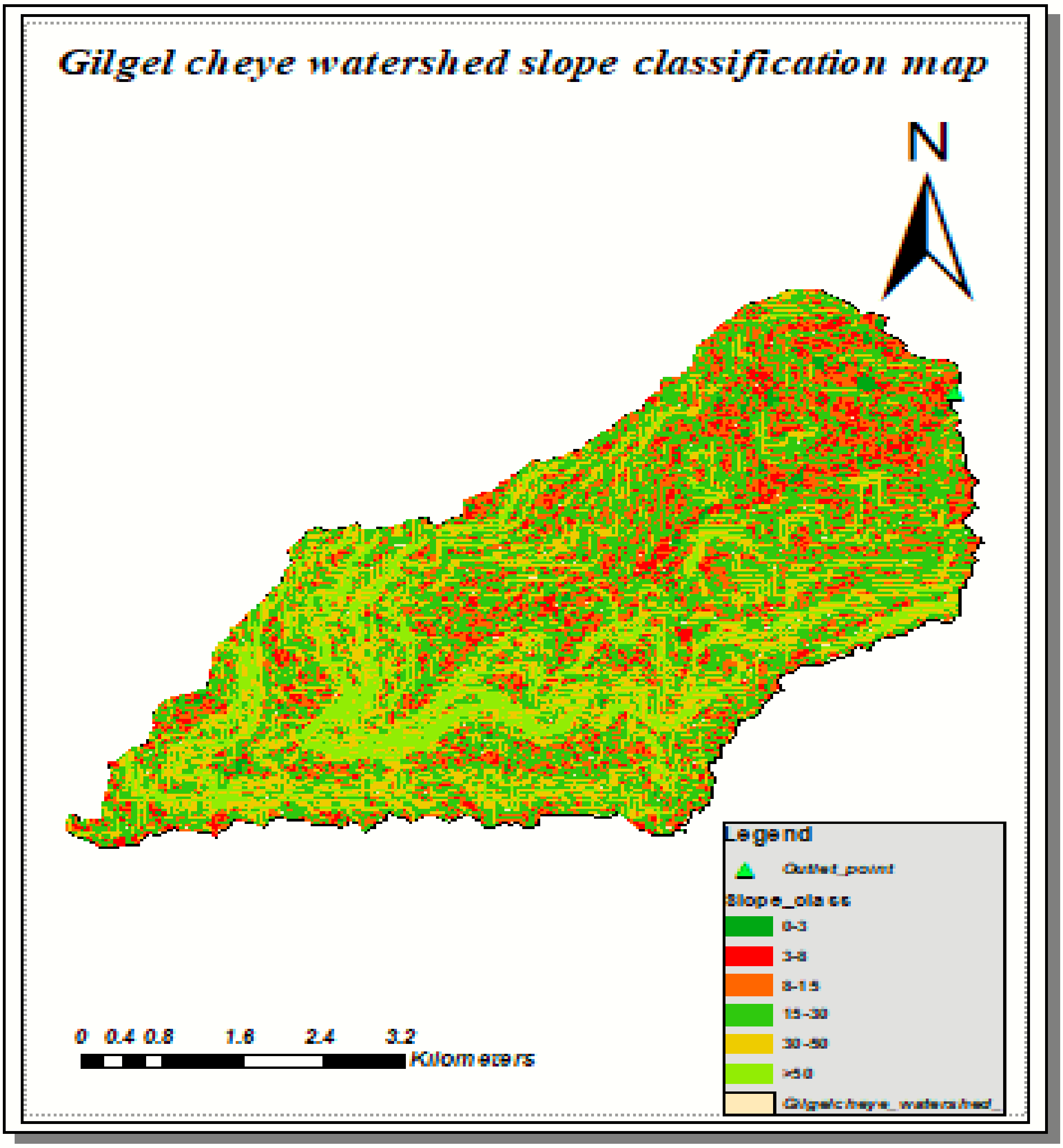


Fig 3: Gilgel Cheye watershed Slope class

2.3.2.2. Soil Texture

Soil conditions determine what agricultural production possibilities exist in a given area from a biophysical perspective. Soil erosion also depends greatly on the infiltration rate of soil. The infiltration rate again depends on the soil texture, structure, humus and moisture content, soil depth and surface roughness. Soil conservation structures used for rehabilitation of degraded lands recommended based on the aforementioned soil properties.

Accordingly from the two soil textural classes; clay loam is the dominant one which covers 83.4% from the total catchment in Gilgel Cheye watershed

OBJECTID *	Shape *	Texture	Shape_Length	Shape_Area	area_sqkm
1	Polygon	CLAY	15065.397646	5229104.608834	5.229105
2	Polygon	CLAY LOAM	37804.750497	26181870.217624	26.18187

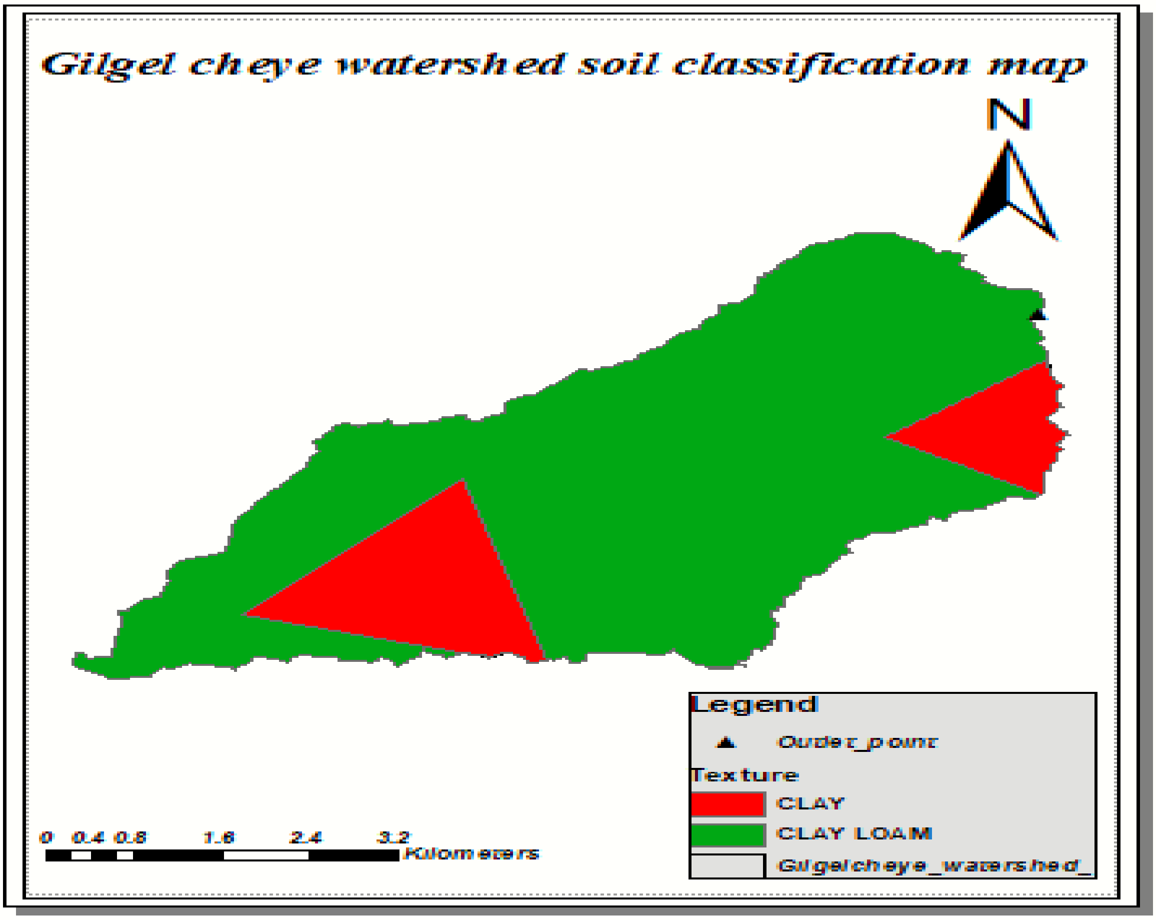


Fig 4: Gilgel Cheye watershed Soil classification

2.3.2.3. Land use and land cover

The major land use/land cover types of Gilgel Cheye watershed have been identified described and mapped. The major land use types includes: Cultivated and plantation forest land. The description of these land use/cover types is presented as follows.

Cultivated Land: This is a major and dominant land use covering about 21.9sq km (69.7%) of the total Watershed area. Annually, this unit is entirely put under cultivation of field crops. Crops grown include Teff, Barley, potato and Wheat. The cultivated fields are considered to be the main sources for erosion at higher elevation is very severe.

Grassland: This land use/cover about 8.5sq km (27.06%) of the total area of the watershed.

Natural forest: This land use/cover includes areas where naturally grown forests. The unit occupied for about 0.2sq km (0.6 %) of the total area of the watershed.

Plantation: This land use/cover includes areas where exotic species is grown. The unit occupied for about 0.32sq km (1.02%) of the total area of the watershed.

Shrub land: This land use/cover about 0.52sq km (1.66%) of the total area of the watershed.

OBJECTID *	Shape *	LC1LU_DES	Shape_Length	Shape_Area	Area_sqkm
1	Polygon	Cultivation	47577.678321	21898637.071447	21.898637
2	Polygon	Grassland	33935.923496	8472338.484215	8.472338
3	Polygon	Natural Forest	3999.9988	199999.88	0.2
4	Polygon	Plantation	5199.9983	319999.77	0.32
5	Polygon	Shrubland	5599.9984	519999.660001	0.52

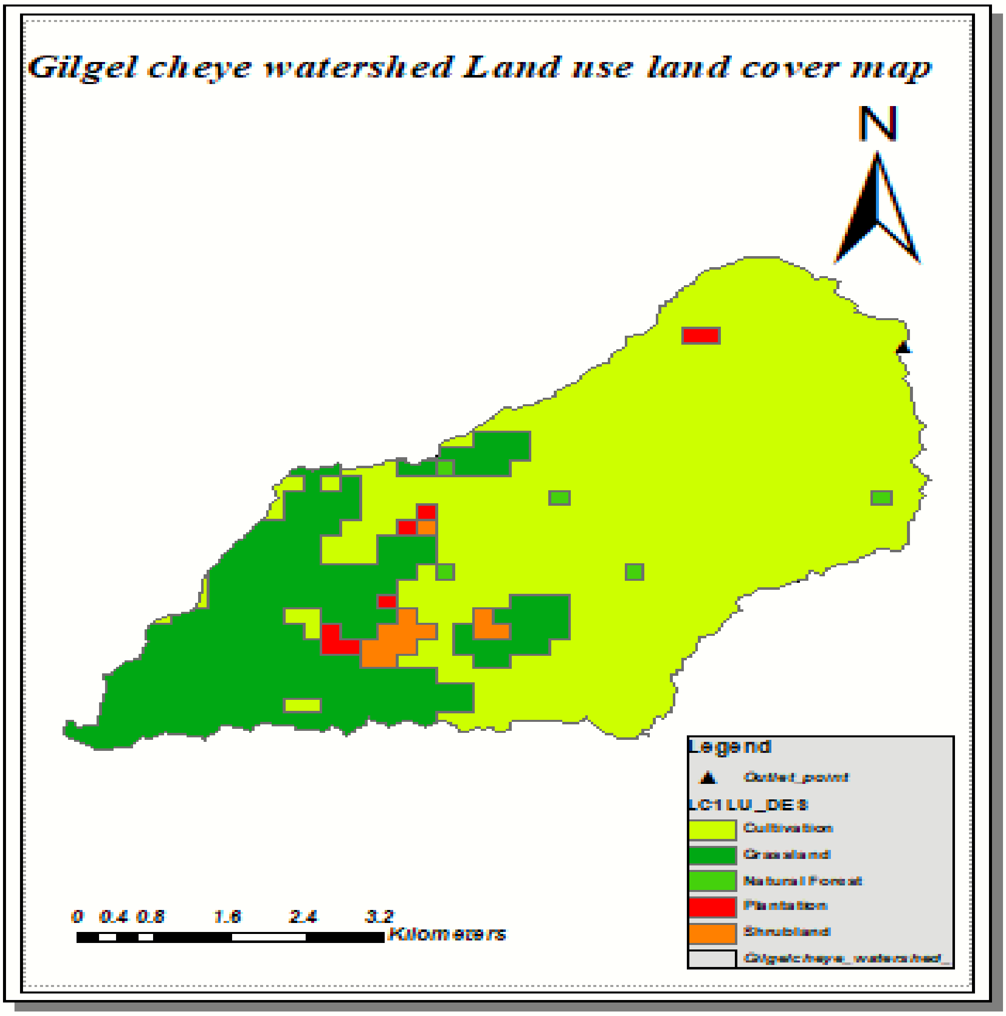


Fig 5: Gilgel Cheye watershed land use land cover

2.3.2.4. Curve number (CN)

Curve number (CN) is achieved based on SCS method the watershed characterize in terms of land cover, treatment, hydrologic condition and soil group. Runoff Curve number for the given areas is taken to other agricultural land and cultivated agricultural land of soil**.** That is ***clay** and **clay loam*** and the Soil have a high runoff potential due to high infiltration rates. From the watershed analysis the curve number at condition II is 84.08

Table 6: Run–off curve number calculation from land use

No.	**Land use**	**Area ratio**		**Hydrologic Soil group**	**Textural Class**	**Hydro logic condition**	**Curve No. 'CN'**	**Weighted "CN"**	**Sum weighted "CN"**	
		Area (ha)	**Ratio**						II	III
1	Cultivation	21.90	0.697	D	Clay, Clay loam	Poor	88	61.36	**84.08**	**92.88**
2	Grassland	8.47	0.270	C	Clay, Clay loam	fair	79	21.30		
3	Natural forest	0.20	0.006	A	Clay, Clay loam	good	30	0.19		
4	Plantation	0.32	0.010	A	Clay, Clay loam	good	30	0.31		
5	Shrub land	0.52	0.017	B	clay	fair	56	0.93		
Total		**31.41**	**1.000**					**84.08**		
NB: ***CN-III = 43.8*LnCN-II - 101.23***										

However, peak run off is estimated for antecedent moisture condition III and the curve number value at condition II should be converted to antecedent moisture condition III.

CN Condition (II) = 84.08

CN Condition (III) = 48.3*Ln CN Condition (II)-101.3 = 92.88. These soils primarily consist of clay with low swelling potential, soil with a clay pan or clay layer at or near the surface.

For the design and analysis of structures to be constructed on the river, estimation of flood magnitude is an important task. This can be done using different techniques depending on the data available. For this particular case, there are no river flow data and hence the flood estimation is done using the rainfall data and applying SCS Curve number method.

2.3.3. Peak Flood Analysis Using SCS Method

For un-gauged rivers, the design flood can be simulated by using SCS unit hydrograph method. The computation is done using design rainfall or storm estimated earlier, In the hydrologic analysis of flood using SCS method; rainfall amount and storm distribution, catchment area, shape and orientation, ground cover, type of soil, slopes of terrain and stream(S), antecedent moisture condition can be used and all such data shall be carefully determined before proceeding to SCS simulation.

2.3.3.1. Time of concentration (Tc)

Time of concentration has been calculated by taking the stream profile of the longest streamline and dividing it in to different elevation ranges. Kirpich formula is adopted for computation.

Table 7: Determination of Time of Concentration

	River Length(m)	River length(km)	Min. Elevation	Max. Elevation	Slope (%)	Tc. (hr)
	3273.16	3.273	2608	2660.00	0.016	0.836
	4390.73	4.391	2650	2840.00	0.043	0.712
	4665.71	4.666	2720	3620.00	0.193	0.420
Sum	***12329.6***	**12.33**			**0.252**	**1.97**

The formula is, ***Tc=1/3000*(L/S^ (1/2)) ^0.77***

In order to increase the accuracy of Tc the partial distance and the elevation is classified based on the slope 0-4, 4-19 and > 19%

Tc = 1.97hr Since Tc < 3hr., duration of excess rainfall difference, D = Tc/6 = 0.33hr

Time to peak, $T_p = \frac{D}{2} + 0.6 * T_c$ = 1.34hr

Base time, $T_b = 2.67 * T_p$ = 3.59hr

Lag time T_l = *0.6*Tp = 1.18hr*

2.3.3.2. Design of point rain fall

Rainfall Profile

Rainfall profile is the distribution of the proportion of design rainfall during every incremental time on the watershed area during the 24 hours duration. Well developed models are needed to determine such an event for the selected basin area. But there are no sufficient modeling studies in the vicinity and adaptation of standard curves has been taken as the only option.

Table 8: Area to point rainfall profile

Areal to point rain fall ratio

Area km²	Duration (hrs)	0.50	1.00	2.00	3.00	4.00	5.00	6.00	9.00	12.00	15.00	18.00	21.00	24.00
25		66	78	82	85	87	88	88	91	92	93	93	94	94
50		61	71	78	82	84	85	87	89	90	91	92	92	93
75		57	67	75	79	82	84	83	87	89	90	91	91	92
100		54	65	73	78	80	82	83	86	88	89	90	91	91
125		52	63	72	76	79	81	82	85	87	88	89	90	91
150		50	61	70	75	78	80	61	84	86	88	89	89	90
175		48	59	69	74	77	79	81	84	86	87	88	89	90
200		46	58	68	73	76	78	80	83	85	87	88	88	89
225		45	57	57	72	75	77	72	82	85	86	87	88	89
250		44	55	66	71	74	77	78	82	84	86	87	88	88
275		42	54	65	70	74	76	78	81	84	85	86	87	88
300		41	53	54	70	73	75	77	81	83	85	86	87	88
325		40	53	63	58	72	73	77	80	83	84	86	87	87
350		38	52	63	68	72	74	76	80	82	84	85	86	87
375		39	51	62	68	71	74	78	80	82	84	85	86	87
400		38	50	61	67	71	73	75	79	82	83	85	86	87
425		37	50	61	67	70	73	75	79	81	83	84	85	86
450		36	49	60	66	70	72	74	79	81	83	84	85	86

Duration, hr.	RF Ratio(by interpolation), %
0.33	27.75
0.66	37.98
0.98	43.48
1.31	49.33
1.64	54.53
1.97	57.83

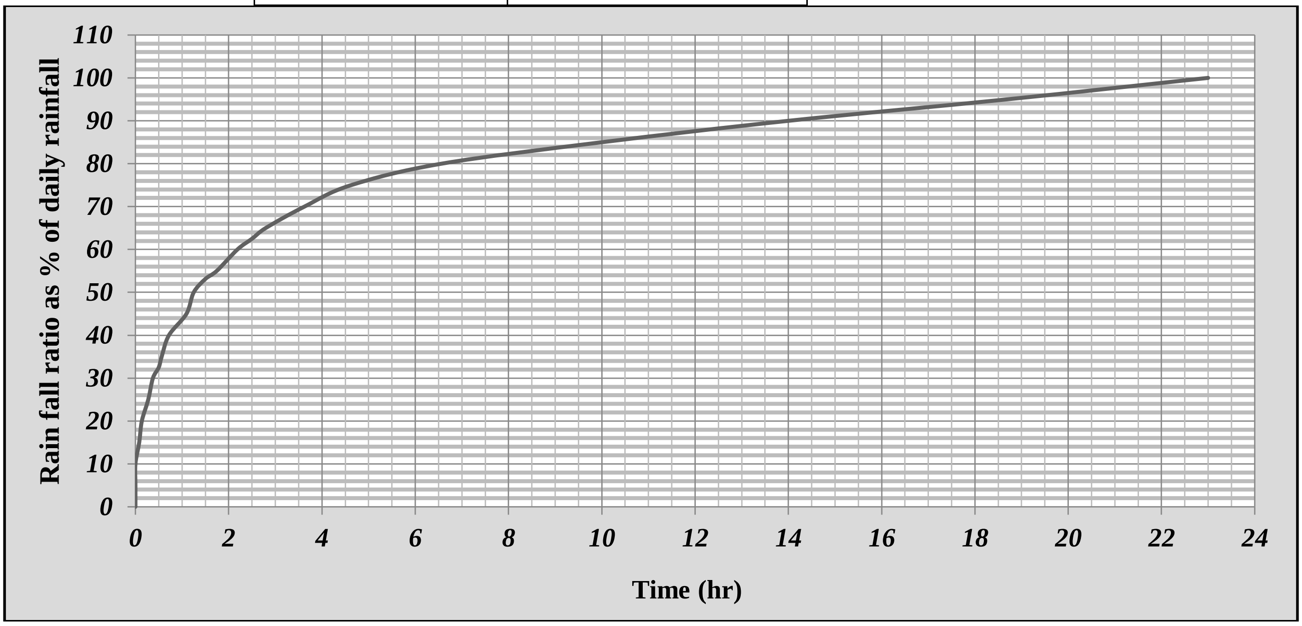


Figure 6: Rainfall ratio distribution curve

Table 9: Design rain fall arrangements

Duration	Daily point rainfall	Rainfall Profile		Areal to point rainfall ratio	Areal-rainfall	Incremental rainfall	Descending order	Descending order
hr	mm	%	mm	Interpolated values	mm	mm	mm	No.
0-0.33	83.11	27.75	23.06	0.65	14.93	14.93	14.93	1
0.33-0.66		37.98	31.57	0.70	22.14	7.21	7.21	2
0.66-0.98		43.48	36.14	0.76	27.41	5.27	5.27	3
0.98-1.31		49.33	41.00	0.79	32.34	4.93	4.93	4
1.31-1.64		54.53	45.32	0.80	36.22	3.88	3.88	5
1.64-1.97		57.83	48.06	0.81	38.88	2.66	2.66	6

