AMHARA NATIONAL REGIONAL STATE
WATER RESOURCE DEVELOPMENT BUREAU
(BOWRD)

FEASIBILITY STUDY & DETAIL DESIGN OF BURKA SMALL SCALE IRRIGATION PROJECT

ENGINEERING FINAL REPORT

CONSULTANT:
AMHARA DESIGN & SUPERVISION WORKS ENTERPRISE
(ADSWE)

Amhara National Regional State

Water Resources Development Bureau

(BOWRD)

Feasibility Study and Detail Design

Of

Burka Diversion Weir Small-Scale Irrigation Project

Volume IV: Engineering design

Final Report

January, 2014(Revised 2016)

Bahir Dar

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FEASIBILITY STUDY & DETAIL DESIGN REPORT STRUCTURE

- Volume I: Watershed Management
- Volume II: Engineering Geology
- Volume III: Irrigation Agronomy
- **Volume IV: Engineering Design**
- Volume V: Socio Economy
- Volume VI: Environmental Impact Assessment

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SAILENT FEATURES

1. **Project name**: Burka weir diversion Irrigation Project
2. **Name of the stream** : Burka river
3. **Location of the weir site** using Total station
    - Northing: 1263238.91 (UTM)
    - Easting: 574995.5792 (UTM)
    - Zone: (South Wollo)
    - Woreda: (Tehulader Woreda)
    - Average Altitude: 1548.5 m.a.s.l
4. **Hydrology**
    - Design rainfall: 102.3 mm
    - Catchment area: 16.16 km^2
    - Longest flow path length: 12.3 Km
    - Design flood: 76 m^3/Sec
    - Design base flow: 160 lit/se.
5. **Diversion Weir**
    - Type: Broad crest Weir
    - Gross crest length: 26 m
    - weir crest level: 1547.69 m.a.s.l
    - U/S HFL: 1548.77 m.a.s.l
    - U/S TEL 1549.138 m.a.s.l
    - D/s TEL: 1547.80 m.a.s.l.
    - D/s HFL:1547.74 m.a.s.l
    - Afflux:1.03 m
6. **Under sluice**
    - Sill level: 1546.8m.a.s.l
    - Dimension: 1.0*0.7 m^2
    - Discharge amount: 0.7m^3/Sec
7. **Outlet**
    - Sill level:1547.19.a.s.l
    - Opening dimension: 0.5m*0.5 m
    - Discharge capacity: 122.4L/Sec.

Irrigation and drainage systems Infrastructure

- Command area size: 80 ha
- Type of soil of the command area is dominantly silt clay soil
- Base flow which is measured in February is 160 l/Sec.
- Design discharge of the main canal = 122.4 l/Sec
- Release to down stream 38 l/sec. (24%)

Project cost without VAT

Bill No.	Descrption	Amount (Birr)	Remark
1	General item	1,400,307.10	
2	Head Work	3,471,828.53	
3	Main, Secondary, Tertiary Canal and Catch Drain	3,723,901.03	
Total		**8,596,036.66**	
VAT 15%		**-**	
Grand Total		**8,596,036.66**	
	Hectar	80	
	Cost /Ha	107,450.46	

1. INTRODUCTION

1.1. Background

In Ethiopia, 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 in 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 small-scale irrigation, water harvesting, and on-farm diversification) coupled with strengthened linkages between agriculture and industry (agro-industry), thereby creating a demand for agricultural output. In line with the above, efforts have been made by the government and NGO's to improve the situation in the country in areas of domestic water supply provision, irrigation, watershed management, etc. Amhara Water Resources Development Bureau is playing its role in the development of small scale irrigation projects in the region. Accordingly, as part of the water sector development program, the office has initiated the study and design of a small scale irrigation scheme on Burka the river at 17 Kebele and signed an agreement with Amhara Design & Supervision Works Enterprise (ADSWE) for the study and design of the project.

1.2. Description of the Project Area

1.2.1 Location

This irrigation project is located mainly at 17 Kebele, Tehuledar Wereda of the south Wollo Zone in the Amhara Region. The proposed irrigation project is to be undertaken on Burka River and the headwork structures are specifically located at an altitude of about 1548.5 masl and geographical coordinates of 1263238.91m N and 574995.58m E.

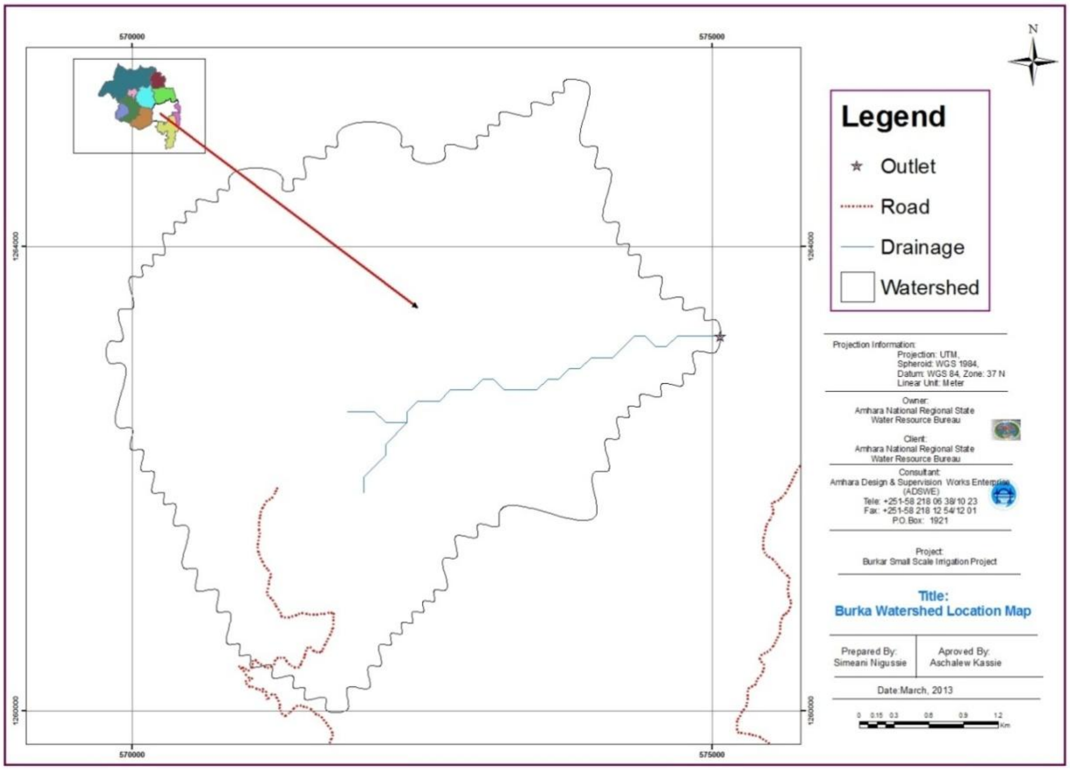

Figure 1-1: Location map of the project area

1.2.2 Accessibility

The Burke irrigation project is founded in South Wollo zone, Tehulederie Wereda, betweenTebisa Kabul (016) & Seglen kebele (017) in the Tach Burka village administratively. The project site is 19km far from Haik, which is the capital city of Tehulederie Wereda, in the north east direction. 17km length of the road is all weathered one and 2km is the distance to be covered on foot. Only 17km is accessible by car at any time. The rest 2km is inaccessible by car at wet season.

1.2.3 Previous Irrigation Practices

There are traditional diversions on the downstream of this river (about 10km down) using different irrigation practices but as the hydrology and Hydrogeology study and respondent farmers indicated, the river has a capacity of recharging as it stretches down from the source area of the river. As a result there will not be a marked reduction or fluctuation of water flows both for the already existing and the newly proposed irrigation schemes. There is no written document which justifies the traditional irrigation practices undertaken that use the river flow to the extreme right side to irrigate minimum command area with hardship. So, the farmers in the project area are very much interested in upgrading the traditional scheme to modern scheme.

1.3. Objectives of the Study

1.3.1 Major Objective

The project area faces variability of rainfall distribution though the overall rainfall generally suffices the rain-fed agriculture. Accordingly, the rain-fed agriculture needs a means of supplementing during distribution failures and further full irrigation is required to maximize the use of the potential land and water resources.

Hence the objective of this project is to contribute a substantial share in the effort to reduce the risk of production decrease due to rainfall variability and increase the productivity of the resource in the project specific area. Specifically, the project is targeted for the following.

- To make sustainable the rain-fed crop production and make extra production in the dry season possible for 80 ha of land through irrigation.
- There is a general consensus that irrigation investments will achieve broader food security and poverty reduction impacts and if efforts are also geared towards up-grading existing traditional farming practices with support to enhance access to input supply, output marketing and extension to facilitate access to information and innovations.
- This objective is to be realized by constructing diversion structures across the Burka River and diverting the river flow.