Rearranged order	Rearranged incremental order	Cumulative order	Times of incremental hydrograph		
			Time of beginning	Time to peak	Time to end

No.	mm	mm	hr	hr	hr
6	2.66	2.66	0	1.34	3.59
4	4.93	7.59	0.33	1.67	3.92
3	5.27	12.86	0.66	2.00	4.25
1	14.93	27.79	0.98	2.33	4.57
2	7.21	35.00	1.31	2.66	4.90
5	3.88	38.88	1.64	2.98	5.23

2.3.4. Direct run off estimation

Runoff Analysis

Direct run-off, $Q_P = \frac{(I-0.2S)^2}{(I+0.8S)}$

Where, I=Rearranged cumulative run-off depth (mm)

S=Maximum run of potential difference, = (25400/CN)-254

Peak run-off rate for incremental runoff depth; Qp = 0.21A/Tp

Where, A= Catchment area= Km^2 Tp = Time to peak hr. Q = Incremental run-off (mm)

Table 10: Direct run off values per unit depth, mm

The maximum potential difference b/n Rainfall (p) and direct runoff (Q), S = (25400/CN) - 254, CN=Value corresponding to AMC III	mm	***S = 19.46***	***(p-0.2*19.46)^2/(p+0.8*19.46)***
$Q = \frac{(P-0.2S)^2}{(P+0.8S}$		P (mm)	Q (mm)
		2.66	0.00
		7.59	0.59
		12.86	2.83
		27.79	13.17
		35.00	19.14
		38.88	22.48

Table 11: Computation of peak discharge for each incremental runoff

No.	Duration	Cumulative run off	Incremental run off	Peak run off for increment	Time of beginning	Time to peak	Time to end
	hrs	mm	mm	m^3/sec	hrs	hrs	hrs
1	0.33	0.00	0.00	0.00	0.00	1.64	3.59
2	0.66	0.59	0.59	2.90	0.33	1.97	3.92
3	0.98	2.83	2.24	10.98	0.66	2.30	4.25

4	1.31	13.17	10.34	50.72	0.98	2.62	4.57
5	1.64	19.14	5.97	29.27	1.31	2.95	4.90
6	1.97	22.48	3.35	16.42	1.34	2.98	4.93

Table 12: Peak Discharge Analysis Using Complex Hydrograph Method

Time	***Complex Hydrograph (m^3/Sec)***						
(hr)	***H1***	***H2***	***H3***	***H4***	***H5***	***H6***	***HT***
0	0.00						**0.00**
0.33	0.00	0.00					**0.00**
0.66	0.00	0.58	0.00				**0.58**
0.98	0.00	1.16	2.20	0.00			**3.35**
1.31	0.00	1.74	4.39	10.14	0.00		**16.27**
1.34	0.00	1.79	4.58	11.01	0.50	0.000	**17.88**
1.64	**0.00**	2.32	6.59	20.29	5.86	3.00	**38.06**
1.97	0.00	**2.90**	8.78	30.43	11.71	6.29	**60.11**
2.30	0.00	2.41	**10.98**	40.58	17.56	9.57	**81.10**
2.62	0.00	1.92	9.13	**50.72**	23.42	12.86	**98.05**
2.95	0.00	1.43	7.29	42.19	**29.27**	16.14	**96.32**
2.98	0.00	1.39	7.13	41.46	28.85	**16.42**	**95.25**
3.59	0.00	0.49	3.69	25.59	19.69	11.28	**60.74**
3.92		0.00	1.85	17.06	14.77	8.52	**42.19**
4.25			0.00	8.53	9.85	5.76	**24.13**
4.57				0.00	4.92	3.00	**7.92**
4.90					0.00	0.24	**0.24**
4.93						0.00	**0.00**
Q_{peak} = 98.05							

From the analysis, the fifty years return period design flood is 98.05m ^{3}/s. In the above table, the ordinates of every triangular hydrographs of incremental runoff are calculated by applying the concept of similarity of a triangle. This is done for known values of "time to begin, time to peak, time to end and peak run off" to each hydrograph, H. Here independent formulas are used for the rising and recession limbs to compute runoff magnitudes among the time to begin, time to Peak and time to end. Mathematically,

$$Rising\ Limb = Q_p - \left[\frac{t_p - Arranged\ time}{t_p - t_{begining}}\right] * Q_p$$

This formula is used to compute an ordinate at some time "Ti" for the rising limb of an incremental triangular hydrograph.

$$\textit{Falling Limb} = Q_p - \left[\frac{\textit{Arranged time} - t_p}{t_{end} - t_p}\right] * Q_p$$

Where: Q_p = Peak runoff value at time to peak for 0D,1D,2D,....or 5D triangular hydrograph, m^3/sec

$T_{begining}$ = Beginning time of every incremental runoff, hrs

Ti = Instantaneous time between time to begin and peak, hrs

T_p = Time to peak, hrs

T_{end} = Time to end, hrs

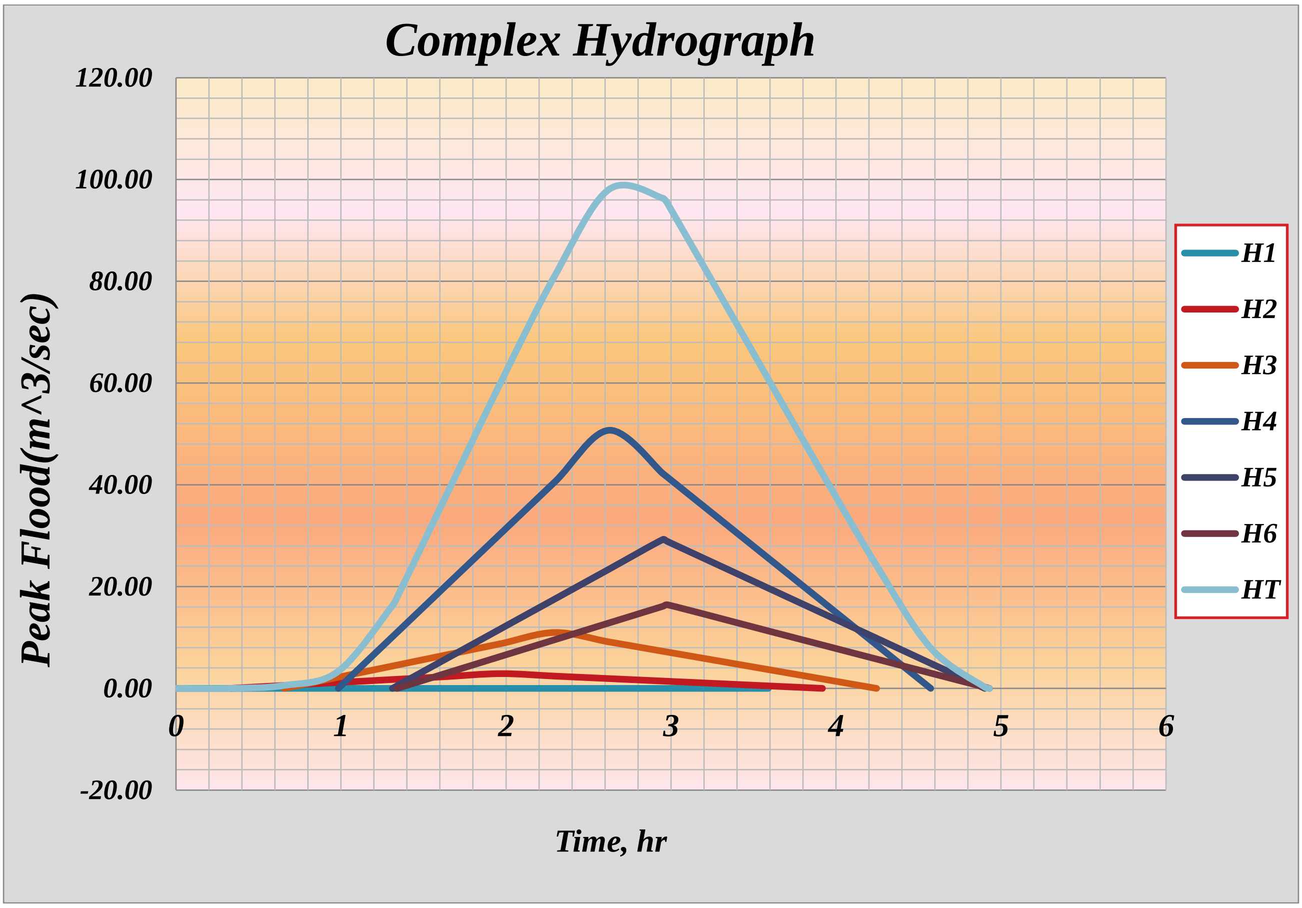


Figure 7: Complex hydrograph curve

2.4. Rating curve (Tail Water Depth Computation)

Tail water depth of the river is equal to the flood depth and amount at the proposed weir site before construction of the weir. It is used to see the flood feature after the hydraulic jump. The water discharge is calculated by Manning's open channel formula. Basic inputs for the analysis and the detail procedure are described as follows:

2.4.1 Average river bed slope

Average riverbed slope of Gilgel Cheye River is estimated by using best-fit line method. The cross section of the river is surveyed at different points along the river channel around the headwork site. Surveying work done for 138.31m lengths. And then, average water surface slope is considered as the riverbed slope.

Table 13: Average river bed slope computation

Points	Northing	Easting	Elevation	Partial distance	Cumulative distance	height	Cumulative height	Total area, m^2	Remark
1	399604.01	1187744.01	2577.28	0	0	0	0	0.00	RC at U/s
2	399617.61	1187747.05	2576.65	13.93	13.93	0.626	0.626	4.36	RC at U/s
3	399627.57	1187755.70	2576.53	13.20	27.13	0.115	0.741	9.02	RC at U/s
4	399647.26	1187764.24	2576.15	21.46	48.59	0.384	1.125	20.02	RC at U/s
5	399657.40	1187767.11	2576.03	10.54	59.13	0.123	1.248	12.51	RC at U/s
6	399678.03	1187772.67	2574.88	21.36	80.49	1.144	2.392	38.88	RC at U/s
7	399668.85	1187769.70	2574.77	9.65	90.14	0.113	2.505	23.62	RC at U/s
8	399685.31	1187773.73	2574.57	16.95	107.09	0.198	2.703	44.14	RC at D/s
9	399710.46	1187774.95	2574.26	25.18	132.27	0.317	3.02	72.05	RC at D/s
10	399699.61	1187776.40	2574.16	10.95	143.22	0.098	3.118	33.61	RC at D/s
11	399706.11	1187775.82	2574.06	6.53	149.75	0.095	3.213	20.66	RC at D/s
Total				***149.75***				***240.57***	
*Average Height $H_{avg.}$ = 2*A/L = 3.21*									
Average Slope, S = $H_{avg.}$/L = 0.021									

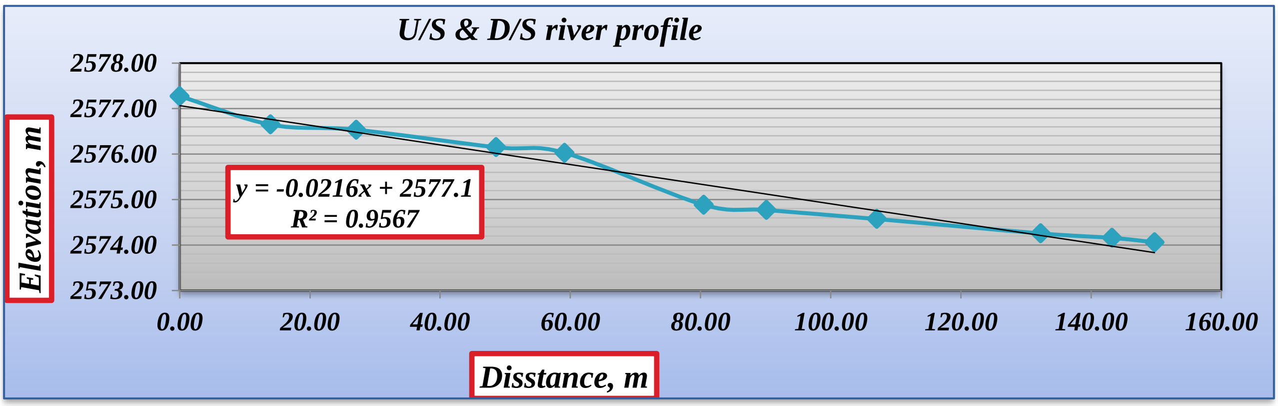


Figure 8: Average river slope profile

From the above U/S and D/S river profile curve the slope of the river is 0.021

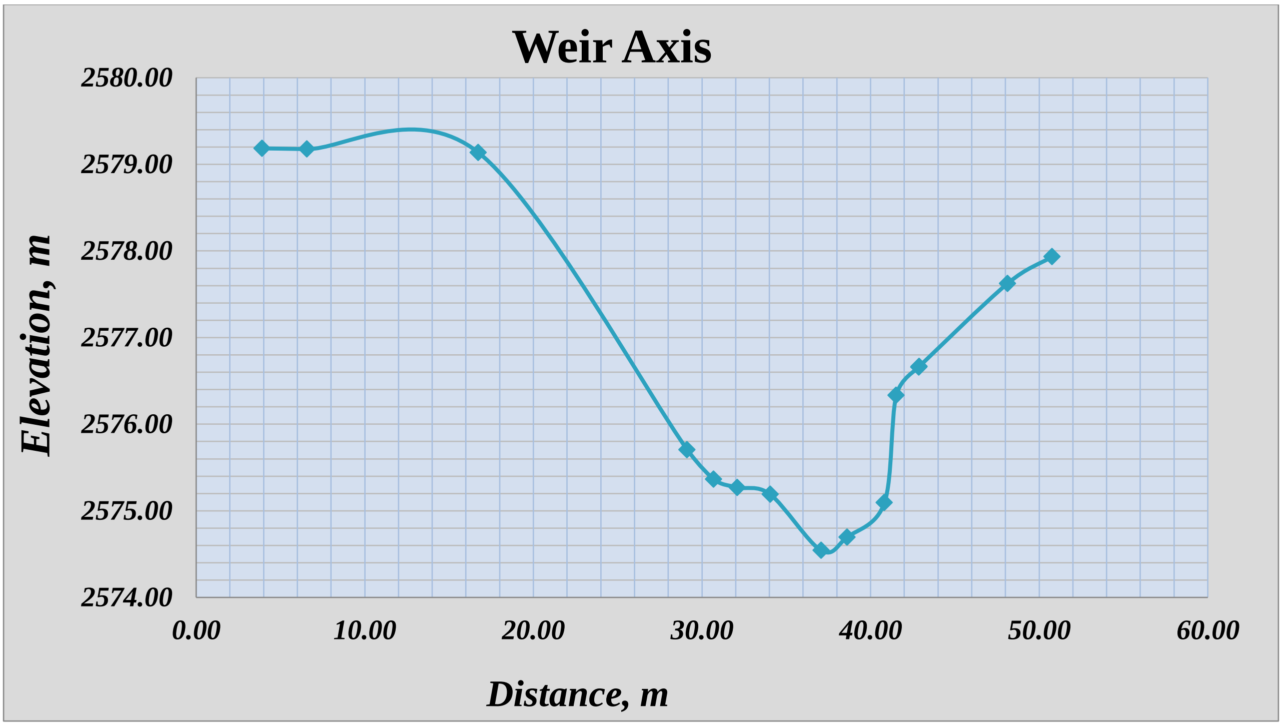


Figure 9: Weir axis

2.4.2. Manning's Roughness Coefficient

The Manning's roughness coefficient is taken from standard table based on the river nature. The river at the headwork site has got braded feature and curving nature. The river bed and banks are defined as channels in gravels with stones and relatively not smooth. Manning's roughness coefficient (n = 0.0325) is adopted.

Table 14: Manning's Roughness Values

S. No.	*Type of Surface*	*Manning's N*
1	Earth channels, clean, straight and uniform	0.016-0.02
2	Earth channels, clean but weathered	0.018-0.025
3	Earth channels, with grass and weeds	0.022-0.033
4	***Channels in gravels with stones***	***0.03-0.035***
5	Channels in rock, smooth and uniform	0.025-0.04
6	Channels in rock, rough	0.035-0.05
Take n = 0.0325		

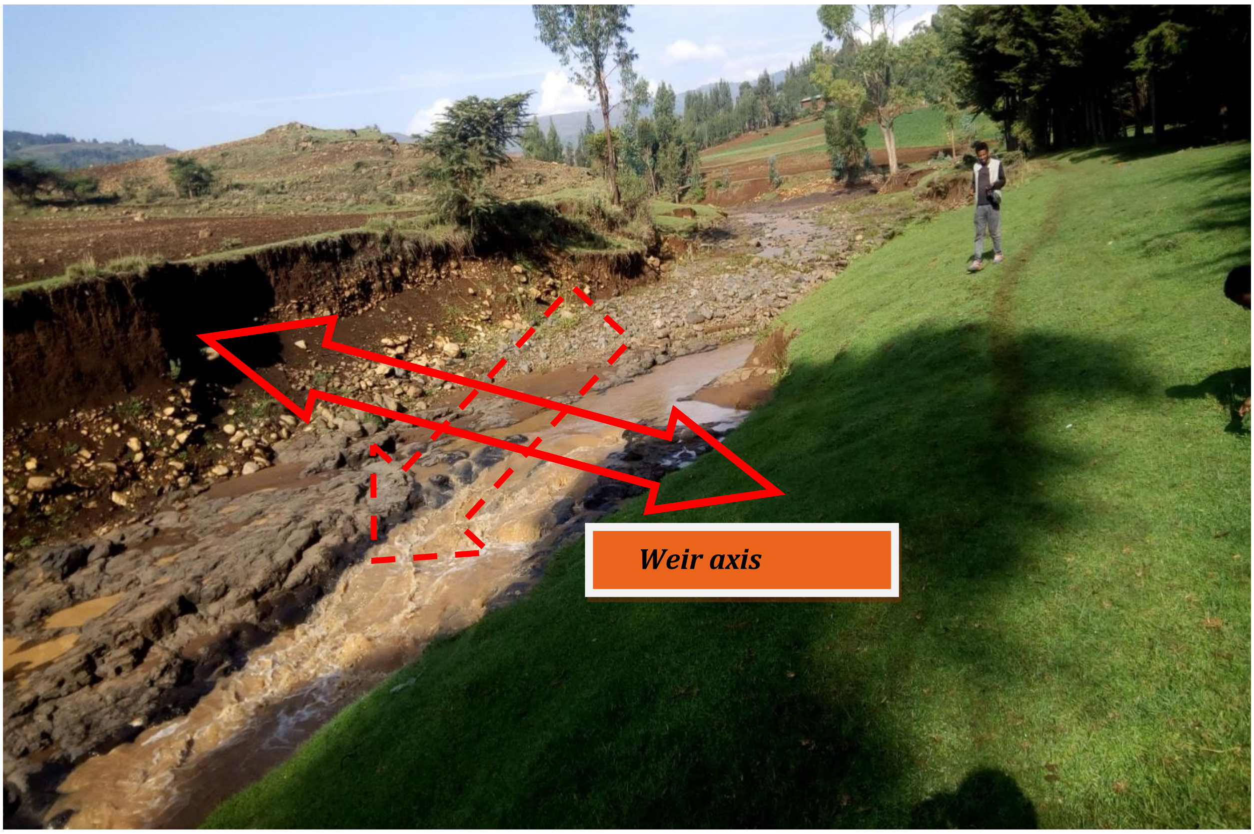


Figure 10: Features of the river

2.4.3. Discharge of the river

Input data:

Manning's roughness coefficient, n = 0.0325

Average river bed slope, S = 0.021

$$V = \frac{1}{n} \times R^{2/3} \times \sqrt{S}$$, Where, R = Hydraulic radius(Area/Perimeter)

$$Q = V \times A$$

Table 15: River Discharge computation at different stage of flow

*Tail water depth computation from **AUTOCAD** results*								
Elevation	***Area***	***Perimeter***	***River Slope***	***Manning's n***	***R***	***V***	***Q***	***D***
2574.54	0.000	0.000	0.021	0.043	0.000	0.000	0.00	0.00
2574.84	0.616	3.816	0.021	0.043	0.161	0.030	0.02	0.30
2575.14	2.199	6.669	0.021	0.043	0.330	0.125	0.27	0.60
2575.44	4.831	10.881	0.021	0.043	0.444	0.226	1.09	0.90
2575.74	8.269	12.408	0.021	0.043	0.666	0.510	4.22	1.20
2576.04	12.117	13.669	0.021	0.043	0.886	0.903	10.94	1.50

2576.34	16.341	14.954	0.021	0.043	1.093	1.372	22.42	1.80
2576.64	21.110	17.130	0.021	0.043	1.232	1.745	36.83	2.10
2576.94	26.625	19.811	0.021	0.043	1.344	2.075	55.25	2.40
2577.24	32.956	22.534	0.021	0.043	1.463	2.457	80.98	2.70
2577.39	***36.424***	***23.855***	***0.021***	***0.043***	***1.527***	***2.678***	***98.05***	***2.85***
2577.54	40.105	25.258	0.021	0.043	1.588	2.896	116.16	3.00

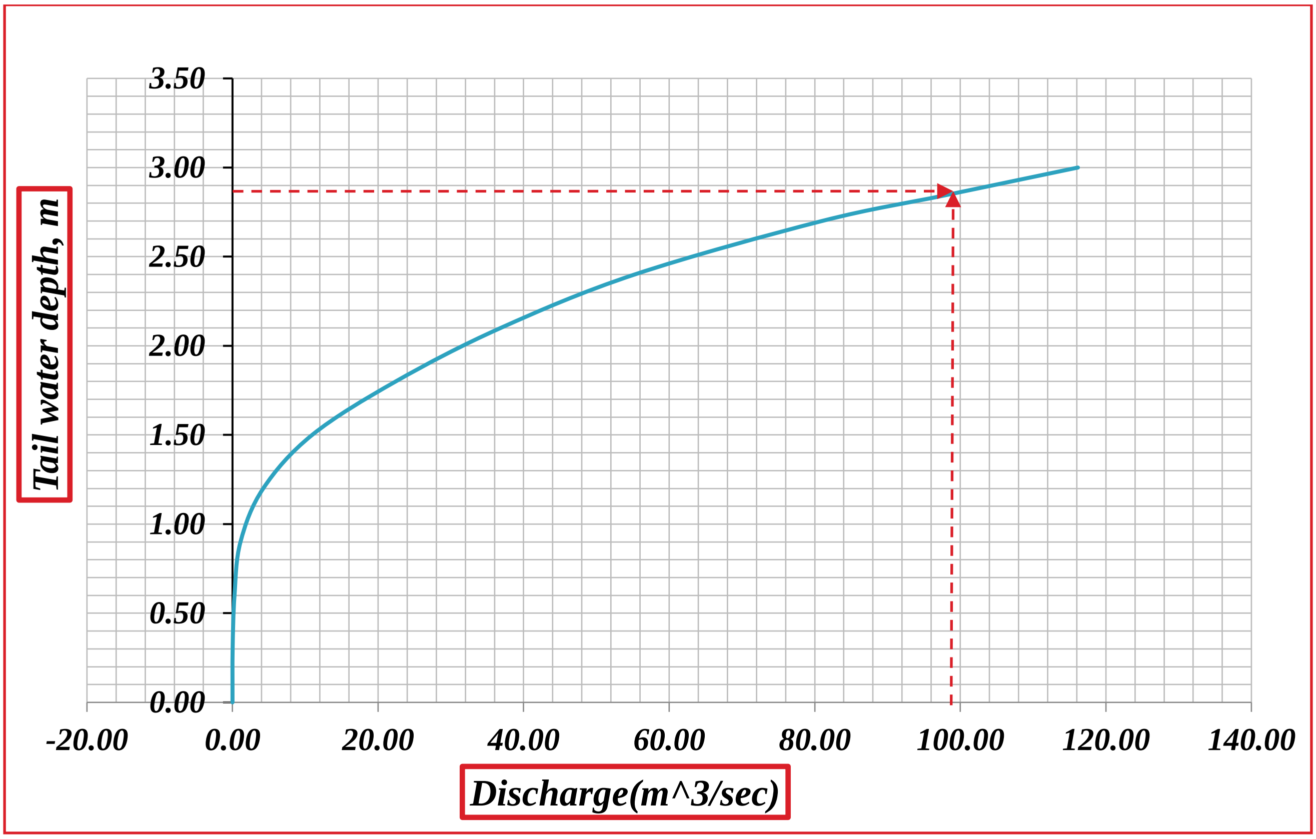


Figure 11: Tail water depth Vs Discharge Curve

From the above table tail water depth of Discharge rating curve, tail water depth equivalent to the flood discharge (Q = 98.05m^3/sec) is found to be 2.85m

Tail water level = 2574.54m a.s.l. + 2.85m = 2577.39m a.s.l

3. HEAD WORK STRUCTURE DESIGN

3.1.1 Upstream and Downstream of the Head Work Site

Gilgel Cheye River has meandering feature on upstream and downstream of the headwork site. The river slope is steep as it goes to the downstream and water flow changes its course forms meandering.

3.1.2 Suspended and Construction Materials

The river transports boulder, sand soil gravel and silt soil from the watershed. But, the sand and boulder materials in the river are not suitable for construction. Sand is accessible from far distance i.e. Abay Valley; masonry stone is obtainable around the proposed site. Thus, for the construction of Gilgel Cheye small scale irrigation project, available materials are attainable along and around the river (from site explanation).

3.2. Hydraulic Design of the Weir

3.2.1. Weir Type Selection and Shape

Encounter the availability of construction easily, materials, considering the river features, the height of the weir that can dissipate the energy of water and to protect its edge from crushing effect of boulder ***broad crest weir type*** is selected for Gilgel Cheye small scale irrigation project. So that, it is better for the structure foundation and the risk of seepage through weir can be minimized. The proposed weir is to be constructed by masonry with reinforced concrete capping on the top, U/S and D/S face of the weir body. The masonry consists of 65% Stone and 35% mortar (1:3 mix ratios).

3.2.2. Weir Crest length

Lacey's regime width, L= 4.75*√Q = 4.75*(98.05 ^0.5) = 47.03m and multiply by looseness factor of 0.5 gives the value 23.52m. But, Actual width of the river section is 16m. Since the right and left banks of the river is weak and the river bed formation is rock formation, the existing section of river width is 16m for overflow section.

3.2.3 Weir Height

The height of weir (crest elevation) is fixed based on the requirements of canal intake and maximum command area elevation (canal full supply level).

- Maximum command area elevation = 2575.19m
- Elevation distance from the weir = 283.19
- Water depth required in the canal = 0.3m
- Free board = 0.2m
- Total canal depth = 0.5m
- Canal slope = 0.1% (1/1000)
- Drop due to main canal slope =0.001*283.19 = 0.2832m
- Head loss across head regulator =0.10m
- Turn out losses = 0.05

- Deriving head = 0.15m
- Bed level (deepest point) of the river = 2574.54m a.s.l

⇒ Weir crest level = 2575.19 + 0.3+ 0.1 + 0.05 0.15+ 0.283 = 2576.07m a.s.l

⇒ Weir Height, H = 2576.07 – 2574.54 = 1.53m

⇒ $\boldsymbol{take\ the\ weir\ height, h = 1.6m}$

3.2.4 Water depth over the crest

The head-discharge equation for the flow over the weir crest: $H_e = \left(\frac{Q}{CLo}\right)^{2/3}$

Where;

Q = Design discharge of the weir (98.05m^3/s)

C = Coefficient of discharge = 1.71 for broad crested type of weir

L_o = Length of the overflow section of the weir = 16m

H_e = Effective head over the crest of the weir including velocity head

$$H_e = \left(\frac{Q}{CLo}\right)^{2/3} = \left(\frac{98.05}{1.71 * 16}\right)^{2/3} = 2.34m$$

Approach velocity head, H_a

The approach velocity head can also be expressed in terms of approach velocity:

$$H_a = \frac{{v_a}^2}{2g}$$

$$Ha = He - Hd = 2.34 - Hd$$

$$v_a = \frac{Q}{A} = \frac{Q}{L_o(\ h + Hd)}$$

$$H_a = \frac{Q^2}{2 *\ g\ {L_o}^2 (h + H_d)^2}$$

$$Ha = \frac{98.05^2}{2 * 9.81 * 16^2 * (1.6 + \mathrm{Hd})^2} = 2.34 - Hd$$

The value of H_d can be calculated by trial and error method as follows;

Hd	left equation	right equation
0.5	*0.434*	*1.84*
1	*0.283*	*1.34*
2	*0.148*	*0.34*
2.21	*0.132*	***0.132***

Therefore; **Hd = 2.21m**

The approach velocity head, Ha = He - Hd = 2.34m – 2.21m = ***0.132m***

3.2.5. High flood and energy level

The upstream and downstream flood energy level becomes:

⇒ $\frac{U}{s}HFL = Zo + h + Hd = 2574.54 + 1.6 + 2.21 = 2578.35m$ a.s.l

Where; Z_o = Elevation of river bed at the weir axis = 2574.54m.a.s.l

⇒ $U/s\,TEL = \frac{U}{S}HFL + Ha = 2578.35 + 0.132 = 2578.48m$.a.s.l

⇒ $\frac{D}{S}HFL = Zo + Y3 = 2574.54 + 2.85 = 2577.39m$ a.s.l.

⇒ $DSTEL = DSHFL + ha$, Where; h_a = velocity head in D/S side when the peak flood passes.

⇒ $ha = \frac{v2}{2g}$ and $V = \frac{Q}{A} = \frac{98.05}{36.424} = 2.692m/sec =$

$the\ values\ of\ Q\ \&\ A\ are\ from\ tail\ water\ depth\ computation$

⇒ $ha = \frac{(2.692)^2}{(2*9.81)} = \mathbf{0.369m}$

⇒ *DSTEL = 2577.39+0.369* ***=2577.76m.a.s.l.***

$$\boldsymbol{Afflux} = USHFL - DSHFL = 2578.35 - 2577.39 = \mathbf{0.96m}$$

Afflux; is the rise in the height of flood level of the river u/s of the weir as a result of construction of the weir. Therefore the actual condition of Gilgel Cheye River afflux is **0.96m**.

3.2.6. Section of the weir body

The dimension of weir body is governed by the stability condition where by all the external force acting on the weir should be counter balanced by the weight of the weir body. Preliminary dimension of the weir body is fixed based on Bligh's recommendation and this should be evaluated by the stability design. Bligh's has given the following expression for the design of the weir wall (Garg, 2007).

The top width of weir:

$b = \frac{Hd}{\sqrt{G-1}} = \frac{2.21}{\sqrt{2.3-1}} = 1.9m\ take\ \mathbf{1.0m}$ After the stability analysis

Where: G is the specific gravity of the floor material, G = 2.3 for masonry, and

The bottom width of the weir:

$B = \frac{h+H_d}{\sqrt{G-1}} = \frac{1.6+2.21}{\sqrt{2.3-1}} = 3.34$ Take **3.0m** after the stability analysis

The preliminary dimension of ***1.0 m*** top width and ***3.0m*** bottom width are adopted in the side slope of 2.56H: 1V on the D/s face to stabilize the structure through summer season at the occurrence of high flood and pond conditions.

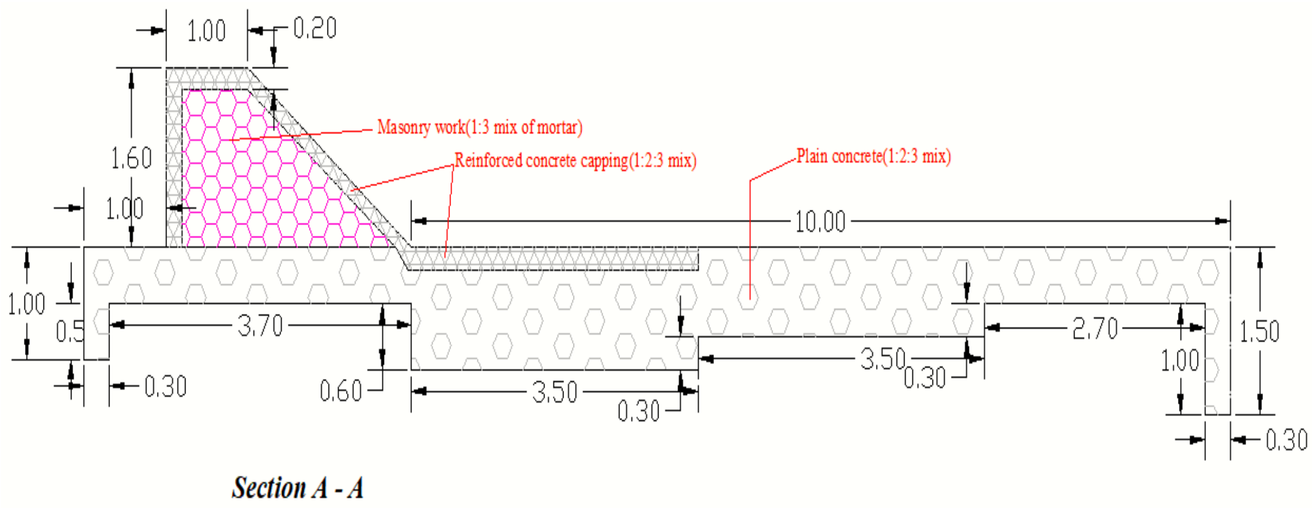


Figure 12: Gilgel Cheye Diversion Weir cross section

3.3 Stability Analysis of the Weir

Gilgel Cheye diversion weir is designed as gravity weir where the bulk of external forces like water pressure and silt pressure are resisted by the weight of the weir. The following parameters have been adopted for the analysis based on the result of geological investigation and standard unit weight of the materials:

Table 16: Considered input parameters for stability of the weir

Unit weight of materials	Value	Unit	Parameters	Value, m
Stone masonry, ϒm	23	KN/m^3	Top width, b	1.0
Water,ϒw	9.81		Bottom width d/s face, b'	2.0
Silt,ϒ'SV	8.19		height canal out let level from river bed, Hs	1.15
Friction coefficient, μ	0.6		Weir height, P	1.6
Angle of internal repose/Ø	30	degree	Flow depth over the weir crest, hd	2.21
Ka	0.33	degree	pre jump depth, y1	0.778
			Total Bottom width of weir , B	3.0
⇒ The foundation material beneath the weir is sandy soil with boulder so allowable bearing capacity of the foundation material is 200 KN/m^2 *Source; (MoWR, 2002)*				

Soil Type	Allowable Bearing Pressure KN/m²
Soft clays and silts.	<80
Firm clays and firm sandy clays.	100
Stiff clays and stiff sandy clays.	200
Very stiff boulder clays.	350
Loose well graded sands and gravel/sand mixtures.	100
Compact well graded sands and gravel/sand mixture.	200
Loose uniform sands.	<100
Compact uniform sands.	150

Note: - that for dynamic loads, a 25% overstress may be allowed

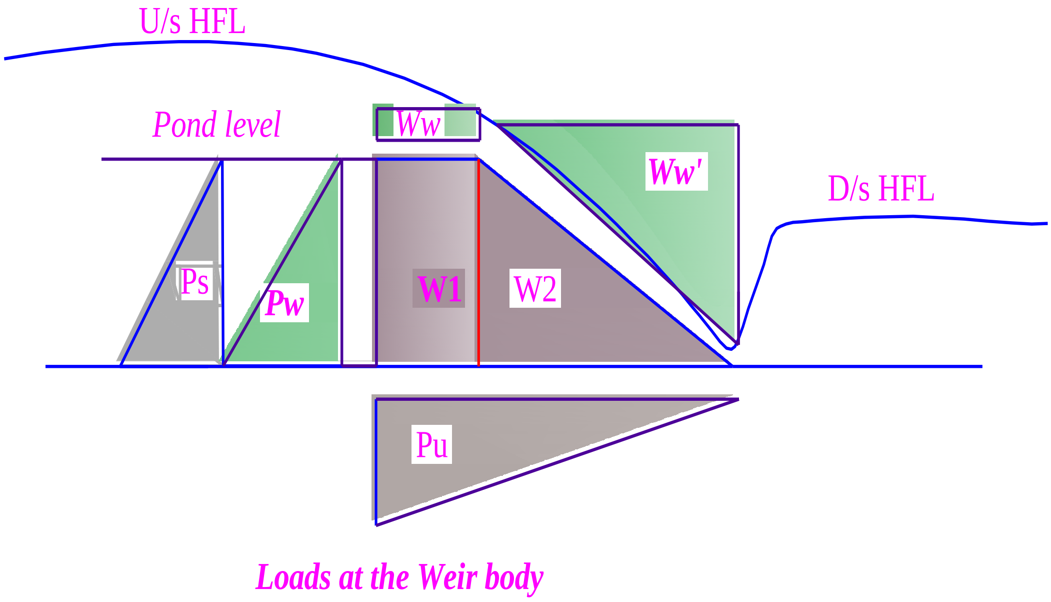


Figure 13: Force diagram and water surface profile of the weir

Table 17: Load analysis **Pond** Condition

Name of forces	Symbol	Description	Forces (KN)		Lever arm	Moments (KN-m)	
			Vertical (+)	Horizontal (-)		(+)	(-)
Weight of weir	W1	ϒm*B*H	36.80		2.50	92.0	
	W2	1/2*ϒm*b*H	36.80		1.33	49.1	
water load	Ph2	1/2*ϒw*H^2		12.56	0.53		6.7

Uplift pressure	Pu1	1/2*ϒw*B*H	15.70		2.0		31.4
Silt Pressure	Ps	1/2*ϒ'SH Ka*Hs^2		3.49	0.53		1.9
Sum			**89.30**	**16.05**		**141.1**	**40.0**

*Table 18: Factor of safety for **pond** condition*

Factor of safety against overturning (Fo),	Fo=(M+ve/M-ve)	***3.5***	***> 1.5***	ok
Factor of safety against Sliding (Fs),	Fs=U*Fv/Fh	***3.34***	***> 1***	ok
Factor of safety against Tension development	X = net moment/Fv	***1.1***		
e	B/2-X	***0.37***	***< B/6***	ok
Maximum and minimum stress at the toe and heal of the Weir,	P=(Fv/B)*(1+/-(6*e/B)	***51.65*** **7.88**	***< 200***	ok
B/6	**0.5**			

Table 19: Load analysis at **HFL** Condition

Name of forces	**Symbol**	**Description**	**Forces (KN)**		**Lever arm**	**Moments (KN-m)**	
			Vertical (+)	**Horizontal (-)**		**(+)**	**(-)**
Weight of weir	W1	ϒm*B*H	36.80		2.50	92.0	
	W2	1/2*ϒm*b*H	36.80		1.33	49.1	
Water load	W1	ϒw*b*Hd	21.68		2.50	54.2	
	W2	1/2*ϒw*B*H	23.5		1.3	31.4	
water load	Ph2	1/2*ϒw*H^2		12.56	0.53		6.7
Uplift pressure	Pu1	1/2*ϒw*B*H	15.7		2.0		31.4
Silt Pressure	Ps	1/2*ϒ'SH Ka*Hs^2		3.49	0.53		1.9
Sum			**134.52**	**16.05**		**226.7**	**40.0**

Table 20: factor of safety at HFL Condition

Factor of safety against overturning (Fo),	Fo = (M+ve/M-ve)	**5.67**	**>1.5**	ok
Factor of safety against Sliding (Fs),	Fs=U*Fv/Fh	**5.03**	**>1**	ok
Factor of safety against Tension development	X=net moment/Fv	**1.39**		
e	B/2-X	**0.11**	**< B/6**	ok
Maximum and minimum stress at the toe and heal of the Weir,	P = (Fv/B)*(1+/-(6*e/B)	**54.9** **34.8**	**< 200**	ok
B/6	**0.5**			

The result of stability analysis of the weir is safe against sliding, overturning, and tension development at pond and high flood level conditions which's indicated in the above table.

3.4 Hydraulic jump computation

3.4.1 Water Profile at the Weir Site

The water surface profile at downstream of the weir is required to carry out stability analysis of the weir; structural design of the weir; design of downstream retaining wall. The water profile at upstream of the weir is required to determine the height of the river banks upstream of the weir and to find out whether the water surface profile is high enough to deliver the required discharge into the off-taking canal. Conjugate depth and jump occurring level of the flow has been computed for different discharges.

Input data for the analysis:

- ✓ Peak discharge, Q = 98.05 m^3/s
- ✓ Unit Discharge, $q = \frac{Q}{L} = \frac{98.05}{16} = 6.13$ $m^3/s/m$
- ✓ Head over the weir, He = 2.34m
- ✓ Weir crest level = 2576.14m
- ✓ U/S TEL = 2578.48m

3.4.2. Water Profile on D/S of the Weir

The water depth at the weir toe can be determined by applying Bernoulli's equation between the weir crest and weir toe assuming that the jump forms on a horizontal surface and friction loss is negligible.

Initial depth of the jump y_1: $Z_o + h + H_e = Z_o + y_1 + \frac{{v_1}^2}{2g}$

Where y_1 = water depth (***initial depth of the jump***) at the weir toe

v_1 = velocity at the weir toe $v_1 = \frac{Q}{A} = \frac{q}{y_1}$

Where q = discharge intensity over the weir = $\frac{Q}{L}$= 98.05/16 = 6.13$m^3/s/m$

$$h + H_e = y_1 + \frac{q^2}{2\,g\,{y_1}^2}$$

$$1.6+2.34 = y_1 + (6.13)^2/(2*9.81*{y_1}^2)$$

$$y_1^{\,3} - 3.94*y_1^{\,2} + 1.915 = 0$$

By trial and error **y_1= 0.778m**

Substituting the given parameter and solving the foregoing equation:

v_1 = 6.13/ 0.778= **7.88m/s**

The Froude number at the weir toe: $Fr_1 = \frac{v_1}{\sqrt{g y_1}} = \frac{7.88}{\sqrt{9.81*0.778}} = 2.85; 2.5 < 2.85 < 4.5 =>$ *the flow is* Super critical and the jump is Oscillating jump with moderate energy loss

The sequent depth of the jump, y_2:

$$y2 = \frac{y1}{2}(\sqrt{1 + 8 * (Fr)^2}) - 1)$$

$$Y2 = \frac{0.778}{2}\left(\sqrt{1 + 8 * 2.85^2} - 1\right) = \mathbf{2.77m}$$

$$v_2 = \frac{q}{y_2} = \frac{6.13}{2.77} = \mathbf{2.211m/s}$$

$$Fr_2 = \frac{v_2}{\sqrt{gy_2}} = \frac{2.21}{\sqrt{9.81 * 2.77}} = \mathbf{0.42}$$

Critical depth $yc = \sqrt[3]{\frac{q^2}{g}}$

$$= \sqrt[3]{\frac{6.13^2}{9.81}} = \mathbf{1.56}m$$

Critical velocity $Vc = \frac{6.13}{1.56}$ = **3.93m/s**

The Froude number at the critical point:

$$Fr_c = \frac{v_c}{\sqrt{gy_c}} = \frac{3.93}{\sqrt{9.81 * 1.56}} = \mathbf{1.005}$$

=> Y1< Yc < Y2 and Fr1 > Fr_c> Fr2 Therefore The Froude number changes from 2.85 to 0.42, the state of flow downstream of the weir changes from supercritical to subcritical flow as a result hydraulic jump is formed downstream of the weir. The formation of hydraulic jump downstream of the weir is used to dissipate energy in water flowing over the weir and thus prevents the D/S structure from scouring.

Head loss $HL = \frac{(y2-y1)^3}{4*y1*y2} = \frac{(2.54-0.95)^3}{4*0.778*2.77} = \mathbf{0.92m}$

The state of flow downstream of the weir changes from supercritical to subcritical flow as a result hydraulic jump is formed downstream of the weir. The formation of hydraulic jump downstream of the weir is used to dissipate energy in water flowing over the weir.

Jump length

The length of the jump can be estimated in terms of sequent depth for a given Froude number as presented in (Garg, 2005)

L = 5(y2-y1) = 5*(2.77 – 0.778) = **9.96m**

Since, the jump length is used *for reducing the effect of scouring on the weir toe; the length of D/s apron from the toe of the weir will not less than* ***10m,*** *since the river bed geological formation is composed of weathered and fractured rock.*

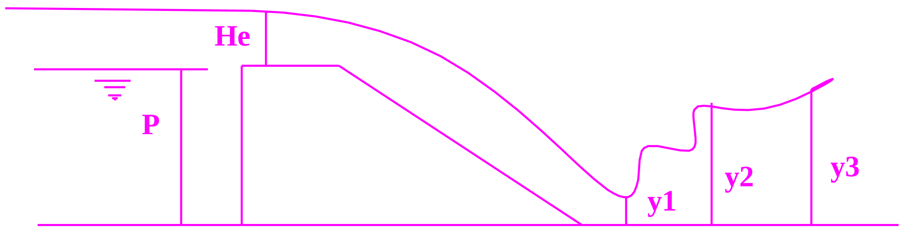


Figure 14: Hydraulic jump profile

3.4.3. Water profile on U/S of the weir

The water profile u/s of the weir helps to analyze the effect of constructing the barrier (weir) on u/s water surface. The water profile u/s of the weir can be computed by approximate method (Baban, 1995):

$Y = \frac{(XS - 2\Delta_o)^2}{4\Delta_o}$ Where;

Y = water rise at a distance X above normal water depth

X = distance from the crest of the weir to the point where y is to be determined

Δ_o = rise of water above normal depth of weir site (X = 0)

Δ_o = U/S HFL – D/S HFL $= 0.96m$

S = Slope of the river = 0.021

$Y = \frac{(0.021*X - 2*0.96)^2}{4*0.96}$ Thus for Y = 0, X = **91.4m**

The effect of constructing the weir on the U/S will be diminished **91.4m** U/S of the weir axis and there is no infrastructure on the river built within this reach. The U/S and D/S river banks in both sides should be protected with retaining walls and apron with appropriate cutoff.

3.5 Scour depth determination

3.5.1 Vertical Cutoff

Vertical cutoff at the upstream and downstream ends of a weir are always provided to guard against scouring at the upstream and downstream ends and the piping effects at the downstream end. The depth of scour below HFL, The regime scour depth, Rs is estimated by *Lacey's formula* as follow;

If the actual waterway provided is less than or equal to the regime width

$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$ Where, q = Unit discharge

f = Silt factor, and d = Average riverbed material diameter (mm).

$$f = 1.76\sqrt{d} = 1.76\sqrt{1.3} = 2.0\ for\ fine\ gravel\ materials$$

$$R = 1.35\left(\frac{6.13^2}{2}\right)^{\frac{1}{3}} = \mathbf{3.59m}$$

Table 21: Particle size, d (mm) for various types of alluvial materials (European Union's, Design manual volume 1, 2008)

Material	Average d_{50} size (mm)	Silt factor f
Very fine SILT	0.05	0.4
Fine SILT	0.12	0.5
Medium SILT	0.15	0.7
Standard SILT	0.32	1.0
Medium SAND	0.50	1.2
Coarse SAND	0.72	1.5
Fine GRAVEL	1.30	2.0
Medium GRAVEL	7.30	4.7
Heavy GRAVEL	26	9.0
Small BOULDERS	50	12.0
Medium BOULDERS	72	15.0
Large BOULDERS	185	24.0

Table 22: Recommended value of score depth

Condition	Value
In a straight reach	1.25R
At a moderate bend, for example, along apron of a guide bank	1.50R
At a severe bend	1.75R
At right angle bends or at noses of piers	2.00R
In severe swirls, for example head of a guide bund.	2.50R

(Source, MOA guide line on irrigation structures)

U/S Cutoff

- *Depth of upstream cutoff below the river bed. d1 = 1.25R-(h+H_d)*

  *= 1.25*3.59 – (1.6+2.21) =* ***0.78m***

Depending on the river bed weathered rock foundation, take ***1.0m*** *cutoff depth to control seepage through the foundation of the weir.*

D/S Cutoff

- *Depth of downstream cutoff below the river bed, d2 = 1.5R-y3 Where, y3 = DSHFL– River bed.*

  *D2 = 1.5*3.59– 2.85 =* ***2.54m***

*Depending on the downstream river foundation, hard rock formation after 1.5m depth, take 1****.5m*** *cutoff depth to make a smooth surface.*

Thickness of the cutoff

The thickness of cut-off for both U/S and D/S and at the toe of the weir is provided 0.3m to protect leakage underneath the structure.

3.6 Determination of Upstream and Downstream Apron

3.6.1 Total length of the floor

The total length of the floor (including twice the length of cut-off, if provided) as per Bligh's:

$L = C\ H_L$= 7*1.6= 11.2

Where:

H_L = seepage head = difference in water levels upstream & downstream of the weir (weir height)

C = Bligh's coefficient for the soil, taken from the table below for coarse grained sand materials

L = total creep length = $2d_1 + 2d_2 + 2d_3 + B + L_U + L_D$ =2*1.0+2*1.5+3.0+1+10 = 19.00m

d_1 = Upstream cut-off depth = 1.0 from the recommended value

d_2 = Downstream cut-off depth= 1.5m from the recommended value

B = Bottom width of the weir = 3.0m

L_U = Length of upstream impervious apron = 1m

L_D = Length of downstream impervious apron: = 10m

Table 23: Recommended value of Bligh coefficient and safe hydraulic gradient

Type of soil	Creep coefficient (C)	Safe hydraulic gradient
Light sand and mud	18	1/18
Fine sand	15	1/15
Coarse grained sand	12	1/12
Boulder or shingles and gravel mix sand	5 to 9	1/5 to 1/9

The exit gradient according to the creep flow theory proposed by Bligh:

$$Ge = \frac{HL}{L}$$

Where; L = total creep length

d_1 = the depths of the upstream cut-off piles

d_2 = the downstream cut-off piles

d_3 = the intermediate cut – off pile

b = horizontal floor length between the two piles

H_s = seepage head, is the difference in the water levels upstream and downstream of the intake

$Ge = \frac{HL}{L} < \frac{1}{C} = \frac{HL}{19.00} < 1/7$, => HL < 19.00/7 > 1.6, i.e. 2.7 > 1.6

Ge = 1/7 from recommended value of Bligh safe hydraulic gradient

Length of impervious floor (apron)

The length of downstream impervious floor (apron) as per Bligh's empirical formula: For weir have no crest shutter

$LD = 2.21\, C\, \sqrt{(HL/10)} = 2.21 * 9\sqrt{(1.6/10)}$ = **7.9m;** considering river bed geological condition /weathered rock/ and moderate energy loss take ***10m*** length including D/S cut off thickness to prevent scouring due to jump and the weir toe from cavitations.

The upstream impervious floor can be taken as the balance length:

To minimize seepage underneath the weir take **1m** length for the u/s apron including U/S cut-off thickness.

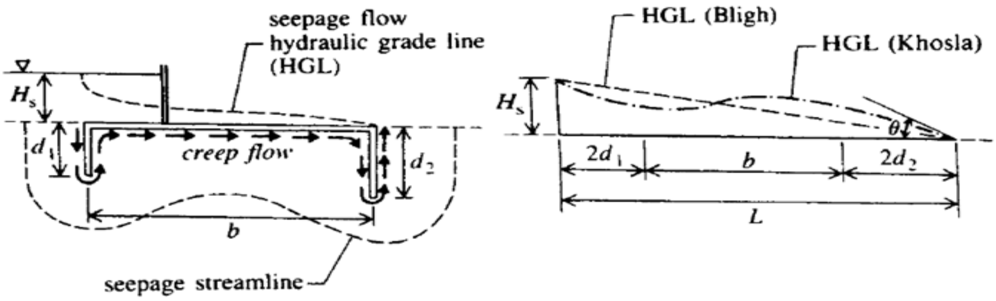


Figure 15: Seepage flow hydraulic gradients

3.6.2 Residual Head and Apron Thickness

The residual head at the d/s point of the weir section h' can be calculated as

Weir height

$$h = h - \frac{h}{LR}(2d1 + LU + B)$$

Where: h' = residual head at the downstream end point of weir (at point x):

Total creep length (LR) required as per Bligh

LR = 2d1+2d2+B =2*1.0+2*1.5+ (10+3.0+1) = 19.00

d_1&d_2 = u/s & d/s cut-off depth

B = the distance b/n d1 and d2

h' = weir height

$residual\ head\ at\ the\ toe\ of\ the\ weir,\ h' = 1.6 - (\frac{1.6}{19})(2 * 1.0 + 1 + 3.0)$ = 1.09m

$residual\ head\ at\ 3.5m\ from\ the\ weir\ toe,\ h' = 1.6 - (\frac{1.6}{19})(2 * 1.0 + 1 + 3.0 + 3.5)$ = 0.8m

$residual\ head\ at\ 7m\ from\ the\ weir\ toe,\ h' = 1.6 - (\frac{1.6}{19})(2 * 1.0 + 1 + 3.0 + 7)$ = 0.51m

$residual\ head\ at\ 10m\ from\ the\ weir\ toe\ or\ at\ the\ end\ of\ \frac{D}{S} apron,\ h' = 1.6 - (\frac{1.6}{19})(19)$ = 0.0m

Thickness of the floor

The downward water pressure is always higher than the uplift pressure in the region of the upstream side of the weir. The thickness of the upstream apron can be based on the practice of the construction and the perfection of leakage proofing 0.5m thick is usually sufficient for this purpose in case of fractured rock foundation.

For the downstream apron, the thickness to be determined depends on;

$$t = \frac{4}{3} * \frac{h'}{(G-1)}$$

$$t1 = \frac{4}{3} * \frac{1.09}{2.3-1} = 1.12m$$

$$t2 = \frac{4}{3} * \frac{0.8}{2.3-1} = 0.82m$$

$$t3 = \frac{4}{3} * \frac{0.51}{2.3-1} = 0.52m$$

3.7 Head regulator (canal outlet)

The head regulator is provided on the left side to;

- ✓ Control the entry of silt into the canal
- ✓ regulate supply water to the canal
- ✓ Shut out the river floods

The sill level of this head regulator is fixed from different point of views. The main conveyance system is more than 0.5km, which passes from different angle of observations. The discharge through the regulator is computed by rectangular weir formula:

$$Q = C\,LH^{3/2}$$

Where; Q = Discharge of the opening portion (m^3/s)

C = Coefficient of discharge = 1.71

L = Width of the opening portion (m), 0.3m

H = flow depth on off take crest (m), 0.2m

Substituting the above values, then **Q = 0.046m^3/sec**, Provide **0.3m* 0.3m** for the opening of the off take canal gate

To minimize the problems in related with canal alignment and for free flow of water in to the off-taking canal take 0.15m working head above the crest level of opening.

⇒ To admit comparatively silt free water to the intake, the invert level of the intake is raised by 0.6m above the river bed center.

Invert level of the orifice = 2574.54+1.15 = **2575.69m.a.s.l.**

The bed level of the off-taking canal is fixed at the invert level of the intake:

The full supply level of the off-taking canal:

FSL = canal bed level + d

Where d = water depth in the off-taking canal = 0.3 m

FSL = 2575.69+0.3 = 2575.99masl

The working head available, H_{av} = Pond level/weir crest level/ – FSL

2576.14 – 2575.99 = 0.15m

The provided working head required to pass the design discharge is 0.15m hence the area of orifice (0.3 m width x 0.3m height) Sliding gates embedded in the body of retaining wall and canal wall is sufficient by adding groove insertion (0.4m*0.35m) canal regulator is needed.

3.7.1 Canal Design Discharge

It is the maximum discharge for which the canal is designed, if the sum of the total crop waters requirement and various losses in conveyance of water.

***Calculation of design discharge and* parameters**

Irrigation requirement for actual area (D) = 1.03 l/s/ha

Manning's coefficient (n) = 0.018(for masonry)

Total irrigable area = 24ha

Project efficiency E_p, 56.7%

$$Design\ Discharge\ need\ for\ the\ command = D * A$$

$$= 1.03 * 24$$

$$Qd\ =\ 0.02472 m^3/s$$

$Design\ Discharge\ needed\ for\ the\ command\ area = \mathbf{0.02472 m^3}$**/sec**

Area of the canal outlet (A) = bd

Wetted perimeter (P) = b+2d Where; d = water depth; b = bed width

From the recommended value b/d =1

Table 24: Recommended Values of b/d ratio and Velocities

Status	Discharge m^3/s	Total Freeboard (m)	Lining Freeboard (m)
Small scale projects	< 0.1	0.2	0.1
	0.1 - 0.5	0.3	0.15
	0.5 - 1.0	0.4	0.2

$A = d^2$

P =3d

Take canal bed slope=1/1000 (consider to irrigate an elevation of 2575.19m. a.s.l. or less).

Table 25: Longitudinal Slope

Type of canals	Range of Slope
Main canals	1/700-1/1,500
Secondary canals	1/700-1/1,000
Teritary canals	1/500-1/700
Field ditches	1/300-1/500

Table 26: Recommended values of maximum velocity & roughness coefficient (MoWR)

Lining type	Max. velocity (m/s)	Roughness "n"
Dry stone	1	0.025
Dry brick	1	0.02
Dressed masonry	***2***	***0.018***
Brick	*1.5*	*0.017*
Random Rubble	1.5	0.020
Unreinforced concrete	2.5	0.015
Concrete	2.5	0.017
Buried membrane and Earth	Design as unlined canals	

Using Manning's formula: $Q = \frac{1}{n} * AR^{\frac{2}{3}} * S^{\frac{1}{2}}$

$$0.02472 = \frac{1}{n} * (d^2) * \left(\frac{d}{3}\right)^{\frac{2}{3}} * \sqrt{0.001}$$

$$0.02472 = \frac{1}{0.018} * \frac{d^{\frac{8}{3}}}{2.08} * \sqrt{0.001}$$

$$0.02472 = \frac{1}{0.018} * \frac{d^{\frac{8}{3}}}{2.08} * \sqrt{0.001}$$

$$\mathrm{d} = 0.27\mathrm{m}, \text{take } \mathrm{d} = 0.3\mathrm{m}$$

$$b = 0.27 \approx Take\ \mathbf{0.30}\boldsymbol{m}\ to\ be\ easy\ for\ construction\ work$$

Take a free board of ***0.2 m***; $D = 0.2 + 0.3 = \mathbf{0.5}\boldsymbol{m}$

Thickness for stone masonry

t = 0.3m for canal capacity (Q_d) < $3m^3/s$

3.7.2 Canal Intake Slab

To carry the weight of retaining wall above canal out let RCC slab is provided.

This slab is having a dimension of 1.5m x 3.5m and to intake water into the canal free from risk.

The thickness of the slab provide t = 0.20m = 20cm with reinforcement bar of φ 12mm at 200mm c/c spacing.

3.8. Design of Under Sluice

The under sluice is mainly provided here to remove silt deposition as a result of barrier structure. Hence the sill level of the under sluice is fixed to facilitate this deposited silt to increase the efficiency of water abstracting to the main canal through the head regulator from the pocket. The sill level of this sluice is fixed at the minimum bed level. Hence the sill level of the under sluice = 2574.54m. Even if the position of the under sluice is on concave side that is on scouring side, there might be boulders that may come into the pocket of the under sluice due to the barrier structure. Hence in addition to the supply of water to the intake and the removal of silt, this acts to remove the boulder that comes to wards it. Considering this, the opening size of the gate is 1.6m*1.2m with spindle type operating from the operation slab. Considering rectangular notch profile of flow of water at the under sluice, the discharge passing is computed using the rectangular orifice formula.

- The capacity should be at least five times the canal discharge to ensure proper scouring.
- Capacity of passing about 10% to 20% of the maximum flood discharge at high floods.
- During construction, it should be able to pass the prevailing (at least base flow) discharge of the river.

But the practical application of those values is not sound so the discharging capacity of under sluice is determined by using weir formula for maximum flood condition.

$$Q = Cd * A * \sqrt{2 * g * he}$$

$$A = L \times H$$

Where C_d - discharge coefficient, mostly taken as 0.60 for rectangular orifices

H - The depth of the sluice, m (1.6 meter)

L - Width of the sluice, m (1.2meter)

h- The difference between upstream and downstream water level, mostly b/n 0.15 &0.25m

Q = 0.6*1.6m*1.2m*(2*9.81*0.2m) ^0.5 = ***2.28 m^3/s***

Q ≈ 2.3 m^3/s

Since the canal design discharge in supplementary irrigation *is **0.02472m^3/sec; 5*0.02472 = 0.124m^3/sec. (i.e. 2.3m^3/sec > 0.124m^3/sec. …… Ok!)***

And the calculated discharge amount ascertains that the under sluice opening can tolerate more discharge than double of the main canal capacity. Thus, a gate size of 1.6 m*1.2m including the grooves depths with spindle type that will be operated from the operation slab should be installed. Hence, during non-rainy time, it is possible to flush the silt easily when required.