1.3.2 Specific Objectives

Other benefits that can be expected to appear with the launching of the project area:

- Efficiency of water use improvement;
- Improved local nutrition/food security gains;
- Improved management of scarce natural resources (land and water);
- Resilience against drought risk;
- Rationale for erosion control and watershed management;
- Rationale for the intensification and modernization of smallholder agriculture and rural lifestyles.

The engineering study and design enables the realization of the project by the provision of engineering structures that will allow the appropriate abstraction of the river water for delivery in the identified irrigation fields of the study area. Hence, this engineering design is specifically targeted:

- Analyze hydrologic requirements of the project and engineering structures;
- The formulation of sound and stable structure, with necessary provisions that allow safe, easy and low-maintenance operation in the service life of the project;
- Develop working drawings;
- Estimation of construction costs.

1.4. Scope of the Study

- The irrigation design shall ensure reliability, equity and flexibility of water delivery to farmers. It will aim at reducing conflicts among water users and will lead to lower operation and maintenance costs.
- Updating the existing, if available, computation of the actual evapo-transpiration, crop water requirement, irrigation demand/duty using the existing and recent agronomic, climatologic and soil data using more appropriate methodologies.
- Establish design criteria for irrigations 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,
- Establish flood protection measures for the command area and canal structures and design the respective drainage system accordingly,
- Planning and layout of the irrigation system, which include irrigation canals, drainage channels, inspection roads and alignments, canal spacing, canal length, location of structures, and water profiles along canal and drains at specified reaches, which is most economical easily manageable and aligned with topographic feature and geological investigation.
- Determination and estimation of water application conveyance and other losses and irrigation efficiencies and consideration of those parameters in design steps.
- Check and test 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, Designers used the following steps.

- Specific Site identification:

  - Review of the reconnaissance survey conducted by the Client
  - 50,000 scale top map and GIS information
  - Local farmers interview and discussion
  - Woreda and Zone Agriculture section expertise
  - Previous studies
  - On foot travel along the river channel and farm areas.
- 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 during the reconnaissance field visit by floating method.
- Irrigable area identification:
    - Using local information
    - 50,000 Topographic map, and GIS information, GPS to see elevation

The design report is organized in three sections. In **Section I** the Hydrology study is presented and in **Sections II** and **III** the Headwork and Irrigation and Drainage Systems designs are discussed respectively. In Section III, planning and design of the irrigation system after diverting the water using the weir will be dealt. The following are major areas of concern in this part.

- Study and design of the irrigation method to be adopted,
- Study and design of the irrigation system layout and associated structures,
- Design of the different conveyance canals,
- Planning and design of the different irrigation and drainage structures,
- Preparation of the longitudinal profiles of the different irrigation and drainage canals.

SETION I: HYDROLOGY

2. HYDROLOGY

2.1 Watershed Characteristics

The Watershed has marked topographic variation. All types of slopes are present. The dominant slope class is moderately steep (15-30%) which covers 30.63% of the total area followed Steep (30-50%) which is 21.84%. Sloping (8,-15%), gently sloping (3-8%) and Very steeply (›50%), Flat or almost flat 0-3%) accounts 21.48%, 19.10%, 6.08% and 0.87% respectively. Table 1 shows the slope classes and the proportion of the watershed.

Table 2-1: Slope classes of the watershed

Designation	Slope	Area (ha)	Percent (%)
Flat or almost flat	0-3%	14.08	0.87
Gently sloping	3-8%	308.81	19.10
Sloping	8-15%	347.23	21.48
Moderately steep	15-30%	495.14	30.63
Steep	30-50%	353.16	21.84
Very steeply	>50%	98.31	6.08
Total		1616.72	100.00

Certain physical properties of watersheds significantly affect the characteristics of the runoff and sediment yield and are of great interest in hydrology analyses. The rate and volume of runoff, and sediment yield from the watershed have much to do with shape, size, slope and other parameters of the landscape. These suggest that there should be some important relations between basin form and hydrologic performance. If the basin and hydrologic characteristics are to be related, the basin form must also be represented by quantitative descriptors. These parameters can be measured from maps, figure 2 below talks more about the morphology of Burka.

The watershed characteristics are summarized as:

- Catchment Area = 16.167 km^2
- Stream Length = 6.15Km
- CN(III) = 85.06

(Extracted from the Watershed Study Report of burka)

Burka River at the headwork site is characterized by a well-defined channel system and considerably low flows. It looks that the gradient of the river is getting low and hence there exists significant deposition of sediment mainly cobbles and boulders.

Table 2-2: Catchments Morphology

S/N	Parameters	Symbol	Unit	Formula	Result
1	Area	A	Km2	Measured	16.167
2	Perimeter	Pb	Km	Measured	22.99
3	Axial Length	Lb	Km	1.312*A^0.568	6.3
4	Basin Width	W	Km	A/Lb	2.5
5	Total No. of Streams	N	No	Measured	3
6	Total Stream Length	L	Km	Measured	4.5
7	Main Stream Length	Lm	Km	Measured	6.15
8	Stream Density	Sd	No/Km2	N/A	0.185
9	Main Stream Slope	S	%	Measured	8.4
10	Stream Order	Os	No	Measured	2^{nd}
11	Drainage Density	D	Km/Km2	L/A	0.278
12	Over Land Flow Length	Lo	M	1/2d	0.179
13	Shape Factor	B	Unit Less	Lb^2/A	2.5
14	Form Factor	Rf	Unit Less	A/Lb^2	0.397
15	Elongation Ratio	E	Unit Less	Dc/Lb	0.71
16	Perimeter of Circle Having Same A	Pc	Km	3.545a^0.5	14.25
17	Circularity Ratio	Rc	Unit Less	$R_c=4\pi A/P_c^2$	0.999
18	Texture Ratio	T	Unit Less	N/Pb	0.13
19	Bifurcation Ratio	Rb	Unit Less	Nw/Nw+	3/1
20	Compactness Co-Eff.	Cc	Unit Less	Pb/Pc	1.6
21	Diameter Of Circle Having Same Area	Dc	Km	1.128a^0.5	4.5

2.2 Hydro-Metrological Data Availability

2.2.1 Climate

Small scale irrigation project designers and planners are faced with lack of good data on the hydrology of the stream/river system that will be their water source and on local weather and climate conditions. Stream gauging stations are virtually non-existent in remote rural areas of Ethiopia; meteorological stations are almost rare. Likewise, at segilen Kebele (Project area location) and in the catchment area of this project, there is no meteorological station of any level. Moreover, there are no flow data for the river near the project. Therefore, data for the hydro-meteorological analysis is taken from the nearby station and similar areas. Rainfall & temperature data are considered from the Bokeksa Meteorological station. The spatial location of the Station, Bokeksa is at Latitude of 11.37^{o} and Longitude of 39.88^{o} with Altitude of 1800 m.a.s.l.

As per the data from the station, March – June are identified as high temperature periods whereas November–January are low temperature periods. The mean annual rainfall amount in Bokeksa is about 1082mm (2000-2008 data) and most of it occurs from July to September. The monthly rainfall distribution (Figure 3) has a uni-modal or mono-modal characteristic with better rainfall distribution from July to September.

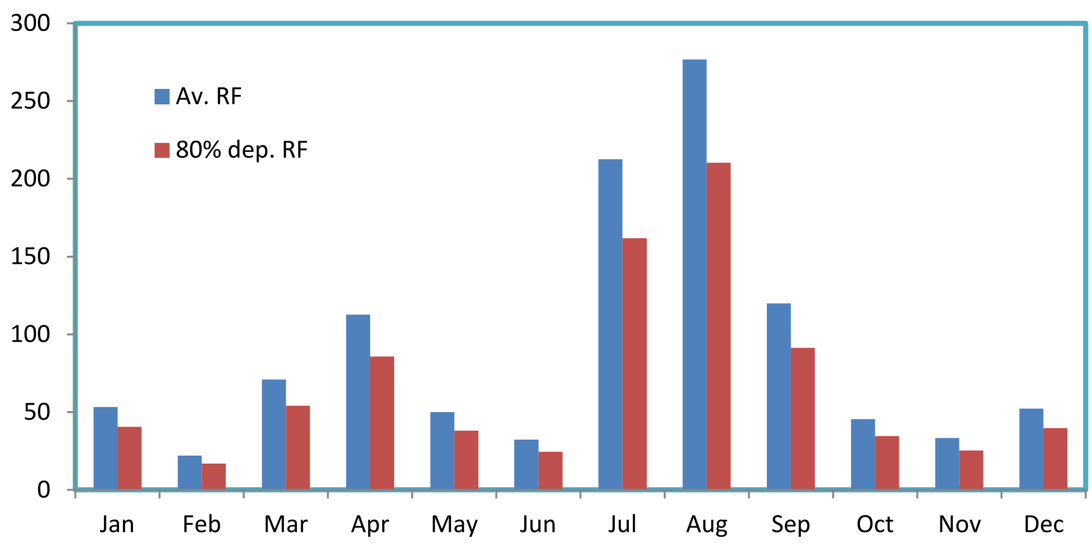

Figure 2-1: Average and 80% dependable rainfall at Bokeksa Station

2.2.2 Rainfall Data

In order to compute the design flood for the intake structure, the daily maximum rainfall is collected from Bokeksa Meteorological stations with a record of 39 years.