The critical case in the case of under sluice is during non-flow condition. The high flood condition is expected during summer when water is not required for irrigation and the under sluice should be fully opened the gate is well protected by concrete breast wall.

3.9. Retaining Wall Design

The height of retaining wall (top level) is determined in accordance with upstream and downstream water elevation by incrementing an appropriate freeboard value.

3.9.1 Height of Retaining Wall

The maximum design flood and flood jump height govern the height of retaining wall in addition to free board provided. The downstream retaining wall is based on the *level of the tail water depth* and free board. However the upstream retaining wall design is based on the *high flood level* on the upstream and free board.

Upstream Retaining Wall Height

The Retaining wall height should at least reach the u/s HFL to protect over flow and allow gate operating during floods.

Upstream retaining wall crest level = U/S HFL + FB_U = 2578.35 + 0.5 = 2578.85m

Where; FB_U = Freeboard at upstream side, the minimum freeboard to be provided upstream of the weir at the maximum design discharge must not be less than 0.5 m (Baban, 1995)

The river attains maximum water level on the u/s side of the weir when the peak flood passes through the weir section.

River bed level = 2574.54m

⇒ Height of U/S Retaining wall above the river bed, H_U = 2578.85 – 2574.54 = 4.3m

Total height of U/S RW including the foundation, H_{TU} = Hu + D = 4.3+ 0.2 = **4.5m**

Where; D = 0.2m: is the depth of foundation below the river bed level for U/s.

Downstream Retaining Wall Height

Height of D/S RW above the river bed = Y_3 + FBj

Where; Jump height Y_3 = 2.85m

FB_J = The freeboard on the d/s due to jump is added to the side wall so that it will not be over topped by surges, splash and sprays and wave action set up by turbulence of the jump.

Y_3 = Tail water depth of the jump = 2.85m

Fb = 0.5

⇒ Height of d/s retaining wall from jump consideration, H_D = 2.85 + 0.5= 3.35m,

The total height of d/s retaining wall including the foundation: HT_D = H_D + D = 3.35+ 0.2= **3.6m**

Where; D = 0.2m: is the depth of foundation below the river bed level for D/s.

3.9.2 Section of retaining wall

Top Width

b = 25 cm to H/12

⇒ Top width of u/s retaining wall, $Tu = \frac{4.5}{12} = 0.375m$

b_U = **0.3m** (adopted) and the length of the retaining wall u/s of the weir is **10.5m** in right side and **14.8m** in left side *including the weir bottom width and u/s apron and cutoff width.*

⇒ Top width of d/s retaining wall,

T_D =3.6/12=0.3 m *take* **0.3m** *to be safe during construction work*

Length of d/s retaining wall from the weir toe is taken **11m** for both the right and left sides.

Bottom width

Bottom width of retaining wall: B = 1/2 H to 2/3 H

Bottom width of u/s retaining wall, B_u = 1/2 x 4.5 to 2/3 x 4.5 = 2.25m to 3m

Take B_u= **2.5 m**

Bottom width of d/s retaining wall Bd= ½*3.6 to 2/3 *3.6

1.8 to 2.4 m ;Take Bd = **2m**

The retaining wall bed width values are taken after checking the stability.

3.9.3 Stability analysis of the retaining wall

Retaining wall is designed first to carry the earth pressure at the back of the wall. The critical condition in the stability of the wall will be occurred when there is no flow in the u/s side while the wall retains the earth at the back of the wall. Accordingly, the stability analysis will be carried for this loading condition for both u/s and d/s retaining wall. The following parameters are adopted for the analysis:

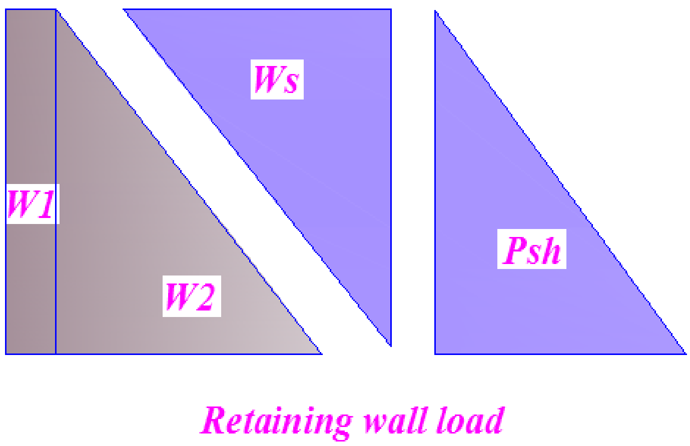


Figure 16: Exerted Load on the retaining wall

Table 27: Retaining wall parameters

Parameters				
	U/s retaining wall		D/s retaining wall	Units
Top width, b	B	0.30	0.3	M
Bottom width, b	B	2.5	2	M
Height of wall	H	4.5	3.6	M
Foundation Thickness	D	0.2	0.2	M
Length	L	14.8	11	M
Y_m of Masonry	γ_m	23	KN/m^3	
Ys of silt (Fill)	γ_s	17	KN/m^3	
Y_{co} of Concrete	γ_c	24	KN/m^3	
Angle of Repose	F	30	Deg.	
Ka=(1-sinf)/(1+sinf)	Ka	0.33		
Bering capacity of Foundation material gravel material = **200KN/m^2**				
Coefficient. Of friction, **η = 0.65**				

Table 28: Stability analysis of U/S retaining wall

Item Description	**Forces(KNm.)**				**Lever arm (m.)**	**Moment about toe (KN.m)**	
	Vertical		**Horizontal**				
	+ve	**-ve**	**+ve**	**-ve**		**Resisting**	**Overturning**
1. Vertical force							
Self weight(W1)	31.05				0.15	4.66	
Self Weight(W2)	113.85				1.03	117.65	
Silt pressure(Ps2)	84.15				1.77	148.67	
2. Horizontal force							
Active pressure (Ps1)				28.61	1.50		42.92
Total	**229.05**			**28.61**		**270.97**	**42.92**

Table 29: Factor of safety values for U/S retaining wall both sides

Over turning = (M+ve/M-ve) >1.5	Fo	**6.31**	**>1.5**	**Ok**
Sliding = (µxFv/Fh), >1	Fs	**5.2**	**>1**	**Ok**
Tension: X= (Net Moment/Sum Fv)	X	**0.99**		
e = B/2-x	e	**0.12**	**<B/6**	**Ok**
B/6	0.42			
Max & Min. compression stress at the toe	**$P_{max.}$**		**118.09**	**Ok**
	$P_{min.}$		**65.15**	**Ok**

Table 30: Stability analysis of downstream retaining wall on both sides

Item Description	Forces(KNm.)				Lever arm	Moment about toe (KN.m.)	
	Vertical		Horizontal				
	+ve	-ve	+ve	-ve	(m.)	Resisting	Overturning
1. Vertical force							
Self weight(W1)	24.84				0.15	3.73	
Self Weight(W2)	70.38				0.87	61.00	
Silt pressure(Ps2)	52.02				1.43	74.56	
2. Horizontal force							
Active pressure (Ps1)				17.6868	1.20		21.22
Total	**147.24**			**17.69**		**139.28**	**21.22**

Table 31: Factor safety values for the downstream both sides

Over turning = (M+ve/M-ve) >1.5	Fo	***6.56***	***>1.5***	**Ok**
Sliding = (µxFv/Fh), ---µ=0.65 >1	Fs	***5.41***	***>1***	**Ok**
Tension: X= (Net Moment/Sum Fv)	X	***0.8***		
e = B/2-x	e	***0.09***	***<B/6***	**Ok**
B/6	0.33			
Max & Min. compression stress at the toe	$P_{max.}$		***93.52***	**Ok**
	$P_{min.}$		***53.72***	**Ok**

3.10. Divide wall design

The divide wall allows a silt free water flow to towards the head regulator (Outlet) direction by facilitating removal of the bed load deposition through the under sluice pocket during flood season. The divide wall is provided between the sluice gate and the weir body and Separate the under sluice and the weir flow section and support operation slab. The under sluice and canal outlet regulating gates are sliding gate with operation slab. The gate here provided is simple and manually operated.

And this wall protects the side movement of silt from the weir to the head regulator. The foundation is hard rock formation, so it is better to anchor the divide wall to the hard rock to a depth of about one meter. Whereby, this wall was designed as plain concrete type rather than masonry in order to form a monolithic structure that strongly keyed with foundation and the weir body.

Hydraulics of divide wall

The critical case for the determination of the wall dimensions is when there is maximum flood and hence the U/S HFL, from the earlier calculation,

- U/S HFL = 2578.35m.a.s.l.

- U/S bed level = 2574.54 m.
- Maximum wall height = U/S HFL - river bed level
- Maximum wall height = 2578.35 m- 2574.54m = 3.81m, take.4.5m wall height with foundation height/equal height with U/S retaining wall.

Thus, provide wall height of 4.5m for a length of 2.0 m at upstream side from the downstream edge of the top width of the weir body.

Stability Analysis for U/S Divide Wall

The force components of this structure are its self weight, silt, water and uplift pressure.

The critical case is during pond level condition when the weir side of the wall is exposed for water and silt pressures while the sluice side is counter balanced (supported) with lesser water and silt loads. Whereas during the hydro dynamic condition, the flow analysis shows that the structure is fully submerged by flood and so almost both sides are equally counter balanced by water pressure. However, when the construction is commenced the fractured layer of the rock will be removed in depth even below the foundation of the wall and the rest part is going to be filled and compacted with concrete.

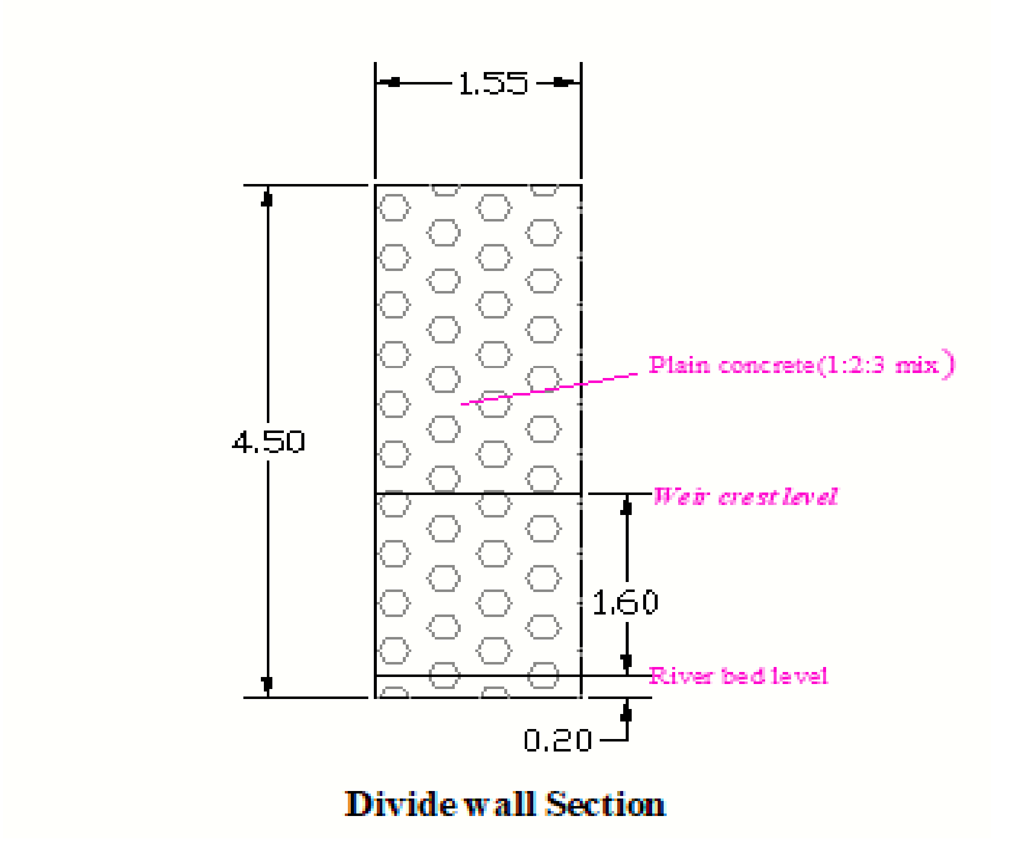


Figure 17: Divide Wall Section

Table 32: **Input Parameters for Load Analysis of divide wall**

Total height(m)	Width(m)	Weir Side Water and silt depth(m)	Total length(m)	γ of Water(KN/m^3)	γ of silt(KN/m^3)	γ of Concrete(KN/m^3)
4.5	1.55	1.6	2.0	9.81	10.84	24

Table 33: Stability Analysis for the Divide Wall and Safety Factors

Name of forces	Symbol	Description	Forces (KN)		Lever arm	Moments (KN-m)	
			Vertical	Horizontal		(+)	(-)
Weight of weir	W	ϒm*B*H	167.40		0.78	129.7	
water load	Ph2	1/2*ϒw*H^2		12.56	0.53		6.7
Uplift pressure	Pu1	1/2*ϒw*B*H	12.16		0.78		9.4
Silt Pressure	Ps	1/2*ϒ'S*Hs^2		13.88	0.53		7.4
Sum			**179.56**	**26.43**		**129.7**	**23.5**

Factor of safety against overturning (Fo),	Fo=(M+ve/M-ve)	***5.5***	***>1.5***	***ok***
Factor of safety against Sliding (Fs),	Fs=U*Fv/Fh	***4.08***	***>1***	***ok***
Factor of safety against Tension development	X = net moment/Fv	***0.6***		
E	B/2-X	***0.18***	< ***B/6***	***ok***
Maximum and minimum stress at the toe and heal of the Weir,	P=(Fv/B)*(1+/-(6*e/B)	***198.14*** ***33.55***	< ***200***	***ok***
B/6 = 0.26				

4. IRRIGATION INFRASTRUCTURES

4.1. Irrigation Area Description

4.1.1. Topography

Topography is an important factor for the planning of any irrigation project as it influences method of irrigation, drainage, erosion, mechanization, and cost of land development, labor requirement and choice of crops.

The topographic feature of the project command area is mainly gently slope type. However, it has identified to be suitable for surface irrigation. Nevertheless, it requires soil and water conservation measures or structures (i.e. constructing bunds, bio-physicals, check dams, artificial water ways, etc). The project command area is situated at left side of Gilgel Cheye River (to the North side of the river flow). The natural topographic feature of the command area has inclined north to south direction.

4.1.2. Soil characteristics

Soil properties (physical, chemical, etc.) greatly influence the growth and thereby yield of crops which is grown. The command area has predominantly clay textured of black cotton soils which can be classified as moderately drained soil. Most of the study area soils are categorized as deep soil (1-1.5 meter depth). Soils of the command area are suitable for most of the selected crops to be grown.

4.2. Irrigation Water Requirement

4.2.1. Crop Water Requirement (CWR)

The calculation of crop water requirement is a very important aspect for planning of any irrigation project. Several methods and procedures are available for this. The Food and Agriculture Organization (FAO) of the United Nations has also made available several publications on this subject and other issues related with this. The computer program available in FAO Irrigation and Drainage Paper No. 56 “CROPWAT” has been used for the calculation of Crop Water requirement. This program is based on Penman-Monteith approach and procedures for calculation of crop water requirements and irrigation requirements are mainly based on methodologies presented in FAO Irrigation and Drainage Paper No. 24 “Crop Water Requirements” and No. 33 “Yield Response to Water”.

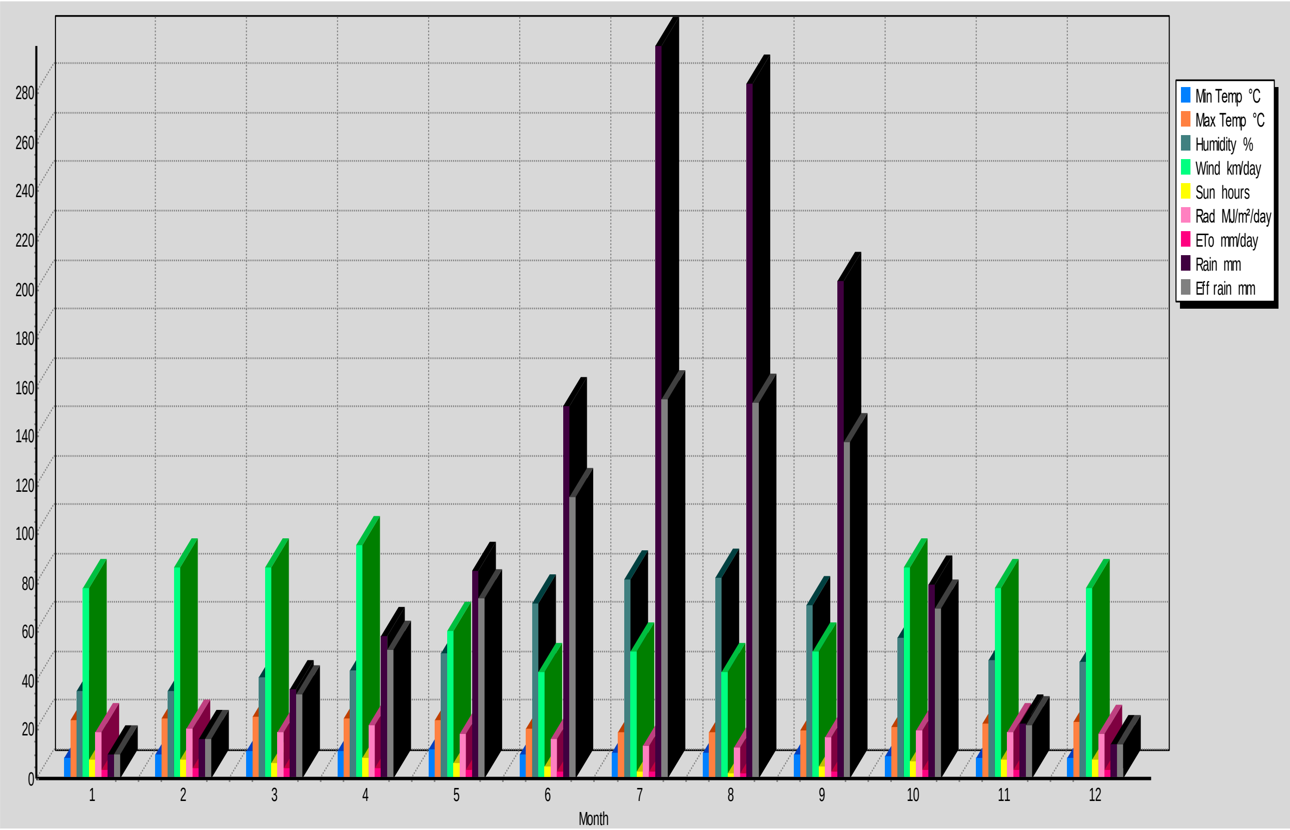


Figure 18: Climate and rainfall data arrangement (results from CROPWAT)

4.2.2.Irrigation efficiency (E_p)

To complete the evaluation of the demand, the efficiency of the water distribution system and of application must be known.

The gross requirement of water for irrigation system is very much dependent on the overall efficiency of the irrigation system, which in turn is dependent on several factors: Method of irrigation, type of canal (Lined and/or Unlined), method of operations (simultaneously and continuous or Rotational water supply), and availability of structures (for controlling and distribution and measuring and monitoring).

Table 34: Conveyance (EC), Field canal (EF), Distribution(ED) and Application efficiency (EA)

No.	Types of efficiencies	Recommendations by ICID/ILRI		
1.	Conveyance efficiency (Ec)			
	a) Continuous supply with no substantial change in flow rotational supply in projects of 3000-7000ha & rotation areas of 70-300ha, with effective management		0.9	
	b) Rotational supply in schemes (>1000ha) and (<1000ha) with respective problematic communication and less effective management		0.8	
	b.1 based on predetermined schedule		0.7	
	b.2 based on advance request		0.65	
2.	Field canal efficiency (E_f)			
	a) Blocks larger than 20ha			
	Unlined		0.8	
	Lined		0.9	
	b) Blocks up to 20ha			
	Unlined		0.7	
	Lined or piped		0.8	
3.	Distribution efficiency (Ed = Ec.Ef)			
	Average for rotational supply with management and communication			
	a) Adequate		0.65	
	b) Sufficient		0.555	
	c) Insufficient		0.44	
	d) Poor		0.3	
4.	Field application efficiency (Ea)	USDA	US(SCS)	
	a) For surface irrigation methods			
	Light soils	0.55		
	Medium soils	0.70		
	Heavy soils	0.60		
	b) Graded border	-	0.6-0.75	0.53
	Basin and level border	-	0.6-0.8	0.58
	Contour ditch	-	0.5-0.55	-
	Furrow	-	0.55-0.70	0.57
	Corrugation	-	0.5-0.70 (0.8)	-
	c) Subsurface irrigation system			
	Sprinkler, hot climate	-	0.6	-
	Moderate climate	-	0.7	0.67
	Humid and cool	-	0.8	0.32

(**Source: Ministry of water resources design guideline on irrigation system, June, 2002)**

On the basis of the above factors and recommendations, the project has planned to impose surface irrigation method (using furrows) and the canal systems are lined. Hence, the conveyance efficiency has been estimated to be 90%, distribution efficiency 63%, field application efficiency 70%, and field canal efficiency of 90%. As a result of these the overall irrigation efficiency has been estimated to be

$$E_P = E_C*Ea*E_f = 0.9*0.7*0.9 = 56.7\%.$$

E_p = 56.7%

4.1.1. Irrigation duty

Flow
Duty

Irrigation duty is the volume of water required per hectare for the full flange of the crops. Moreover, it helps in designing an efficient irrigation canal system.

The area, which will be irrigated, can be calculated by knowing the total available water at the source and the overall duty for all crops required to be irrigated in different seasons of the years.

The proposed cropping pattern of Gilgel Cheye diversion irrigation project has showed a maximum net irrigation water requirement (NIWR) in the month of February with the amount of 3.8mm/day for 24 working hours (for overall proposed crops).

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation deficit												
1. MAIZE (Grain)	106.6	112.6	50.7	0	0	0	0	0	0	0	3.5	30
2. Small Vegetables	100.7	89.3	0	0	0	0	0	0	0	0	11.6	64
3. Potato	103.2	107.2	85.3	1.9	0	0	0	0	0	0	7.6	43.2
Net scheme irr.req.												
in mm/day	3.4	**3.8**	1.6	0	0	0	0	0	0	0	0.2	1.3
in mm/month	104.4	106.3	50.9	0.6	0	0	0	0	0	0	6.4	40.8
in l/s/h	0.39	0.44	0.19	0	0	0	0	0	0	0	0.02	0.15
Irrigated area	100	100	80	30	0	0	0	0	0	0	100	100
(% of total area)												
Irr.req. for actual area	0.39	0.44	0.24	0.01	0	0	0	0	0	0	0.02	0.15
(l/s/h)												

However, for the designing of the irrigation water application and the flows in the entire canal systems, from the overall proposed crops the one that has maximum NIWR was used for irrigation duty calculation. Accordingly, maize has showed the maximum NIWR (i.e. 3.8mm/day); and hence taken for the irrigation project duty calculation as indicated here below:

For the designing of the project, the GIWR is given as follows:

GIWR = 3.8/0.567= 6.702 [mm/day]

The Net Irrigation Water Requirement NIWR, 3.8 mm/day, represents the daily quantity of water that is required to be applied. This water quantity is also used for the determination of the canal discharge in consideration of the time of flow and is defined as the duty, expressed as l/s/ha.

The duty is calculated by:

The duty is calculated by: $$q = \frac{10000 * GIWR}{(t * 60 * 60)}$$

Where: **q** = unit design discharge (lit/ sec. /ha.)

GIWR = Irrigation Requirement [mm/day]

t = 18, Irrigation hour per day (hr)

The duty for the NIWR of 3.8mm/day and 18 hours of daily irrigation time is supported to be used with furrow irrigation method. Hence, Duty for 18working hours is computed as follows:

$$q = \frac{10000 * 6.702}{18 * 60 * 60} = 1.03 lit / \sec / ha$$

4.1.2. Irrigation method

Among the different irrigation systems Surface irrigation system will be used for the project area; and the irrigation water will be obtained from Gilgel Cheye River and by constructing diversion weir structure to convey the water through main canals and then leading to field canals; and finally irrigation takes place mostly in furrows.

For this project, among the various irrigation methods, surface irrigation method has been selected. Of the surface irrigation methods furrow, border and basin irrigation methods can be used to supply irrigation water to the plants/crops. However, each method has its own advantages and disadvantages. Care should be taken when choosing the method which is best suited to the local circumstances, i.e., depending on slopes, soil types, selected crop types, amount of water available, etc. of the command area.

Based on the above factors surface irrigation method has been proposed for the proposed crops in this project. The method allows applying light irrigation and can be laid out in sloping fields along the contour. Furrow irrigation method is best suited for most of the proposed and row planted crops. In general, furrow irrigation method is simple, manageable and widely practiced irrigation method. This method is suitable for row crops that cannot stand in water for long periods. The only thing required to use this method is row planting of crops. Besides, basin and border irrigation method would be used for the non-row planted crops. Rotational flow water distribution is also recommended for the project area.

4.2. Irrigation and Drainage System Layout

The irrigation system layout for the project is prepared taking the following points into consideration besides other factors.

- ✓ A primary concern in the layout of the system is that it serves the purpose of conveying and distributing water to the command area.
- ✓ The excavation and earth fill volumes not be excessive, otherwise the construction costs can be tremendous.
- ✓ The selection of longitudinal bed slope is made taking into account the existing slopes of the terrain, so as to minimize deviations in canal routing.
- ✓ Curves in canals should not be too sharp.

The proposed irrigation system layout is in the left side of Gilgel Cheye River, and there is no secondary, tertiary and other canal as shown on the layout Drawings. The main canals run for most of its length parallel to the contours and several changes of direction are necessary to follow the topography.

The conveyance system is by constructing the main canal to irrigate total command area of 24ha. The main canals start from Water abstraction sites on left side. Main canals are aligned along contours and supplies directly to unlined field canals through turn out gates.

4.2.1. Design of Main Canal

The main canal is masonry lined for a length of 350 meters starting from the off-take canal/outlet to make maintenance easier since this part of the canal may be subjected to flooding during high flood flows.

4.2.1.1. Canal Profile and Alignment

Table 35: Main canal Profile

Chainage(m)	OGL(m)	CBL(m)	FSL(m)	Long. Slope	Depth of cut	Depth of fill
0	2576.716	2575.690	2575.990	0.000	0.726	
20	2576.705	2575.670	2575.970	0.020	0.735	
40	2576.863	2575.650	2575.950	0.040	0.913	
60	2576.621	2575.630	2575.930	0.060	0.691	
80	2576.788	2575.610	2575.910	0.080	0.878	
100	2576.514	2575.590	2575.890	0.100	0.624	
120	2576.701	2575.570	2575.870	0.120	0.831	
140	2576.563	2575.550	2575.850	0.140	0.713	
160	2576.878	2575.530	2575.830	0.160	1.048	
180	2576.653	2575.510	2575.810	0.180	0.843	
200	2576.826	2575.490	2575.790	0.200	1.036	
220	2576.656	2575.470	2575.770	0.220	0.886	
240	2576.505	2575.450	2575.750	0.240	0.755	
260	2576.542	2575.430	2575.730	0.260	0.812	
280	2576.492	2575.410	2575.710	0.280	0.782	
300	2576.561	2575.390	2575.690	0.300	0.871	
320	2576.539	2575.370	2575.670	0.320	0.869	
340	2576.430	2575.350	2575.650	0.340	0.780	
360	2576.350	2575.330	2575.630	0.360	0.720	
	Average depth of cut ,m				***0.82***	

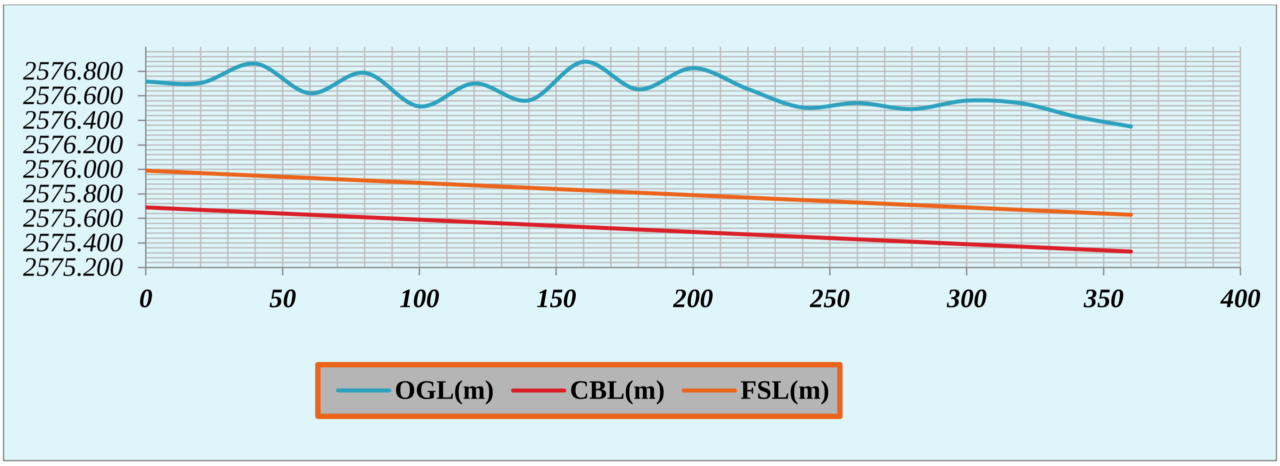


Figure 19: Canal profile

4.3.1.2. *Canal design parameters*

Bed to width ratio, side slope, freeboard and Velocities

Side slope of the main canal and tertiary canal is zero because of rectangular canal consideration. The freeboard for main canal is 0.2m. Bed to width ratio of the canal is 1m with 0.25m/s of minimum flow velocity.

Table 36: Recommended values of b/d ratio, side slopes, freeboards and Velocities by discharge rate value

Status	Discharge m^3/s	Total Freeboard (m)	Lining Freeboard (m)
Small scale projects	< 0.1	0.2	0.1
	0.1 - 0.5	0.3	0.15
	0.5 - 1.0	0.4	0.2

Discharge m^3/s)	b/d	Side slope	Freeboard (m)	Velocity (m/s)
0 -0.15	***1.0***	***1:1***	***0.20***	***0.25***
0.15-0.30	1.0	1:1	0.	0.30
0.30-0.40	1.2	1:1	0.35	0.35
0.40-0.50	1.5	1:1	0.40	0.40

Manning's roughness coefficient (n)

The canal discharge has to be reduced to below the design discharge, in order to avoid overtopping. The following standards of the manning's roughness has been adopted from (Garg, 1976).

Table 37: Recommended values of maximum velocity & roughness coefficient (MoWR)

Lining type	Max. velocity (m/s)	Roughness "n"
Dry stone	1	0.025
Dry brick	1	0.02

Dressed masonry	***2***	***0.018***
Brick	*1.5*	*0.017*
Random Rubble	1.5	0.020
Unreinforced concrete	2.5	0.015
Concrete	2.5	0.017
Buried membrane and Earth	Design as unlined canals	

Longitudinal Slope

The minimum longitudinal slope of the main canal is 1/1000 set to provide reasonable minimum velocity for prevent weed growth and transport the anticipated sediment load. The following standard of maximum hydraulic gradient (bed slope) has been given by the IDD Manual.

Table 38: Recommended Longitudinal Slope

Type of canals	Range of Slope
Main Canals	***1/700 to 1/1500***
Secondary canals	1/700 to 1/1000
Territory canals	1/500 to 1/700
Field ditches	1/300 to 1/500

4.3.1.3. Design Discharge Determination and canal detail

The basic requirements to design the main canal are:

- ✓ Peak water requirement or duty of the area 1.03 l/s/ha
- ✓ The maximum command area, 24ha
- ✓ Manning's roughness coefficient, 0.018
- ✓ Canal gradient or slope for the given soil, 1/1000
- ✓ b/d ratio, 1m.
- ✓ The type of canal section is rectangular section

Flow Depth and Section Capacity

The earthen canals have been designed with a trapezoidal shape and the lined ones with rectangular x-section using Manning's Formula as follows

$$Q = \frac{AxR^{2/3}xS^{1/2}}{n}$$

Where Q = discharge (m^3/s)

R= Hydraulic radius (Flow area/wetted perimeter)

S = Hydraulic gradient

n= Manning's roughness coefficient

Canal capacity is fixed based on the command area that could be irrigated within one irrigation period for the given block of land to be delivering by the canal.

Canal discharge using gross command area = duty irrigated area*

$$Qd = D * A$$

$$= 1.03 * 24$$

$$\boldsymbol{Qd = 0.02472m^3/s}$$

Canal design capacity is designed using 0.02472m3/sec, Q_d, the canal section parameters are fixed by the above value of Q_d, by considering supplementary irrigation.

Area of the canal outlet (A) =bd

Wetted perimeter (P) = b+2d

Where; d = water depth, and

B = bed width

From the recommended value b/d = 1; $A = d^2$; $P=3d$

Using Manning's formula: $Q = \frac{1}{n} * A * R^{\frac{2}{3}} * S^{\frac{1}{2}}$

$$0.02472 = \frac{1}{n} * (d^2) * \left(\frac{d}{3}\right)^{\frac{2}{3}} * \sqrt{0.001}$$

$$0.02472 = \frac{1}{0.018} * \frac{d^{\frac{8}{3}}}{2.08} * \sqrt{0.001}$$

$$\mathrm{d = 0.27m, take\ d =}\ \mathbf{0.3m}$$

$$b = 0.27 \approx Take\ \mathbf{0.3m}\ to\ be\ easy\ for\ construction\ work$$

$$Take\ a\ free\ board\ of\ 0.2m;\ and\ total\ depth\ of\ flow\ including\ free\ board,$$

$$D = 0.3 + 0.2 = \boldsymbol{0.5m}$$

Considering the length of the main canal, the slope of the land, the seepage of water, the scarcity of water and the costs during operation and maintenance, the main canal is proposed to be rectangular masonry lining so as to maximize the efficiency of the irrigation. The canal section is masonry lined and is designed using the manning's formula. $Q = \frac{A}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$

Using this formula for different design discharge, the canal section is determined and the result is shown in the table below. The main canals have been designed for 1:1000; the detail shall be shown in canal profile drawing.

Table 39: Hydraulic parameters of main canal

Parameters	***value***	***unit***
Design discharge ,Q	*0.02472*	*M^3/sec*
Bed width to depth ratio (b/d)	*1.00*	*-*
Side slope ,Z	*0.00*	*m*
Manning's roughness, n	*0.018*	*-*
Longitudinal slope, S	*0.001*	*-*
water depth, d	*0.30*	*m*

bed width, b	0.30	m
Area	0.09	m^2
Wetted perimeter	0.90	m
Hydraulic radius	0.10	m
Maximum velocity, v	2.00	m/sec
Canal Discharge capacity	0.180	m^3/sec
Free board	0.2	m
Total depth with free board	0.50	m
Adopted depth, D	0.50	m
Adopted Bed width, b	0.3	m

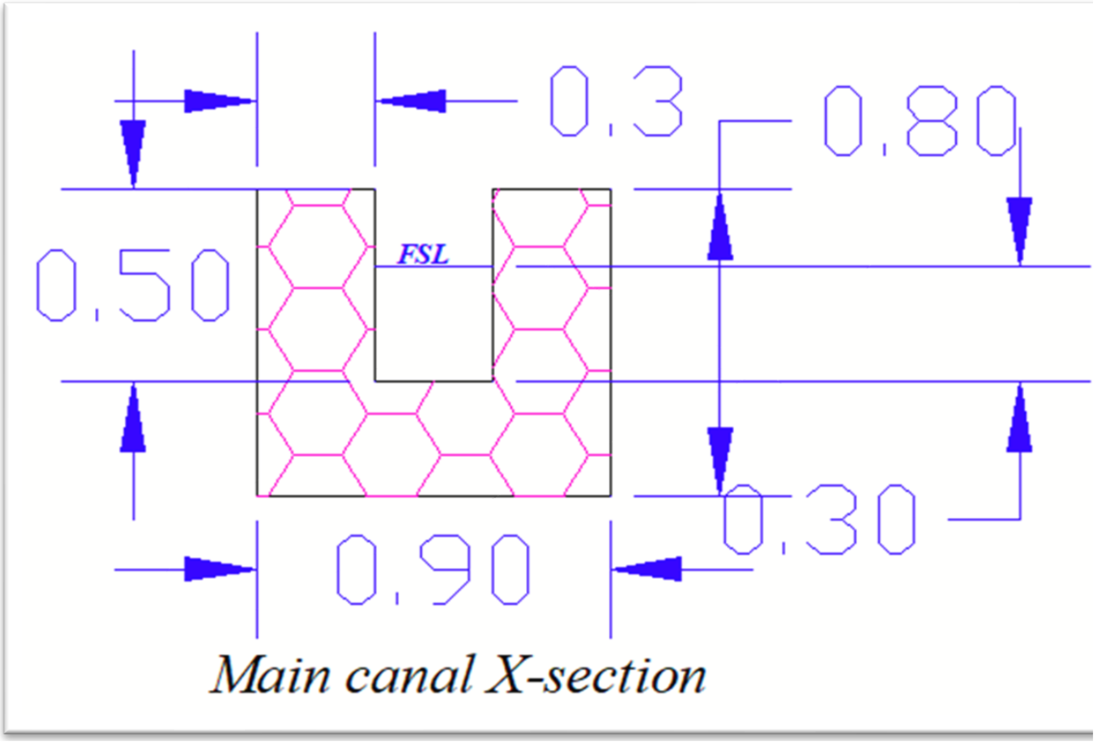


Figure 20: Main Canal X-section

4.4. Canal Appurtenant Structure

4.4.1. Gully Crossing/Flume structures/

When the FSL of the canal is sufficiently above the bottom of the drain trough, we use flume structure. In this canal profile feature there are two flumes at 180m and 340m distances from the head work site. The flume structure has Reinforced concrete (1:2:3 mix) slab covered type above from the bottom level of drain with ϕ12 mm @ 200mm c/c spacing with masonry wall both sides.