2.2.3 River Flow Data

The base flow which is measured during the month of February(driest season) 2013 is 160 l/s. The study team has measured the base flow during the feasibility study period on the weir axis. The available water in the this season the team recommended the irrigation schedule in each month based on availability of water and the demand for irrigation which is dealt in the infrastructure part of this study. Due to this the base flow has 160 l/s, then 25% release to downstream. There for we have left 122.4 l/s. Due to this base flow there is around 80ha irrigation command area will cover in dry season because there are no upstream users to reduce the base flow so we have only downstream release for the case of ecological balance and locals and cattle provisions.

2.3 Upstream & Downstream Utilization

Downstream of the proposed site, appreciable need for water is anticipated for locals and cattle provisions. Therefore, at least 25% around 38 l/s of the minimum flow has to be released for downstream requirements.

For the sake of planning and design, however, the outlet for the diversion is designed for a discharge of 122.4 l/s for this project and the project is to be developed for 80 ha of land for dry season, which is most of the time achievable as the flow for most of the time is significant to support this size of command area.

2.4 Design Flood Analysis

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 Method.

2.4.1 Design Rainfall Computation

Based on the data of 24hr peak rainfall given in below table shows the design rainfall is computed using only by Gumble’s Extreme Value Method because it is recommended for best and safe for structural safety avoiding of structural engineering of under estimation and the client interest.

3.1.1.1 Outlier Test

Higher Limit, $Y_H = Ymean + Kn * Sy$, Kn = 2.036 for 10 Years of data.

Lower Limit, $Y_H = Ymean - Kn * Sy$, Kn = 2.036 for 10 Years of data.

Table 2-3: Outlier test analysis

Year	Max.Rf (X)	Descending order(X)	Rank	Log value(Y)
1970	47.0	104.2	1	2.02
1971	42.3	91.7	2	1.96
1973	40.8	81.5	3	1.91
1974	42.5	76.0	4	1.88
1975	65.2	72.0	5	1.86
1976	39.7	71.9	6	1.86
1977	44.3	71.6	7	1.85
1979	46.3	70.3	8	1.85
1980	40.6	67.4	9	1.83
1981	80.6	66.5	10	1.82
1982	44.8	65.2	11	1.81
1983	80.7	65.0	12	1.81
1984	71.9	64.0	13	1.81
1985	72.0	62.7	14	1.80
1986	44.2	62.6	15	1.80
1987	81.5	61.6	16	1.79
1988	62.6	80.7	17	1.78
1989	42.8	80.6	18	1.78
1990	67.4	59.4	19	1.77
1991	56.5	58.8	20	1.77
1993	62.7	56.5	22	1.75
1994	61.6	52.6	23	1.72
1995	52.6	48.4	24	1.68
1996	48.4	47.0	25	1.67
1997	47.0	47.0	26	1.67
1998	71.6	46.3	27	1.67
1999	43.2	44.8	28	1.65
2000	56.7	44.3	29	1.65
2001	104.2	44.2	30	1.65
2002	58.8	43.9	31	1.64
2003	43.9	43.2	32	1.64
2004	70.3	42.8	33	1.63
2005	59.4	42.5	34	1.63

Year	Max.Rf (X)	Descending order(X)	Rank	Log value(Y)
2006	64.0	42.3	35	1.63
2007	65.0	42.1	36	1.62
2008	66.5	40.8	37	1.61
2010	91.7	40.6	38	1.61
2011	42.1	39.7	39	1.80
sum		2,259.40		68.24
mean		57.9		1.75
standard deviation		15.06165108		0.11
skewness coefficient		0.969504424		0.40

Higher Limit, YH = 2.06

Lower Limit, YL = 1.43

Therefore,

Upper limit of rainfall = 10^2.06 = 116.48mm

Lower Limit of rainfall = 10^1.43= 27.10mm

Conclusion: The rainfall values are within the limits.

3.1.1.2 Check for variance

After checking the outliers, the data should be checked for variability. For variability the formula used is $\alpha = \left(\frac{\delta_{\cap-1}}{\sqrt{\mathrm{N}}*\mathrm{Mean}}\right) * 100\%$

Where, $\delta_{n-1} =$ Standard deviation =0.1074

N = Nr of recorded data =39

M_{ean} = 1.75

α = Standard error

$\alpha = \left(\frac{0.1074}{\sqrt{39}*1.75}\right) * 100\% = 1 < 10\%$ Acceptable

Therefore the data shows no variation.

The candidate distributions give almost identical correlation coefficients. However, the standard errors are significantly lower for the Gumbel EVI Method which is 102.36mm. Accordingly, the design rain for this distribution has been selected as the best fit for this study as per our employer

The design rainfall using 102.36 Method is given as

$$R_f = R_{mean}. + \sigma_{n-1} * K$$

Where R_f = Design rainfall

R_{mean} = average of all values of annual heaviest fall = 57.93 mm

σ_{n-1} = standard deviation of the series = 15.061 mm

$$K = \frac{Yt - Yn}{Sn}$$

$$Y_t = -\ln\ln(\frac{T}{T-1}),$$ T= Return period = 50 years

$$Y_t = -\ln\ln(\frac{50}{50-1}) = 3.9$$

Yn, Sn = constant found from Gumble's extreme value distribution table for N= 39 Years

- Yn = 0.5423 and Sn = 1.1388

$$K == (\frac{3.9 - 0.543}{1.1388}) = 2.9478$$

$$R_f = 57.93 + 15.06 * 2.9478 = 102.36 mm$$

Point Design Rainfall = 102.36 mm

The design rainfall at points for 50 years return period is 102.36 mm and the areal design rainfall is calculated in the following section.

2.4.2 Peak Discharge Determination

The River is not gauged river. The design flood is calculated by using SCS unit hydrograph method. Thus, it is preferred to base the flood analysis on rainfall data, which are better both in quantity and quality of data. In the hydrologic analysis for drainage structures, it must be recognized that there are many variable factors that affect floods. Some of the factors that need to be recognized and considered on an individual site by site basis are; rainfall amount and storm distribution; catchment area, shape and orientation; ground cover; type of soil; slopes of terrain and stream(S); antecedent moisture condition; Storage potential (over bank, ponds, wetlands, reservoirs, channel, etc.)

2.4.3 Peak flood analysis by SCS unit hydrograph method

Design flood is calculated SCS (The United States Soil Conservation Service). This method is widely adopted and more reliable method for flood estimation. The approach considers, watershed parameters, like Area, Curve number, and time of concentration.

2.4.4 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 reaches. Kirpich formula is adopted for computation.

Table 2-4: Time Calculation of concentration

Partial Distance/km/	Cumulative distance/km/	Elevation/m/	Elevation diff./Meter	TC/hr
0	0	2077	0	
0.14	0.14	1980	117	0.02
0.11	0.25	1900	80	0.02
0.14	0.39	1880	40	0.02
0.22	0.61	1820	40	0.04
0.6	1.21	1740	80	0.10
0.49	1.7	1700	40	0.10
0.38	2.08	1680	20	0.10
0.34	2.42	1680	20	0.09
0.75	3.16	1640	20	0.22
0.47	3.63	1620	20	0.13
0.76	4.39	1800	20	0.23
0.8	5.19	1580	20	0.24
0.96	6.15	1556	24	0.28
Time of concentration in hour				**1.59**

The formula is,

$$Tc = \sum 0.948\left\{\left(\frac{{L_1}^3}{H_1}\right)^{0.385} + \left(\frac{{L_2}^3}{H_2}\right)^{0.385} + \cdots + \left(\frac{{L_n}^3}{H_n}\right)^{0.385}\right\}$$

- Tc = 1.59Since Tc < 3hr., duration of excess rainfall difference, D = 0.5 hr or Tc/6 =~ 0.3
- Time to peak,

  $T_{p=}\frac{D}{2} + 0.6 * T_c$ = 1.15 hr

- Base time,

  $T_b = 2.67 * T_p$ = 3.08hr
- Recession time,

  $T_r = 1.67 * T_p$ = 2.655 hr.

2.4.5 Curve number (CN)

Curve number (CN) is achieved based on USSCS method by watershed characterization in terms of land cover, treatment, hydrologic condition and soil group. From the watershed analysis curve number at condition II =70.53. Since peak rainfall is found in an antecedent moisture condition III state, this value has to be changed to antecedent moisture condition III.