Table 40: Flume structure dimensions

flume	Chainage, m	Span Length	Span Width	Height/thickness of concrete	Remarks
1	230	8	1	0.3	Provide 3.4m height masonry walls both sides as a pier
2	340	6	1	0.3	Provide 2.6m height masonry walls both sides as a pier

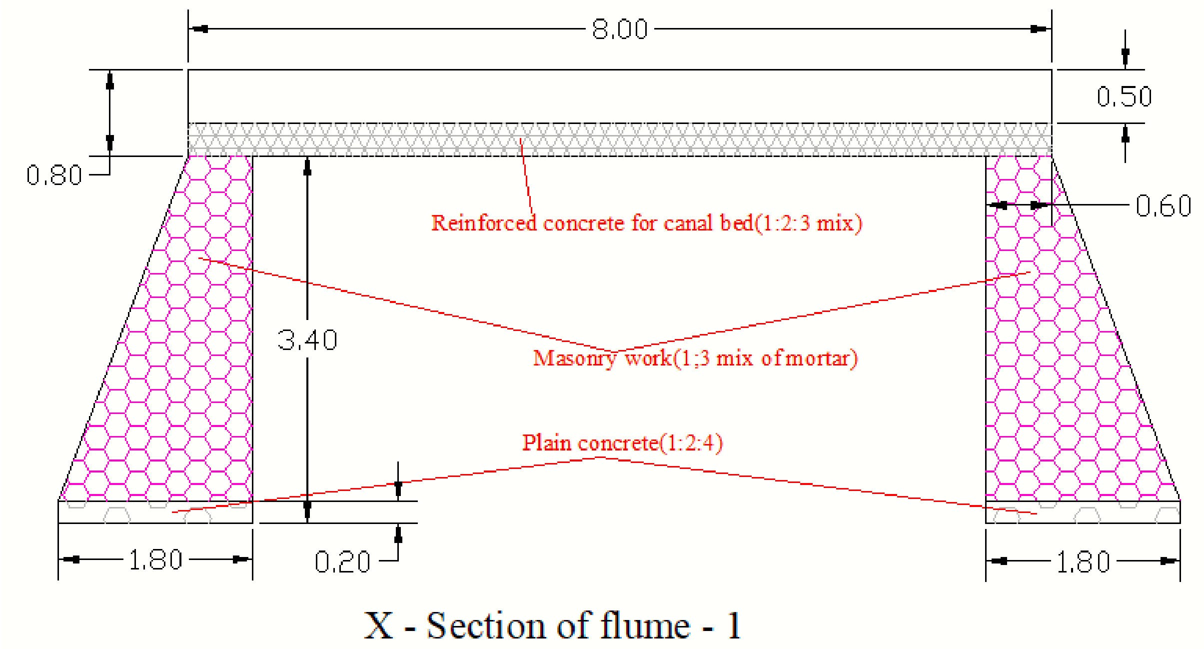


Figure 21: Cross section of flume - 1

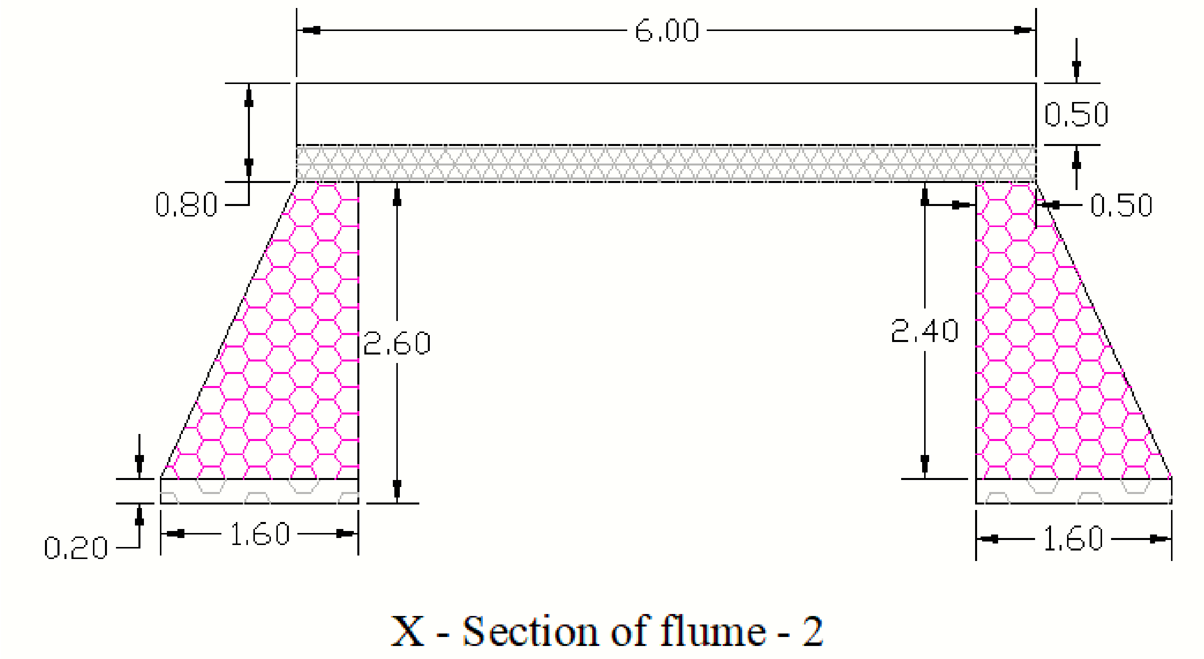


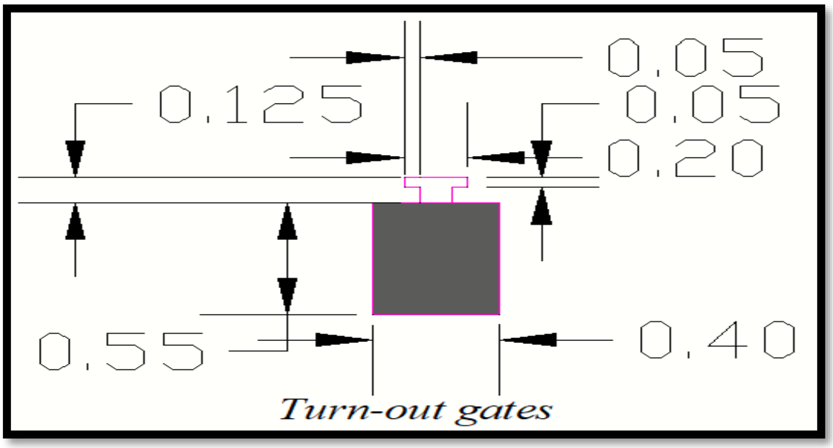
Figure 22: Cross section of flume - 2

4.4.2. Design of canal turnout gates

The width of the turnouts has been decided to take the same as the bed width of the main canal since the computed value for such small flows is minimum (i.e., 0.3m bed width and 0.5m height turnouts for all field canals). On the main canal 6 water flow controlling gates are provided within the dimension of (0.4*0.55) m at 3mm thickness. Angle of iron for grooves is provided for all sheet metals with the total length of 18m. Based on the system alignment and the nature of topography, a turnout can supply water for field canals. The detail is shown in the following table.

Table 41: Turnout gates from masonry lined main canal

Turnouts	1	2	3	4	5	6
Chainage, m	70	120	180	230	270	310



5. BILL OF QUANTITIES

***Zone:-** East Gojjam*						
Wereda**:-** *Enarji Enawuga*						
***Kebele:-** Felege Zachena*						
***Project name:-** Gilgel Cheye Small Scale Irrigation Project*						
A. HEADWORK STRUCTURE						
S. No.	***Item of work***	***Unit***	***Quantity***	***Unit price/Birr***	***Total price/ Birr***	***Remark***
1	**Excavation work**					
1.1	**Oridinary Soil excavation**					
1.1.2	**retaining wall**					
	Excavation of soil on U/S left sides to a depth of 1.05m, 1.25m width and 10m length	m^3	13.13	378.00	4,961.25	
	Excavation of soil on D/S left sides to a depth of 1m, 1m width and 11m length	m^3	11	378.00	4,158.00	
2	**Concrete Work Use C-20 (1:2:3) mix ratio**					
2.1	**Weir body**					
2.2	U/S apron with 17m length, 0.5m width and 0.5m depth	m^3	4.25	15804.50	67,169.13	
2.3	D/S apron with 17m length, 1m width and 0.5m average depth	m^3	8.5	15804.50	134,338.25	
2.4	**Retaining wall**					
	Upsteam retaining wall at the left side to a depth 0.2m with 2.5m width and 10m length	m^3	5	15804.50	79,022.50	
	Downsteam retaining wall at the left side to a depth 0.2m with 2m width and 11m length	m^3	4.4	15804.50	69,539.80	
2.5	Divide wall with 2m length, 1.55m width and 4.3m height	m^3	13.33	15804.50	210,673.99	

2.6	Canal outlet slab with 3.5.m length, 1.5m width and 0.2m thickness	m^3	1.05	15804.50	16,594.73	
2.7	Operation slab for under sluice gate with a length of 3m, 2m width and 0.3m thickness	m^3	1.8	15804.50	28,448.10	
2.8	Columns for Under sluice gate insertation and operation slabs with 4.5m height, 0.4m width and 0.5m length	m^3	1.8	15804.50	28,448.10	
2.9	Breast wall with a height of 2.9, 1.55m length and 0.3m thickness	m^3	1.3485	15804.50	21,312.37	
3	**Masonry work using 65% stone and 35% mortar (with 1:3 mix ratio of mortar)**					
3.1	Upstream retaining wall left sides at 0.3m top width, 2.5m bottom width, 10m total length and 3.5m height	m^3	49.00	9051.00	443,499.00	
3.2	Downstream retaining wall left sides at 0.3m top width, 2m bottom width,11m length and 3.6m height	m^3	45.54	9051.00	412,182.54	
4	**plastering work with 1:3 mix of mortar**					
4.1	**Retaining Wall**					
	Upsteam retaining wall at the left side to a height of 3.5 m and 10m length with 3cm thikness	m^2	35	628.00	21,980.00	
	downsteam retaining wall at the left side to a height of 3.6m and 11m length with 3cm thikness	m^2	39.6	628.00	24,868.80	
	Top of retaining walls in u/s and d/s at the left sides with a total length of 21m, 0.3m width and 3cm thickness	m^2	6.3	628.00	3,956.40	
	the edges of retaining wall in 2 sides with average height of 3.5m, average width of 1.3 and 3cm thickness	m^2	9.1	628.00	5,714.80	
5	**Pointing work with 1:3 mix of mortar**					
	U/S Retaining wall at the left side with a height of 3.5m and 10m length with 3cm thikness	m^2	35	656.00	22,960.00	

	D/S Retaining wall at the left side with a height of 3.6m and 11m length with 3cm thikness	m^2	39.6	656.00	25,977.60	
6	**Back fill use selective materials**					
	upstream retaining wall at the left side to a height of 3.5m and 10m total length with 0.4m width	m^3	14	362.00	5,068.00	
	downstream retaining wall at the left side to a height of 3.6m and 11m length with 0.4m width	m^3	15.84	362.00	5,734.08	
7	**Reinforced bar ø 14 mm**	kg	1914.0	165.00	315,810.00	
8	**Gate**					
	Undersluice spindle gate with 1.6m height and 1.2m width at athickness of 6mm	No.	1	60,000.00	60,000.00	
	the canal intake gate 0.55m height and 0.4m width at a thickness of 4mm	No.	1	15000.00	15,000.00	
9	**Angle iron**					
	angle of iron for groove of canal out let (40*40*4)mm	m	3	400.00	1,200.00	
10	**Form work 4m*20cm*2cm**	No	30	650.00	19,500.00	
11	**Permanent Store with CIS at a dimensions of (6m*5m*3m) and 10cm hard core and 7cm lean concrete for floor with door and window**	No	1			
12	**Dewatering pump**	No	1	30,000.00	30,000.00	
	Total cost without VAT				***2,078,117.44***	
	VAT 15%				***311,717.616***	
	Total cost with VAT				***2,389,835.056***	

B. *FARM STRUCTURE*

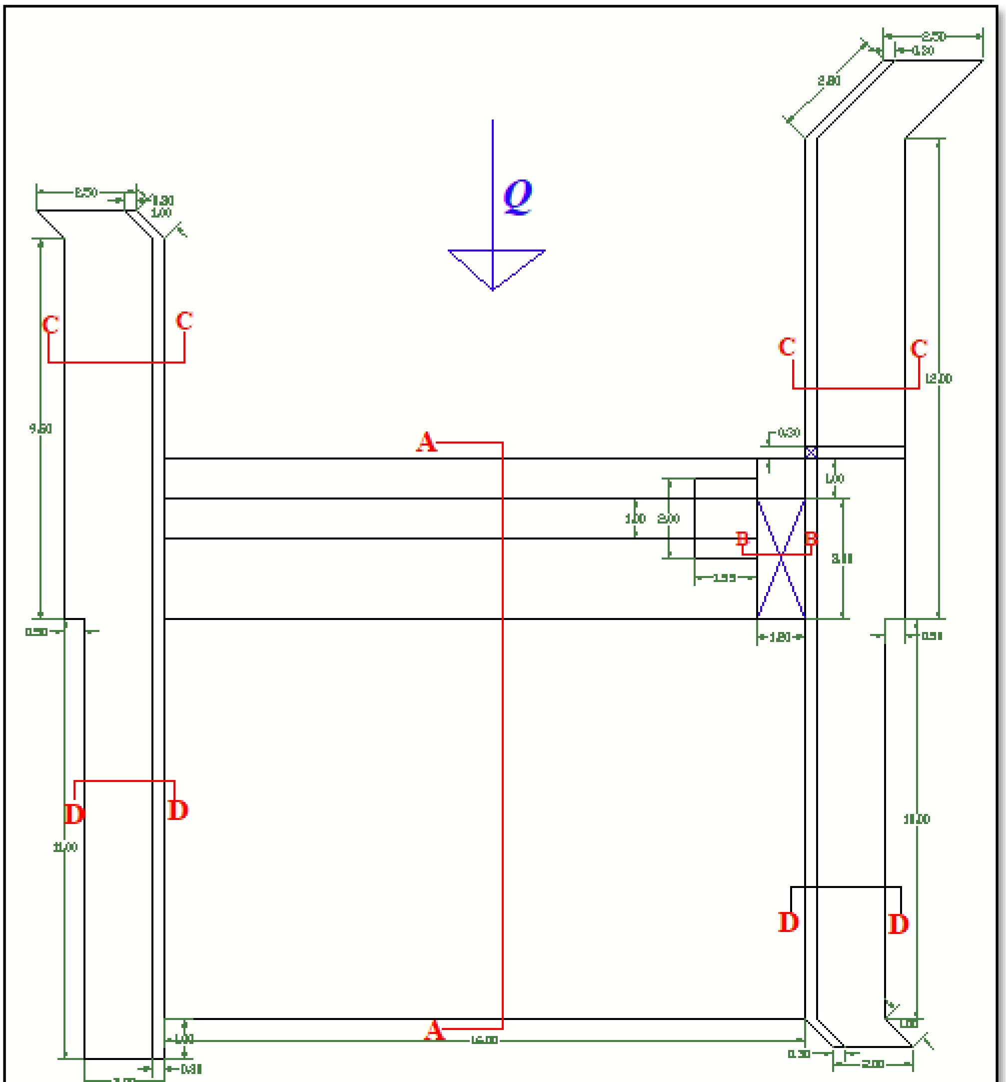
No.	*Item of work*	*Unit*	*Quantity*	*Unit Price/Br./*	*Total Price /Br./*	*Remark*
1	**Excavation Works**					
1.1	**Oridinary Soil Excavation**					
	Main Canal					
	An average (depth of 0.82m 1.3m width and 550m length)	m^3	586.3	378.00	221,621.4	
	Retaining wall for flumes both sides with a total length of 14m, 1.8m width and 3m average depth	m^3	75.6	378.00	28,576.8	

2	**Concrete Work for slabs (1:2:4) mix ratio**					
	Flumes with 14m total length, 0.3m height and 1.4m width	m^3	5.88	15804.50	92,930.46	
3	**Masonry Works (1:3) mix ratio**					
	Main Canal					
	an average (thickness of 0.3m, 0.6m height and 550m length)	m^3	198	9051.00	1,792,098	
	Flume with 8m total length, 1.2m width and 3.4m height	m^3	32.64	9051.00	295,424.64	
	Flume with 6m total length, 1.05m width and 2.6m height	m^3	16.38	9051.00	148,255.38	
4	**Plastering (1:3) mix ratio**					
	Main Canal					
	an average (thickness of 3cm, 1.9m width and 550m length)	m^2	1045	628.00	656,260	
	Retaining wall for flumes					
	an average (thickness of 3cm, 3.4m height and 8m total length)	m^2	27.2	628.00	17,081.6	
	an average(thickness of 3cm, 2.6m height and 6m total length)	m^2	15.6	628.00	9,796.8	
5	**Reinforced Bar(Iron Bar)**					
	12 mm @ 200mm c/c spacing	Kg	189.16	175	33,102.2	
6	**Turn out Sheet Metals**					
	at the dimension of (0.4*0.55) m at 3mm thickness	No.	6	40,000	240,000	
7	**Angle iron**					
	angle of iron for groove of canal turn out on the main canal (40*40*4)mm	m	36	400	14,400	
8	**Back Fill**					
	Main Canal					
	Total (depth of 0.82, 0.4m width and 550m length)	m^3	180.4	362.00	65,304.8	
9	**Formwork**	No.	20	650	13,000	
	Total Cost without VAT				***3,627,852.08***	
	VAT 15%				***544,177.812***	
	Total cost with VAT				***4,172,029.892***	

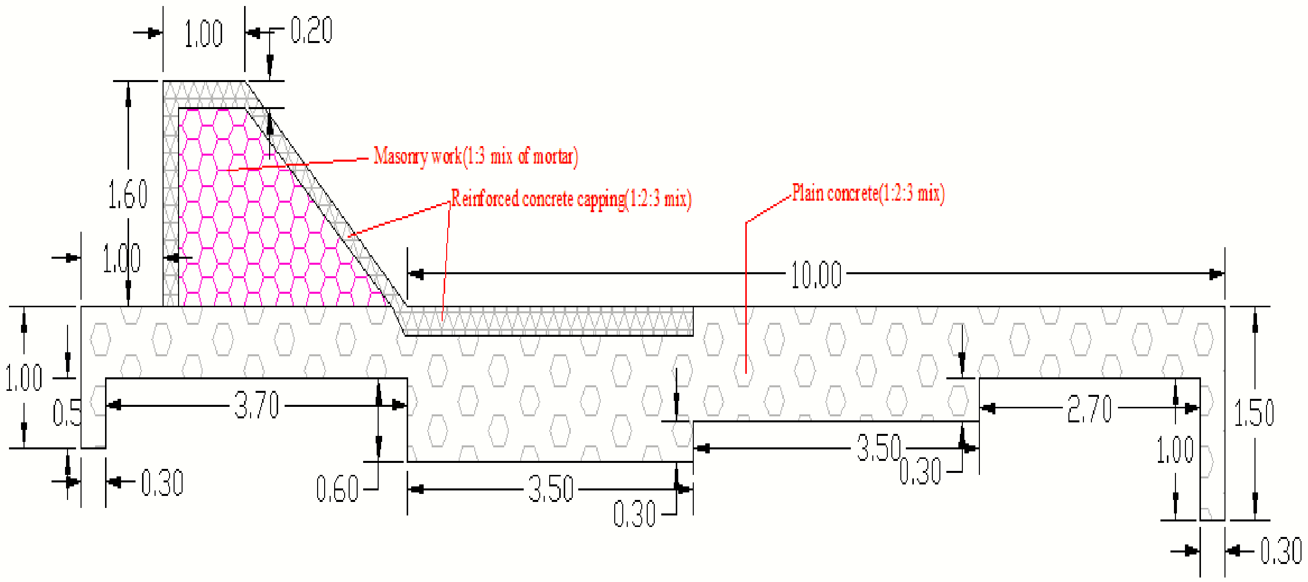
BOQ SUMMARY			
S. No.	Work Item	Project cost/Birr	Remark
1	Headwork Structure (A)	2,078,117.44	
2	Farm work Structure (B)	3,627,852.08	
Project cost (A+B) without VAT		5,705,969.52	

VAT (15%)	855,895.43	
Grand Total Project Cost With VAT	6,561,864.95	

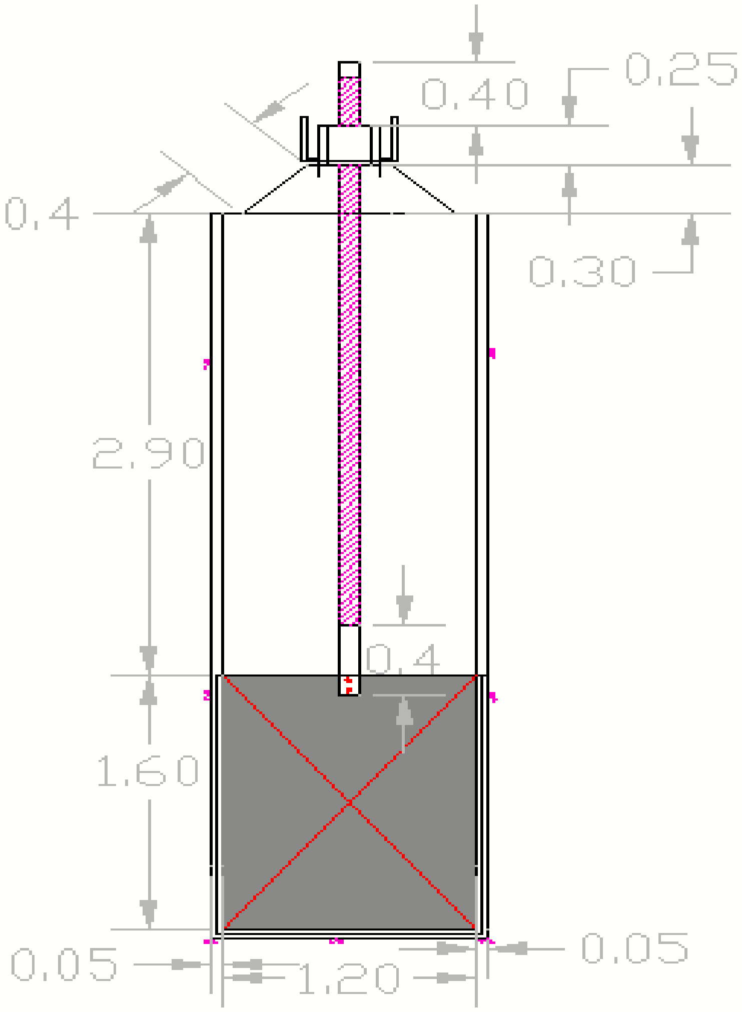
6. Drawings Of All Plans And Sectional Views