⇒ Conversion factor = 1.206

⇒ CN Condition (III) = (Factor from Table x CN condition II) =70.53*1.206 = 85.06. Area Rainfall

2.4.6 Area Rainfall

As the area of the catchment gets larger, coincidence of all hydrological incidences becomes less and less. This can be optimized by changing the calculated point rainfall to aerial rainfall. The conversion factor is taken from standard table that relate directly with the size of watershed area and type of the gauging station. (IDD manual)

For the case of Burka irrigation project,

⇒ Total watershed area = 16.167 Km^2

⇒ Type of gauging station = Daily rainfall (24 hr.)

⇒ Aerial Rainfall = (Point Rainfall) x (Conversion factor)

2.4.7 Run off Analysis

Input data:

Design Point Rainfall = 102.36mm

Curve number at antecedent moisture condition III = 85.06

Catchment Area, A = 16.167Km^2

Tc = 1.65hr, D = 0.5hr., Tp = 3.08 hr; Tb = 2.655 hr; Tr = 0.5 hr.

Direct run-off, $Q = \frac{(I-0.2*S)^2}{(I+0.8*S)}$,Where, I = Rearranged cumulative run-off depth (mm

S = Maximum runs off potential difference, $S = \left(\frac{25400}{CN}\right) - 254$

Peak run-off for incremental; $Q_p = 0.21 * \frac{(A*Q)}{T_p}$, Where, A = Catchment area = 16.167 Km^2

T_p = Time to peak (hr)

Q = Incremental run-off (mm)

Table 2-5: Runoff analysis

Duration	**Hr**	**0-0.5**	**0.5-1**	**1-1.5**	**1.5-2**	**2-2.5**	**2.5-3**
Design Point rainfall	mm	102.36					
Rainfall profile in (%)	%	32.50	43.75	53.13	57.89	62.50	66.24
Rf profile	mm	33.27	44.78	54.38	59.25	63.97	67.80
Areal to point rainfall ratio	%	79.31	85.77	88.36	90.30	91.59	92.24
Areal rf	mm	26.38	38.41	48.05	53.50	58.80	62.54
Incremental Rf	mm	26.38	12.03	9.64	5.46	5.09	3.95
Descending order	mm	26.38	12.03	9.64	5.46	5.09	3.95
Rearranged order	No	6.00	4.00	3.00	1.00	2.00	5.00
Rearranged incremental Rf	mm	3.95	5.46	9.64	26.38	12.03	5.09
Commutative Rf	mm	3.95	9.40	19.04	45.42	57.45	62.54
Time of begging	Hr	0.00	0.28	0.56	0.84	1.11	1.39
Time to peak	Hr	1.14	1.42	1.70	1.98	2.25	2.53
Time to end	Hr	3.05	3.33	3.80	3.88	4.16	4.44
Commutative RUN OFF	mm	0.00	0.01	1.88	16.45	25.31	29.30
Incremental RUNOFF	mm	0.00	0.01	1.87	14.57	8.87	3.99
Peak discharge	m^3/s	0.00	0.02	5.57	43.35	26.38	11.86
Time of begning	Hr	0.00	0.28	0.56	0.84	1.11	1.39
Time to peak	Hr	1.14	1.42	1.70	1.98	2.25	2.53
Time to end	Hr	3.05	3.33	3.80	3.88	4.16	4.44

Table 2-6: Hydrograph Coordinates

Unit	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s
Time	**U1**	**U2**	**U3**	**U4**	**U5**	**U6**	**Base flow**	**Total**
0.00	0						0.08	0.09
0.28	0	0					0.08	0.09
0.56	0	0.00	0.00				0.08	0.09
0.84	0	0.01	1.36	0.00			0.08	1.46
1.11	0	0.01	2.72	10.57	0.00		0.08	13.39
1.14	0	0.01	2.85	11.63	0.64	0.00	0.08	15.23
1.39	0	0.02	4.08	21.15	6.43	2.13	0.08	33.90
1.42	0	0.02	4.21	22.20	7.08	2.37	0.08	35.97
1.70	0	0.01	5.57	32.78	13.51	4.74	0.08	56.70
1.98	0	0.01	4.76	43.35	19.94	7.11	0.08	75.27
2.25	0	0.01	3.94	37.02	26.38	9.49	0.08	76.92
2.53	0	0.01	3.13	30.69	22.52	11.86	0.08	68.30
3.0	0	0.00	1.63	18.99	15.41	8.66	0.08	44.78
3.3		0.00	0.81	12.66	11.56	6.93	0.08	32.05
3.6			0.00	6.33	7.70	5.20	0.08	19.32
3.9				0.00	3.85	3.46	0.08	7.41
4.2					0.00	1.73	0.08	1.82
4.4						0		0

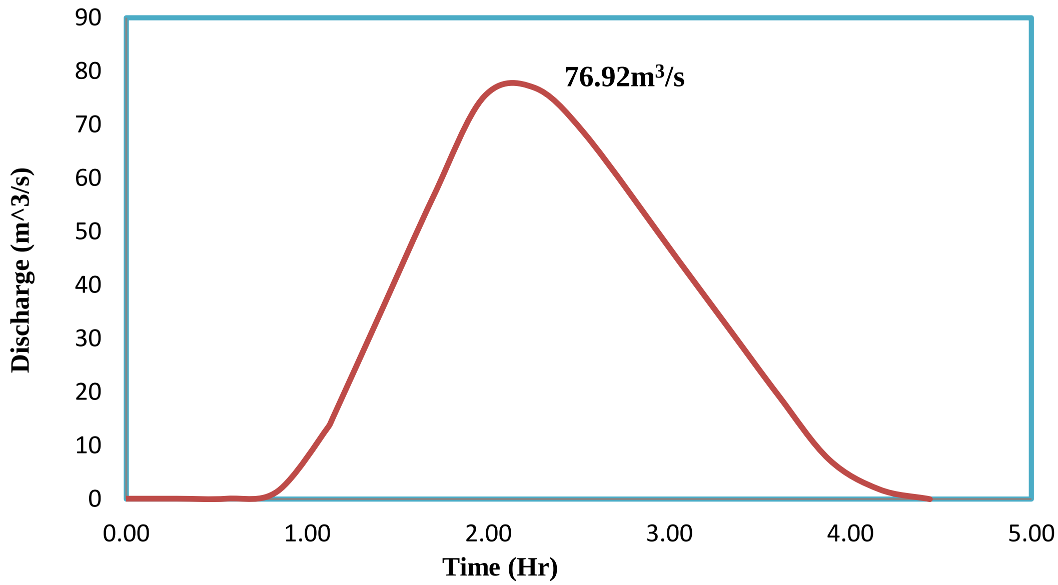

Figure 2-2: Complex Hydrograph

2.5 Tail Water Depth Computation

Tail water depth of the river is the depth of flow at the proposed weir site before construction of the weir. It is used to crosscheck peak flood estimated by the SCS unit hydrograph method with flood mark method and to see the flood feature after the hydraulic jump. During field visit, the flood mark of the river at the proposed diversion site was marked based on dwellers information and physical indicative marks. Also detail river cross-section data has been collected to be used for the computation of tail water depth.

2.5.1 Average River Bed Slope

Average river bed slope of River is estimated by two different techniques. One is by end area method and the other is by using best fit line method. Designers have adopted the end area method output for further analysis.

The water level of the river is taken at different points along the river channel around the head work site. Surveying work done for 445m length. And then, average water surface slope is considered as the river bed slope. For comparison of the two procedures, refer the attached Excel file.

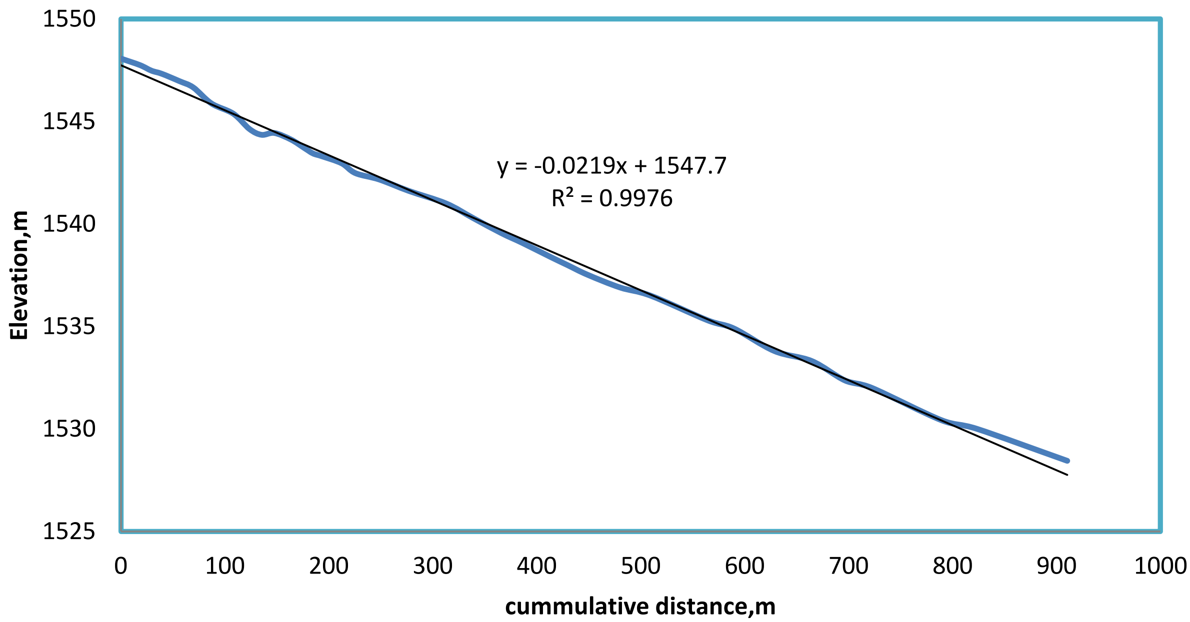

Figure 2-3: River bed profile

2.5.2 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 curving nature. The river banks are defined and relatively smooth. Manning's roughness coefficient (n = 0.0225) is adopted.