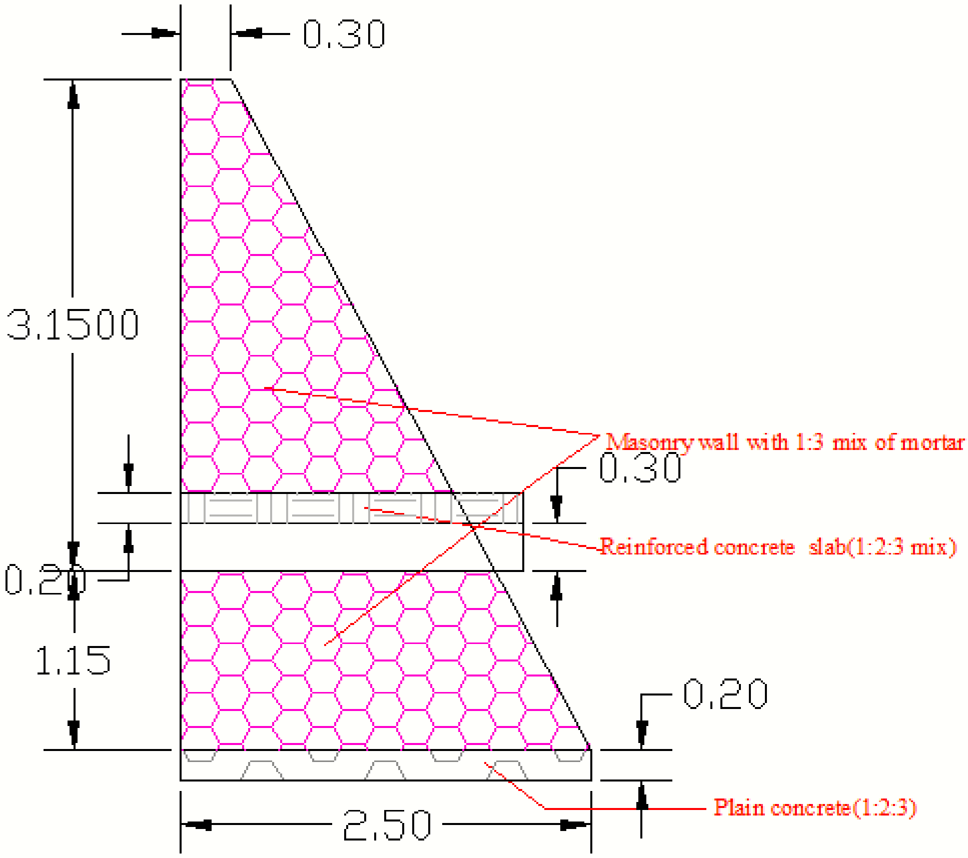
Gilgel Cheye divertion weir plan view



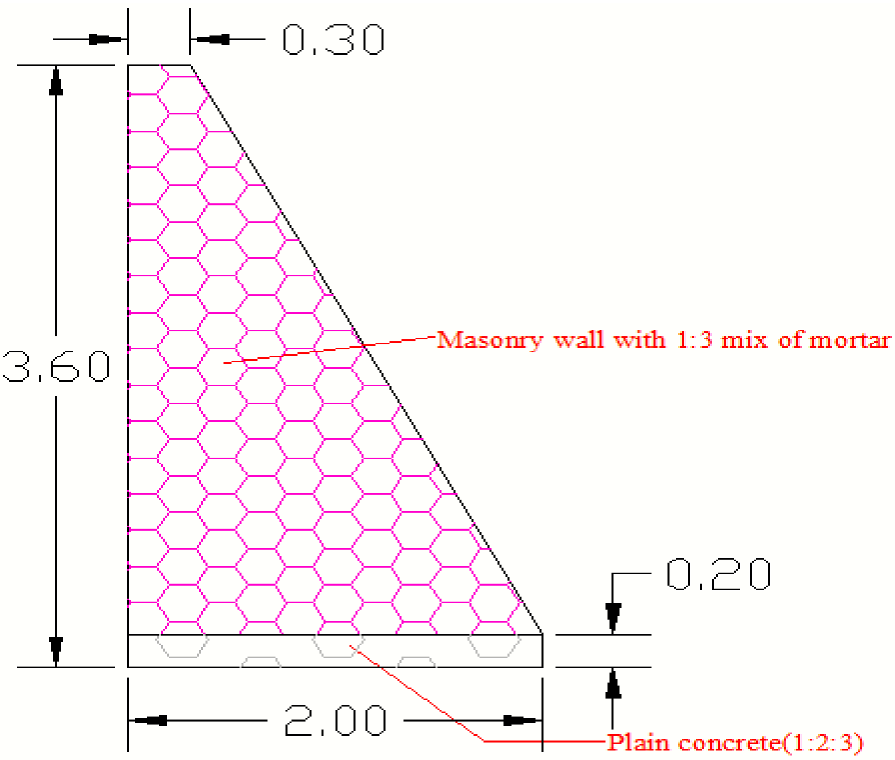
Section A - A



Section B-B



Section C-C



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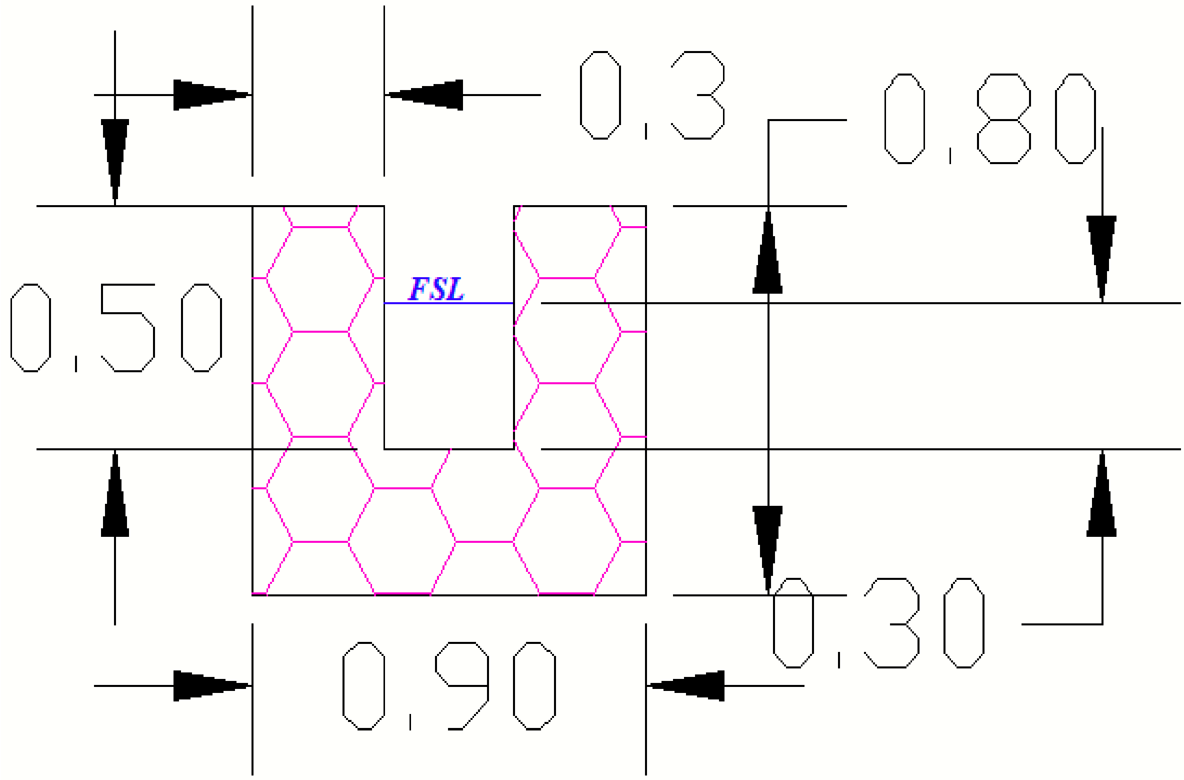
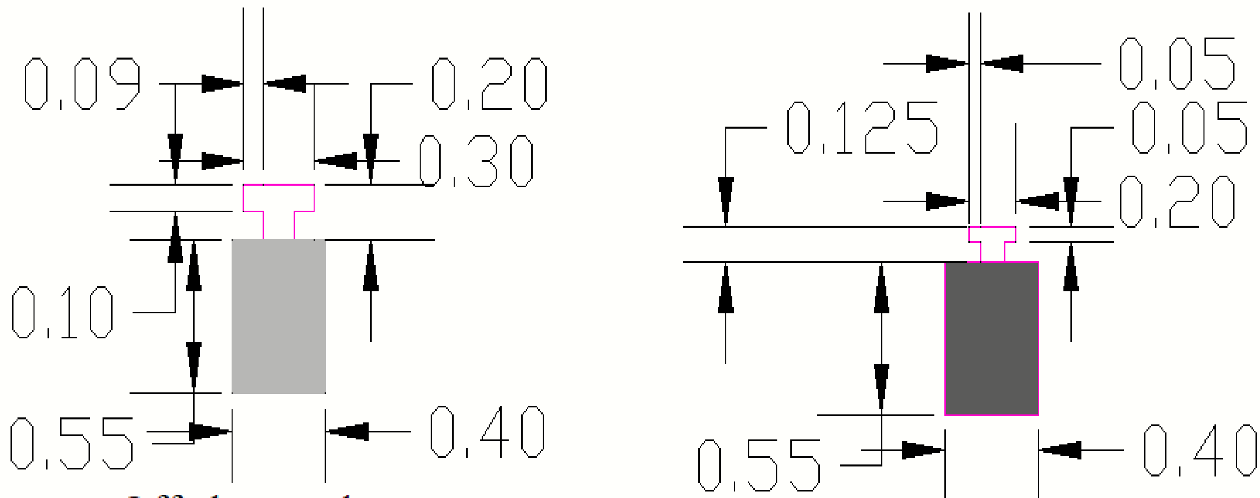
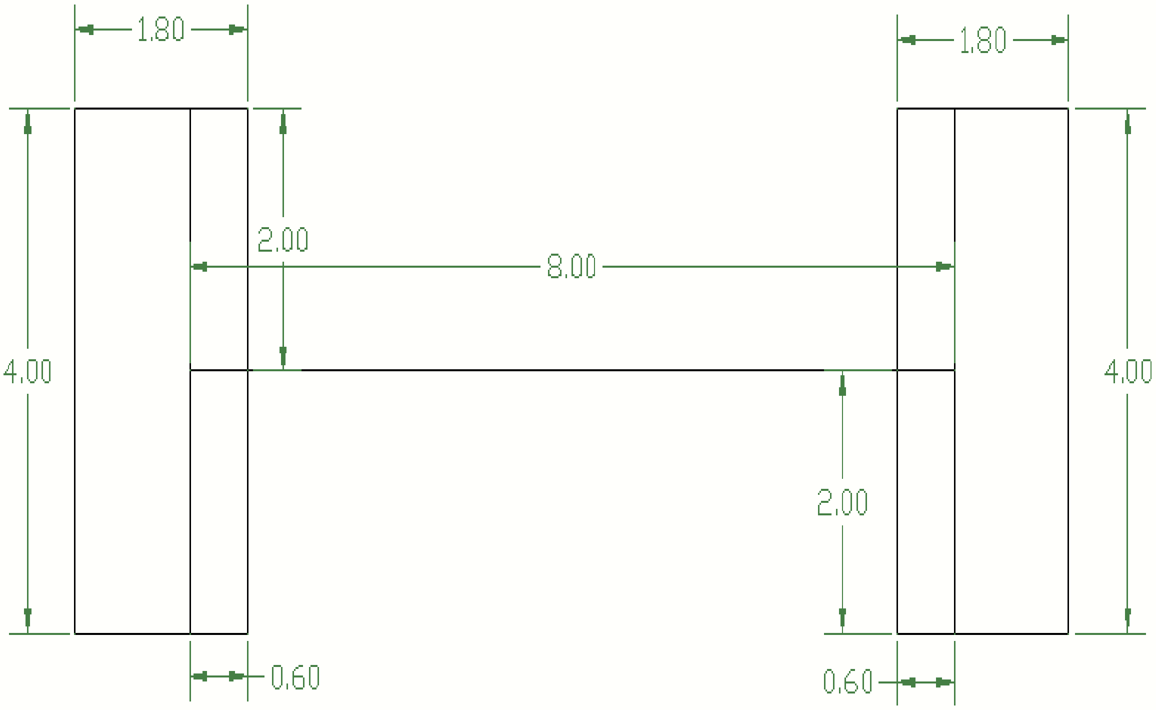


Main canal X-section

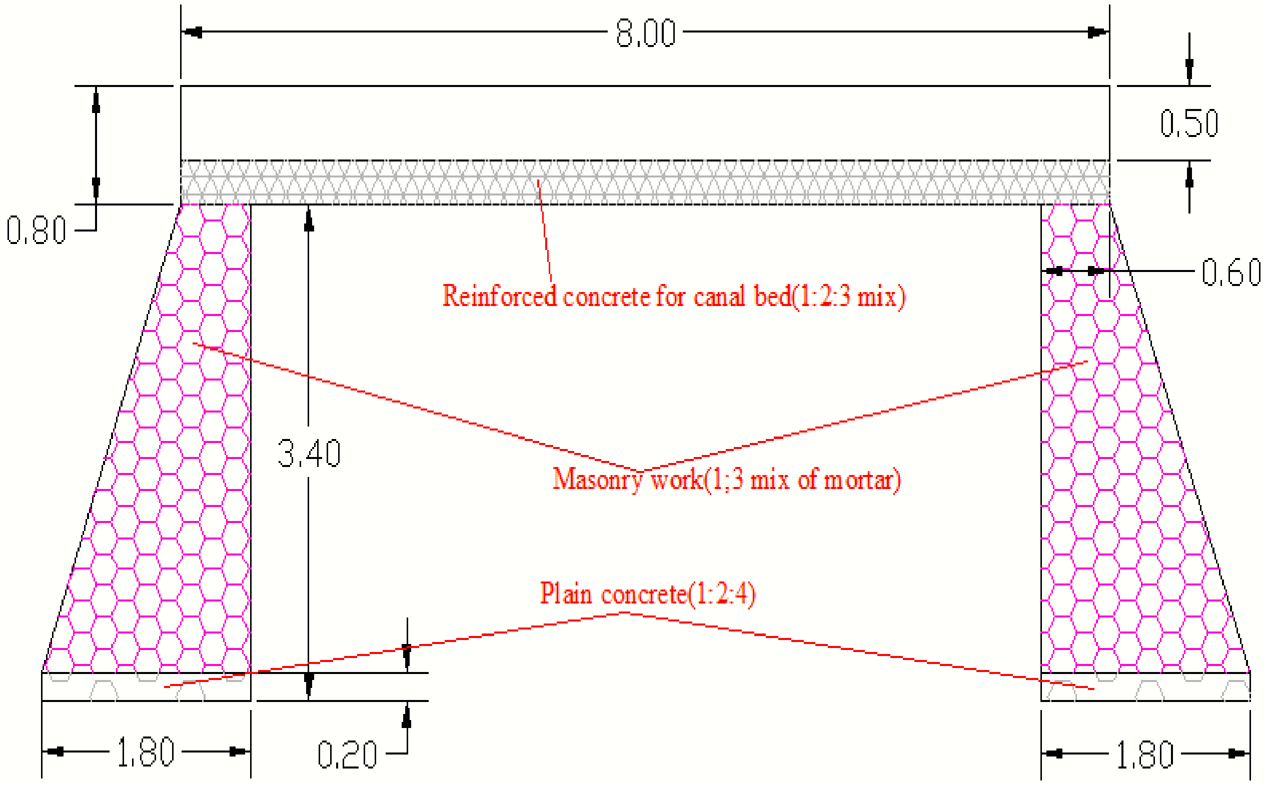


Offtake canal gate

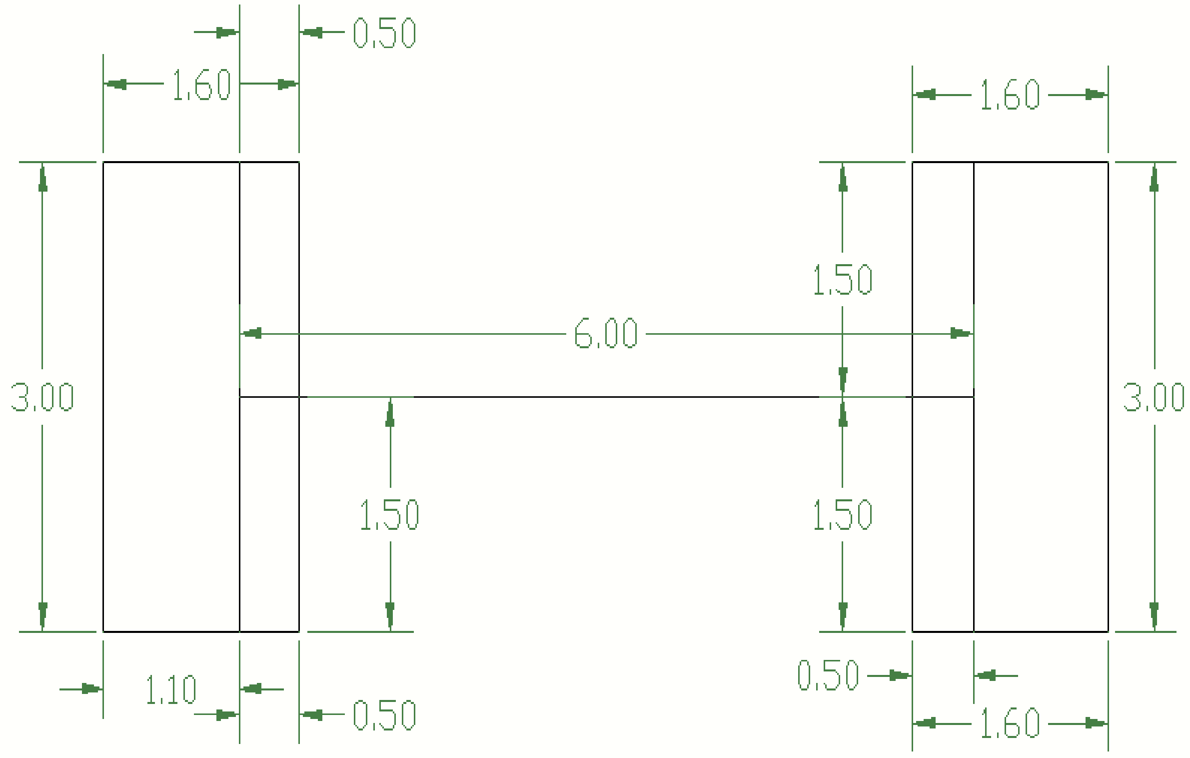
Turn-out gates



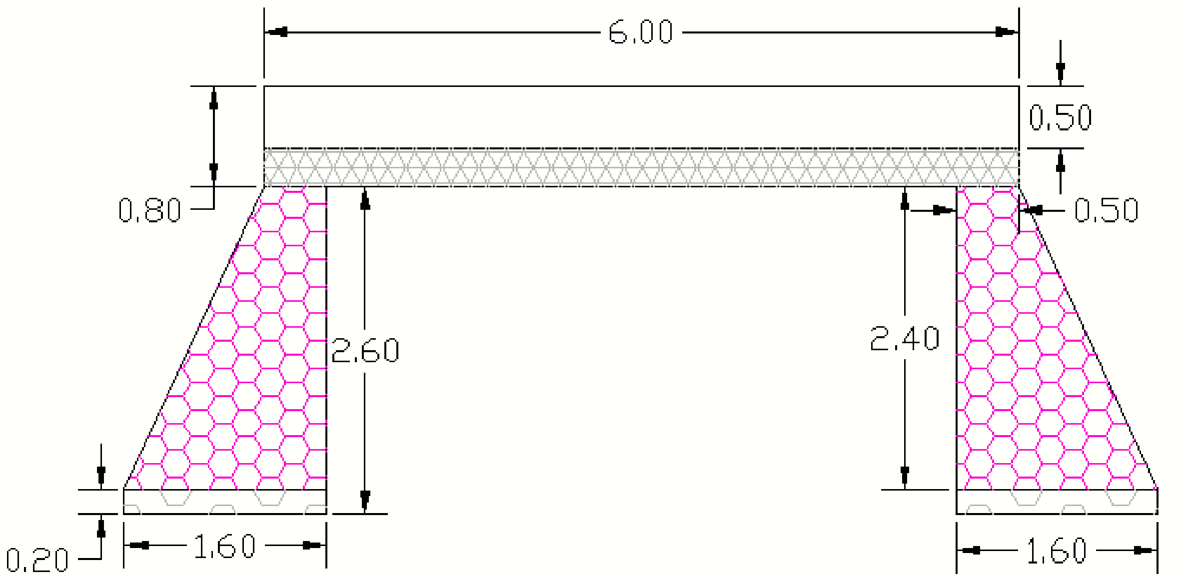
Plan view for flume - 1



X - Section of flume - 1



Plan view for flume - 2



X - Section of flume - 2

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