2.5.3 Discharge of the river

Computations of the river discharge at different stages of the flow have been made using the river cross section and longitudinal slope of the river.

Input data:

- Manning's roughness coefficient, n =0.0225
- Average river bed slope, S = 0.01
- $V = \frac{1}{n} \times R^{2/3} \times \sqrt{S}$, Where, R = Hydraulic radius = (Area/Perimeter)
- $Q = V * A$

Table 2-7: River discharge computation at different stages of flow

Stage	**Area**	**Wetted Perimeter**	**R**	**V**	**Q**
1546.50	0.00	0.00	0.00	0.00	0.00
1547.00	1.63	7.22	0.23	0.33	0.55
1547.50	13.35	45.34	0.29	0.40	5.34
1548.00	36.57	50.35	0.73	0.73	26.70
1548.50	62.10	55.36	1.12	0.98	80.57
1548.77	76.91	58.42	1.31	1.08	84.29
1549.00	90.04	61.13	1.47	1.17	105.31
1549.50	120.83	67.12	1.80	1.34	161.56
1550.00	157.95	73.62	2.15	1.50	237.39
1550.50	191.20	80.55	2.37	1.61	307.40
1551.00	231.00	87.52	2.64	1.73	398.61

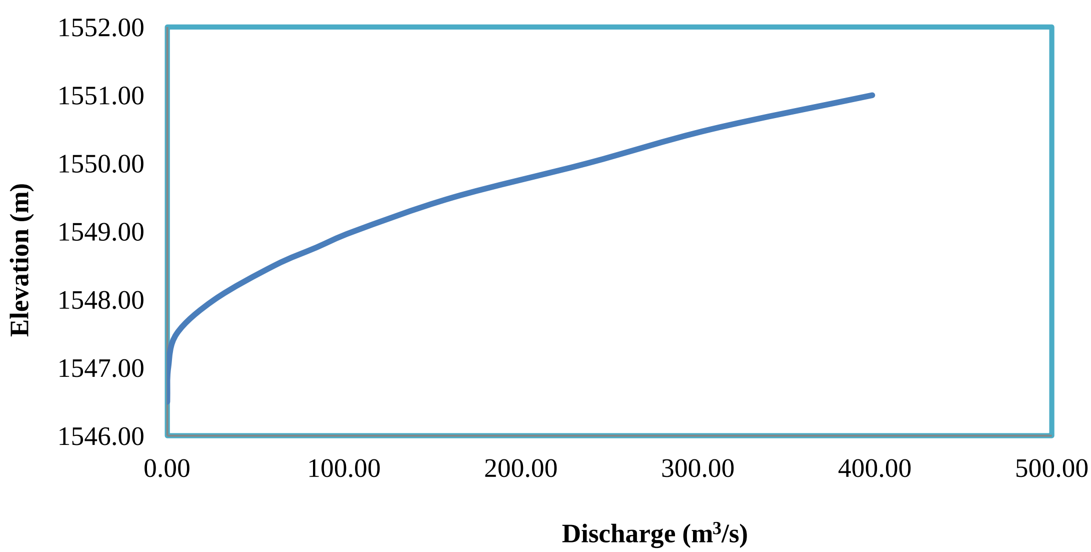

Figure 2-4: Discharge -elevation rating Curve

From the above stage discharge table and curve the maximum flood level corresponding to the computed design peak discharge is 1548.77m.a.s.l (1.31m from the river bed) and it is considered as the d/s high flood level i.e. expected at the weir axis before construction of the weir. Then from this point we are fix the tail water depth, afflux and post jump see below two tables.

Table 2-8 Tail water depth, afflux and post jump both tables

Stage	Area	P	R (m)	V (m/s)	Q (m^3/s)	Z (m)	q ($m^3/s/m$)	He (m)	Hd	He = $q^2/((.8+Hd)^2*(2*9.81))$	Ha = He-Hd	H = $v^2/(2*g)$ (m)	D/s HFL = BL+TWD (m)	D/S TEL = D/S HFL + ha (m)
[1]	**[2]**	**[3]**	**[4]**	**[5]**	**[6]**	**[7]**	**[8]**	**[9]**	**[10]**	**[11]**	**[12]**	**[13]**	**[14]**	**[15]**
1546.50	0.00	0.00	0.00	0.00	0.00	1.20	0.00	0.00	0.000	0.000	0.00	0.00	1546.49	1546.49
1547.00	1.63	7.22	0.23	0.33	0.55	1.20	0.02	0.05	0.027	0.000	0.03	0.01	1546.74	1546.74
1547.50	13.35	45.34	0.29	0.40	5.34	1.20	0.21	0.24	0.240	0.004	0.00	0.01	1546.99	1547.00
1548.00	36.57	50.35	0.73	0.73	26.70	1.20	1.03	0.71	0.676	0.039	0.04	0.03	1547.24	1547.27
1548.50	62.10	55.36	1.12	0.98	80.57	1.20	2.33	1.23	1.130	0.104	0.10	0.05	1547.49	1547.54
1548.68	76.91	57.59	1.26	1.06	76.91	1.20	2.96	1.45	1.08	0.21	0.37	0.06	1547.74	1547.80
1549.00	90.04	61.13	1.47	1.17	105.31	1.20	4.05	1.78	1.000	0.372	0.78	0.07	1547.74	1547.81
1549.50	120.83	67.12	1.80	1.34	161.56	1.20	6.21	2.37	1.200	2.373	1.17	0.09	1547.99	1548.08
1550.00	157.95	73.62	2.15	1.50	237.39	1.20	9.13	3.07	1.220	1.436	1.85	0.12	1548.24	1548.35
1550.50	191.20	80.55	2.37	1.61	307.40	1.20	11.82	3.64	1.212	2.432	2.43	0.13	1548.49	1548.62
1551.00	231.00	87.52	2.64	1.73	398.61	1.20	15.33	4.33	1.590	2.743	2.74	0.15	1548.74	1548.89

U/s HFL = 1547.69+He-ha (m)	U/s TEL = U/sHfL+ha (m)	He+d	Y1 depth	Y2 depth	Y2 Level	Z+He	Y1 +($q^2/2gy_1^2$)	Y3 (TWD)	TWD level	Afflux	Lj
[15]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]	[24]	[25]	[26]
1547.691	1547.691	1.20	0.00	0.00	1546.50	1.20	0.00	0.00	1546.50	1.20	0.00
1547.718	1547.744	1.26	0.01	0.09	1546.59	1.25	0.27	0.25	1546.75	0.98	0.42
1547.931	1547.935	1.45	0.04	0.44	1546.94	1.44	1.34	0.50	1547.00	0.94	2.00
1548.367	1548.406	1.92	0.18	1.00	1546.70	1.91	1.83	0.75	1547.25	1.13	4.12
1548.821	1548.925	2.44	0.37	1.56	1547.26	2.43	2.40	1.00	1547.50	1.33	5.93
1548.770	1549.138	2.65	0.45	1.77	1547.47	2.65	2.63	1.25	1547.75	1.03	8.18
1548.691	1549.475	2.99	0.57	2.15	1547.85	2.98	3.15	1.25	1547.75	0.95	7.93
1548.891	1550.064	3.57	0.77	2.83	1548.53	3.57	4.06	1.50	1548.00	0.90	10.26
1548.911	1550.758	4.27	1.02	3.80	1549.30	4.27	5.10	1.75	1548.25	0.67	12.91
1548.903	1551.335	4.85	1.45	3.77	1549.47	4.84	4.85	2.00	1548.50	0.41	11.63
1549.281	1552.024	5.53	1.79	4.36	1550.06	5.53	5.53	2.25	1548.75	0.54	12.83

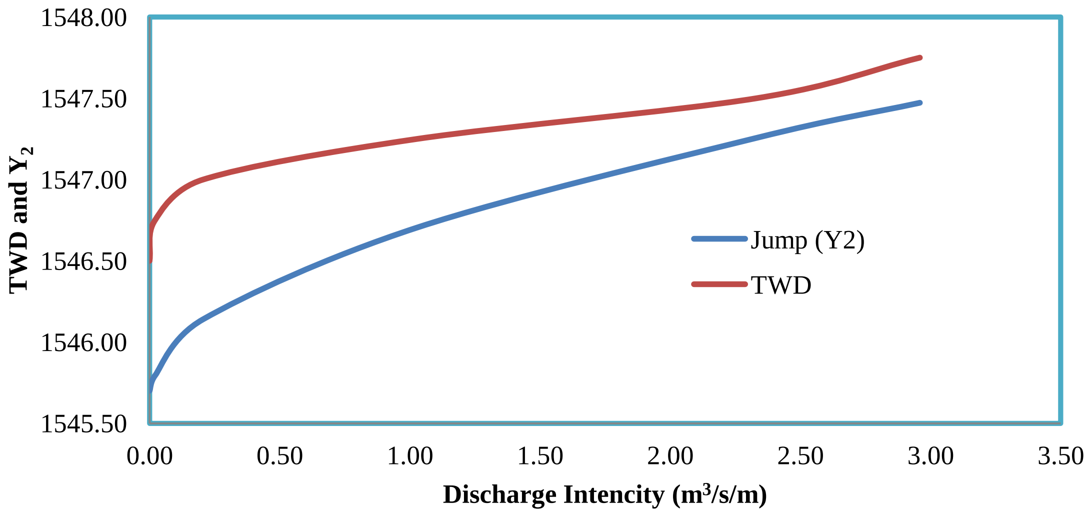

Figure 2-5: Comparison of Jump depth versus Tail water depth

SECTION II: HEADWORK

3. HEADWORK DESIGN

3.1 Headwork Site Selection

The headwork site is situated at 1263238.91 m N, 574995.5792m E and river bed elevation of 1548.5 m above sea level. At this site the river course is well defined, Natured with fixed width and forms nearly a U-shaped valley. At this specific site, covered by recent alluvial deposit sediment at the river bed whereas the left bank is also made from silt clay soil but the right bank is covered by silt clay soil at the top for few centimeter thickness below this soil there is highly to moderately weathered soil which extend in upstream direction. The river is flowing through a defined channel and its scouring effect on the river course is significant as its bed covered by loose, alluvial deposit which is easily erodible and transported by running water. The different sections of the stream in the proposed headwork site are described separately below.

3.2 River Geomorphology

It is a common fact that the river development tends to accommodate itself to the local geology that develops along the structurally weak zones like faults, joints, folds, etc. The drainage system of the study area is strongly influenced by geological structures and formations, the nature of the vegetation cover and climate. The nature of geological formations and structures has also a strong influence on the development of the channel.

The present morphology of the Burka River channel is a function of a number of processes and environmental conditions, including the composition of the bed and the banks (moderately weathered, fractured boulders along right side whereas loose soils and recent coarser deposit at the left bank and bed); the size and composition of the sediment moving through the channel; the rate of sediment transport through the channel and deposition on the banks and beds; and the regional degradation due to erosion processes. Both left bank and the bed is composed of loose soil and coarser alluvial sediment as the result the stream shows highly meandering nature both up and downstream from the proposed site but at the particular weir site it shows nearly straight river channel.

3.2.1 River Bed Condition

As it was observed during the detail study, the river bed is made up of the recently deposited alluvial sediments like silt, sand and gravel with the ratio of 25%, 25%, and 50% respectively. As it has been checked by excavating the test pits at the edge of the bank, the thickness of these deposits is expected more than 5meters in which the height of the bank of the bed is 1.5meters. The spring of the Burka River is located at about 100meters away from the weir axis in the upstream direction. Consequently, it is expected that if the spring is filled by the siltation, the spring may get out at the downstream of the weir axis and we may lose the flow water. Therefore, the only option is increasing the score depth of the bed bar of weir up to 6meter from the bed level. The hard rock is not expected at shallow depth. The major deposition category groups represent a unique type of soils and suggest the heterogeneity of the deposits. This deposit soil is therefore finally grouped under GM soil, which allows high seepage of the water through it. Therefore, it should be removed out.

Figure 3-1: River bed at the proposed weir site

3.2.2 River Bank Condition

3.2.2.1 Right Bank

The right bank of the river Burqa is characterized with two major soil horizons. One test pit was dug at the margin of the riverbank, which has 3meters depth. According to the log data from the test pit, the upper part is silt and sand soils, which is deposited recently or it is not the residual soil. The thickness of this soil is about 2meter. The lower part of this bank is characterized by the expansive black cotton clay. This soil is the residual soil. The thickness of this soil on this bank is more than 1meter. The height of this bank is about

1.5meters from the bed level. The retaining wall needs on this direction of the bank for about 20meters on the downstream and 10meters on the upstream.

3.2.2.2 Left Bank

The left bank of this river is made up of two major different geologic materials. The top part is transported silt, clay and sand soils. The thickness of these coarse grained materials is about 2m. And the lower part is very fine-grained expansive black cotton clay. The thickness of this soil is more than 2m as it has been observed from the test pit log.
The retaining wall is necessary on this side of the bank on the downstream about 20metre and on the upstream it needs about 10metre.

3.2.3 Sources of construction materials

During site investigation, natural construction materials required for the construction of the various proposed engineering structures at the headwork and within the farmland have been assessed, and possible quarry sites and borrow areas have been identified within the vicinity of the study area as close to the project site as possible. In addition to identifying the quality, quantity and accessibility conditions of the construction materials, ownerships of each proposed production sites have also been studied and described in this report, on separate sub-sections below. The materials needed for the construction of the structures include rock for masonry stones, aggregates (both coarse and fine), and water.

3.2.3.1 The Borrow Material

This material is used for the general fill of the foundation or sub grade of the layers of the native soils with the crushed aggregates or other native soils having different densities. It is formed from the deeply weathered alkaline rock. It is native soil which has no swelling and shrinking nature when compacted. In other cases it stays as it is when compacted or not recover to have the previous volume. This borrows material is used to fill specifically the foundation of the weir and also the foundation of the main canal. For the Burqa project this material has been identified from the two places at a GPS location
Point 1=E 0574902m, N 1262972m and Elevation =1575m
Point 2= E 0574889m, N 1263169m and Elevation=1561m
The first option is found at a distance of 4.5km to the south direction from the head work site. The second option is present also south of the head work site 4km far away. For both borrow sites the road is accessible easily and both are found at the foot of the ridge. The quality of this material is excellent. The quantity is also available in the required amount so that it is greater than 4000m3. The selected sites have not belonged to

anyone. It is found along the road cuts. After excavation of this fill material the road should be cleared and maintained.

3.2.3.2 Rock for Masonry and Crushed Coarse Aggregate

The granular material is a masonry rock which may produce either by the manpower, by an excavator or by blasting. The distance of the construction material from the site plays a great role in the costs of the total project.

The Burqa project granular material is available around the headwork area at a distance of about 2.5km Northeast direction of the headwork. It is welded basaltic rock, which has shiny minerals of the olivine, pyroxene plagioclase. This material, which has selected at nearest place, suffices the need of the project.

It is slightly fractured with some joints, which is excellent quality, and so it can be used for the construction of both foundations of the weir and the main canal. By simple estimation, the quantity of this material is more than 1000m3. This material is present at the surface and subsurface. If the rock found at the surface is finished, the subsurface one will be excavated. This selected place for quarry site of masonry is accessible through the farmland. The selected site is also presented within the farmer's farmland and so which needs some an appropriate compensation payment for the owners of the selected site because their farmland is going to be excavated from the quarry site.

Table 3-1: Location of rock for masonry

GPS Location			
Material type location	**Easting**	**Northing**	**Elevations**
Masonry rock point (1)	0575495m	1264274m	1587 masl
Masonry rock point (2)	0575529m	1264103m	1566 masl

3.2.3.3 Fine Aggregates

Borrow areas for fine aggregate or natural sand have been assessed starting from the project stream itself. The sand is one of the main raw materials for construction. This material is used for mixing of the cement, fine aggregates, coarse aggregates, for the construction of concrete structure.

The availability of this material at nearby place highly influences on the costs of the project. If the sand quarry is present at a far distant place the costs of the project increases. The sand quarry for the Burqa project is identified in Mersa town within the beds of the two rivers Melka Chefie River and Abuarie River. These rivers are found in the south and North direction of the Mersa town respectively. They bound the town. The sand quarry belongs to the licensed ownerships. For instance the ownership of the Melka chefie river quarry is Ato Hussien Yasin. The ownership of the Abuarie River quarry site is Ato Demilew Gizaw. These quarry sites are accessible and just located 100meters downstream of the bridges. This quarry site is present at a distance of 77kms from the head work of the Burqa project in which 17kms is all weathered roads starting from the Hayqe town. The other 80kms is asphalt road from Hayqe to Mersa.

Table 3-2: Location of rock for aggregate

GPS Location			
Material type location	**Easting**	**Northing**	**Elevations**
Melka Chefie River quarry	0572484m	1288566m	1565 masl
Abuarie River quarry	0571437m	1290857m	1618masl

3.2.3.4 Water

Water for construction purposes can be found from the project stream, burka, itself. The stream is perennial throughout the year that there is some amount of flow along its course. During this field time the stream flow was more than 160L/second

3.3 Hydraulic Design of Headwork Structure

3.3.1 Headwork Type Selection

Looking the availability of natural construction materials and considering the river features and expected flood amount, broad crest type of weir is chosen. As it is:

- Simple for construction
- Weir section is expected only 50% of the peak flood while the remaining flood will pass over the overflow section of the river course.

3.3.2 Weir Height Determination

The following major factors have been seen in determining the weir/intake crest level:

- Maximum command area elevation
- Deriving head of the intake structure
- Main canal slope

- Loss
- Lowest Point of river center

3.3.3 Base flow of the River

The study team has assessed that the stream is not used for irrigation along its entire course except at the proposed diversion site where farmers are using thehf for traditional SSI (Irrigation Infrastructure Report). The study team has measured the base flow using the float method during the feasibility Study time is160l/s. Out of this 122.4 l/s will be required for the proposed scheme and the rest will be released for downstream The purpose of releasing the 38 l/s to downstream is for the sake of downstream users water balance

3.3.4 Weir Dimensions

3.3.4.1 Weir crest

Crest Length

- Lacey's regime width, $L = 4.75 * \sqrt{Q}, = 4.75 * \sqrt{76.96}$ = 41 m.
- Actual river section width of the over flow section of the river is = 26m

3.3.4.2 Discharge over the weir section

- Design discharge, Q = 76.92 m^3/s

3.3.4.3 Top and bottom width

According to the Bligh's formula, top and bottom width of the weir body is determined as follows

- Input Data:

P: Height of weir (m) = 1.2

He: specific energy head (over flow depth + approaching velocity head (m)) = 2.12

σ : Specific weight of weir body (σ = 2.3 for cyclopean concrete)

Top width, $B = \frac{He}{\sqrt{\sigma - 1}}$ = 1.289 ----taken 1, where He = 1m

Bottom width, $B' = \frac{He + P}{\sqrt{\sigma - 1}} = 4.44m, where...p = 2m; B = 3m....adopted$

Provide 1m and 3 m top and bottom width respectively, which will be tested for adequacy during stability analysis.

3.3.5 U/S and D/S HFL Calculation & Determination

From the stage –discharge curve prepared in section I, in Hydrology Part the high flood level before construction (i.e. D/s HFL) corresponding to the design flood is 1547.58m. a.s.l.

D/s HFL = 1547.58 m a.m.s.l -------------------------------------- (a)

U/s HFL = U/s bed level + weir height + H_d -------------------- (b)

H_d is the depth of water over the weir crest. This is calculated by assuming broad crested weir formula.

$Q = C * L * H_e^{\frac{3}{2}}$, where C=1.7

$H_e = \left(\frac{Q}{C*L}\right)^{\frac{2}{3}}$ = 1.43m

The velocity head, h_a is computed from the approach velocity as shown below

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

Where g: acceleration due to gravity = 9.81m/sec^2

Va is Approach velocity determined by

$$V_a = \frac{Q}{Lxh_d}$$

L is Weir crest length = 26 m,

h_d is flow depth over the weir and also,

$$h_d = H_e - h_a$$

$$h_a = H_e - h_d = \frac{\left(\frac{Q}{L*h_d}\right)^2}{(2g)} = \frac{\left(\frac{76.69}{(26)*h_d}\right)^2}{(2*9.81)}$$

By trial and error method, h_d is found to be 1.09 m

- $h_a = H_e - h_d$ = 1.43m-1.29m = 0.40m
- Velocity head, h_a = 0.40m

U/s HFL =U/s TEL –velocity head =1549.13m a.s.l – 0.4m = 15478.78m a.s.l

- ❖ Afflux

⇒ Afflux = U/s HFL- D/s HFL = 1548.78m a.s.l – 1547.74 m a.s.l = 1.03m.

From the flood level analysis, it is seen that the flood overtops the banks of the river u/s of the structure. This condition is allowed to take place as it doesn't bring pronounced negative impacts on the structures, rather than constructing bulky structures to confine it.

3.3.6 Hydraulic Jump Calculation

As discussed in the geologic report, the river bed is alluvial deposit and hence stilling basin for energy dissipation is required. Both left and right side banks are not sound rock, a wing walls are required at u/s and D/s sides, so as to protect the scouring of the bank due to the formation of jumps, and not to follow the river out of a river bank in high flood cases.

In the determination of pre and post jump depths the basic energy equation between upstream face of the weir and the point where the hydraulic jump starts to form (where depth y1 is achieved). The head loss between these two points due to friction is assumed to be zero. Accordingly: P + He = Y1 + Hv

P + He = Y1 + q2/2gy12) ----------------------------- (1)

Where: P = Weir height

He = Head over the crest including the velocity head (m)

Hv = Velocity head at the point where hydraulic jump starts to be formed

= (q2/2gy12)

q = Discharge intensity (m3/sm)

g = acceleration due to gravity=9.81m2/s

From equation_1 above the value of Y1 was determined for different values of q using equality solved by solver and goal seek by Microsoft excel 2010 using the known values of He and weir height determined from previous sections and ogee weir formula using the corrected discharge coefficient.

The value of Y2 was determined by the following formula:

$Y2= [\frac{Y1\sqrt{8F1^2+1}}{2}-1]$ and F1, the Froude number= $\frac{V1}{\sqrt{(gY1)}}$

Water level at pre jump depth = Bed level +Y1

Water level for post jump depth = Bed level +Y2

The total energy levels (TEL) and the high flood levels (HFL) both in the upstream and downstream reaches of the proposed weir axis are computed using standard procedures. These energy and flood levels are required to compute basic hydraulic dimensions of the weir, to effect structural design of the overflow section and other components of the weir and to fix the levels of flood protection and retaining walls of the proposed head work structures.

- Weir crest length = 26m
- Weir height = z = 1.2m
- Pre-jump depth = y_1 = 0.45 m
- Post -jump depth =y_2 = 1.72m

Neglecting losses between point P and D and considering similar datum

$z + H_e = y_1 + h_a$

But, H_e = 1.43m

$$q = \frac{Q}{l} = \frac{2.95m2/s}{26m} = 2.95m^2/s$$

But Y_1 =0.45 m for Q =76.69m from the spread sheet calculation

V_1=q/y_1=2.95/0.45=6.55m/s

$$F_r = \frac{V_1}{\sqrt{gy_1}} = \frac{6.55}{\sqrt{9.81 * .45}} = 3.12$$

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 8 * {F_r}^2} - 1\right) \qquad y_2 = \frac{0.45}{2}\left(\sqrt{1 + 8 * 03.12^2} - 1\right) = 2.95m$$

Hydraulic jump length (L) for Fr=13 from the graph L=5*(y1-$y_{2)}$ =5*(2.6) = 13 but we take only 8m because the downstream stealing basin is lowered by 0.8 depth from original ground level for to safe the tail water came to over post jump and reduced the jump length by counter of tail water depth including downstream sill at the end of the basin

3.3.7 Impervious floor

3.3.7.1 D/s impervious floor (Ld)

For under seepage the worst condition would be when the water on the upstream side is at the level of the weir crest & there is no tail water. Seepage head loss at

1) Pond level case:

Hs = crest level –bed level 1546.69-1547.19 = 1.2m

2) Maximum flood case:

Hs = U/s HFL- D/s HF = 1548.77 m-1547.74m = 1.03m

Therefore maximum seepage head occurs when water is stored up to the pond level and there is no water on the d/s.

= Bligh's constant, C_b depend on the type of the foundation.

$$L_d = 2.2 * C_b * \sqrt{\frac{H_s}{10}} \qquad L_d = 2.2 * 6 * \sqrt{\frac{2}{10}} = 11.50\ M$$

Therefore total impervious floor length is taken to be 11.2m long.

3.3.7.2 U/S Impervious Floor Length, (Lu)

The u/s impervious floor, (L_u) = 1.5*d_1 = 1.5*0.45 = 0.8m. But total length of the u/s impervious floor is taken to be 1.5m long.

3.3.7.3 Floor thickness (t)

The thickness of the apron floor for different reach is summarized in table below

Table 3-3: Apron floor thickness

Distance from end of stilling basin (m)		**Thickness**	**unit**
From	Up to		
0	5	1.01	m
5	8.0	1.43	m
8	11.5	1.63	m
11.5	13	0.40	m

3.3.8 Cut off Depth Calculation

The riverbed may be secured during flood flow and large scour holes may develop progressively adjacent to the constructed head work structures, which may cause undermining of the structure. Finally, the structures become out of their functions. Hence, to provide a proper cutoff, it is important to determine the scour depth. This depth can be computed by Lacey formula.

And the scour depth(R)) is given by;-

$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$ Where $f = 1.76 * \sqrt{d}$,

d=mean diameter of particle size in mm $upto\ possible\ scour\ depth.$

$$q = discharge\ intesity = \frac{maxumumflood}{river\ width(wetted)}$$

The mean particle size of the headwork site is found to be 50.1mm.

$$f = 1.76 * \sqrt{d} = 1.76 * \sqrt{50.1} = 12.5$$

$q = \frac{Qpeak}{L} = \frac{76.69}{26}$ =2.95 m^3/s/m

$$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 2.83\ taken\ as\ 2.8m$$

The scour depth(R) again should be multiplied by proper scour factor which depends on the condition of reach bends. As per Lacey theory the following bend conditions have been formulated as general rule as shown in table below.

Table 3-4: Scour depth factor

Type of reach	Mean value of scour factor
Straight	1.25
Moderate bend	1.5
Sever bend	1.75
Right angled bend	2
Nose of Guide Banks	2.25

3.3.9 Energy Dissipation

The Froude number and the relation of post jump depth to the tail water depth indicate which type of energy dissipation mechanism to use. USBR type IV stilling basin is recommended for Froude number=3.26 which is 2.5<Fr<4.5, the jump is in transition stage.there is no effective dissipation waves persist. The water depth in the basin should be about 10% greater than the computed conjugate depth. The jump length is found from L/d2 versus Froude number graph in USBR 1987 design of small dams.

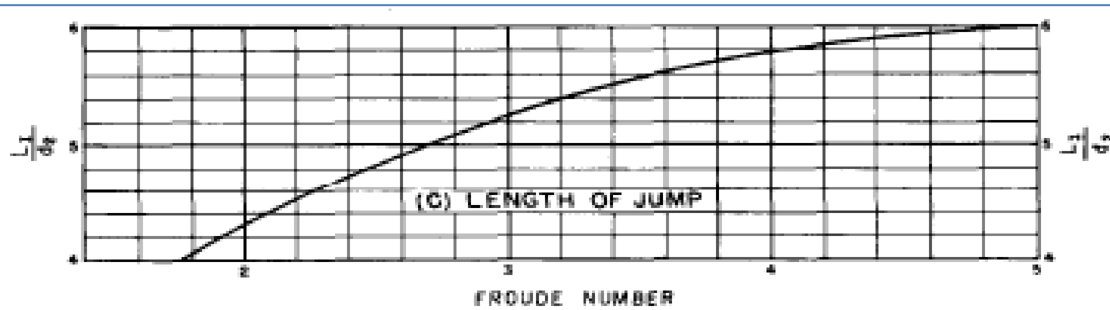

Figure 9-39.—Stilling basin characteristics for Froude numbers between 2.5 and 4.5. 288-D-2425.

Figure 3-2:-Froude number graph

5=L/Y2=>L=5*Y2 =5*(1.72-0.4)=11.5m and 1545.7masl is the basin level bt the jump is located at 1546.2 on the sloping apron , the more flatter downstream apron helps to resist the impact. More over to match the tail water level to the conjugate depth level large boulders downstream are recommended.

3.3.10 Determination of Scour and Cutoff Depth

Depth of scour below high flood level

$R=1.35\left(\frac{q^2}{f}\right)^{1/3}$, $f=1.76\sqrt{d}$

Where : - d is average particle size in, mm.

But as it is stated in the geological report the present morphology of the burka River channel is a function of a number of processes and environmental conditions, including the composition of the bed and the banks (moderately weathered, fractured boulders along right side whereas loose soils and recent coarser deposit at the left bank and bed); due to this both upstream and downstream cutoffs have been provide 1.5m and 3m depth below river bed level were recommended. The scour depth below river bed level is found out to be 3.04m but taken as 3m. But, the subsurface geology report indicates the older alluvial deposits, on which the structures have to lay is obtained at a depth of 1.78m. Hence, for this case, 1.5m scour depth below river bed level depth has to be provided.

Table 3-5: Scour depth computation cutoff provision

U/s cutoff	**value**	**unit**
Provide u/s cutoff 1.25R below the u/s water level	3.41	m
U/s High flood level	1548.77	m.a.s.l
Level of bottom of u/s cutoff	1545.36	m.a.s.l
Hence Depth of u/s cutoff below u/s bed level	1.13	Take1.50m
Therefore bottom level of u/s cutoff	1544.99	m.a.s.l
D/s cutoff	**Value**	**unit**
Provide downstream cutoff 1.5R below the d/s water level	4.09	m
downstream High Flood Level	1547.739	m.a.s.l
Level of bottom of d/s cutoff	1543.65	m.a.s.l
Hence Depth of d/s cutoff below d/s bed level	2.04	Take3.00m
Therefore bottom level of d/s cutoff	1542.69	m.a.s.l

3.4 Stability Analysis of weir

Stability analysis is carried out to see the already determined weir section is safe against overturning, sliding, tension. The stability analysis is carried out considering the effect of the following forces.

- Water pressure
- Weight of the overflow weir section

• Sediment load

The extreme load combination is the case where the head is at crest level of the weir and there is no flow over the weir (static case)

A. Stability of the weir on Static condition

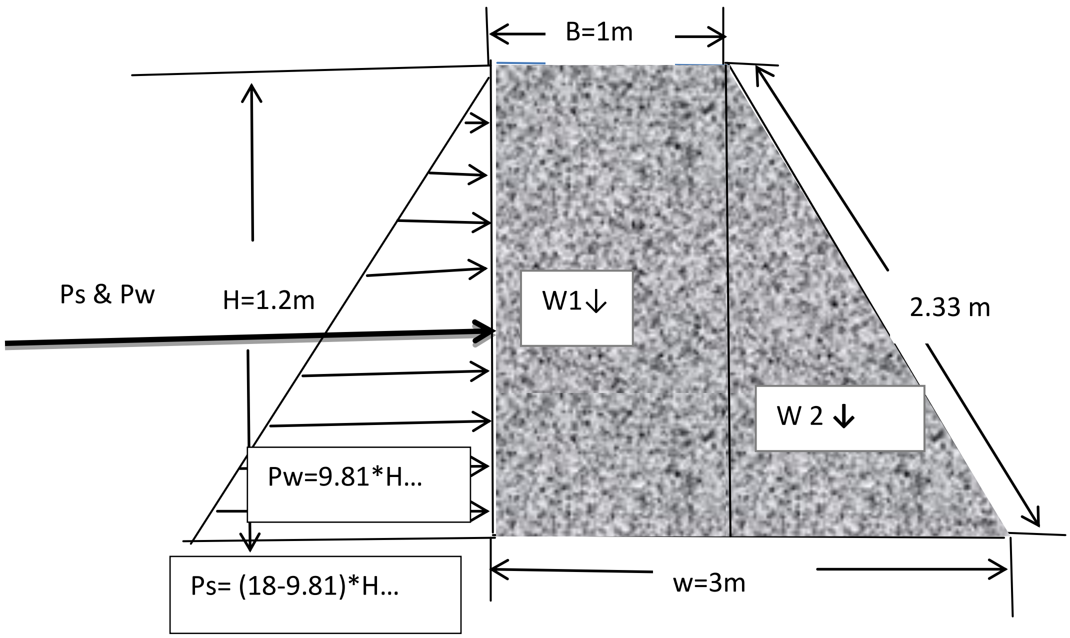

Figure 3-3: Loading arrangement on weir body

Table 3-6 : Weir body stability analysis

Code	Dimension		Load		Lever arm	Moment	
	Width	Depth	Vertical	Horizontal	R	Positive	Negative
Pw	3.00	1.20	0.00	17.69	0.40	0.00	7.09
W1	1.00	1.20	27.65	0.00	2.50	69.11	0.00
W2	2.00	1.20	27.65	0.00	1.33	36.86	0.00
Ps	9.84	1.20	0.00	2.40	0.40	0.00	0.96
Sum			55.29	20.09		105.98	8.05
Factor of safety against							
Over turning = (M+ve/M-ve) >1.5				Fo	13.17	>1.5	OK
Sliding = (µxFv/Fh),---µ=0.7 >1.5				Fs	1.93	>1.50	OK
Tension: X= (Net Moment/Sum Fv), e=x-B/2, e<B/6				X	1.77		
		B/6=	0.50	e	0.27	<B/6	OK

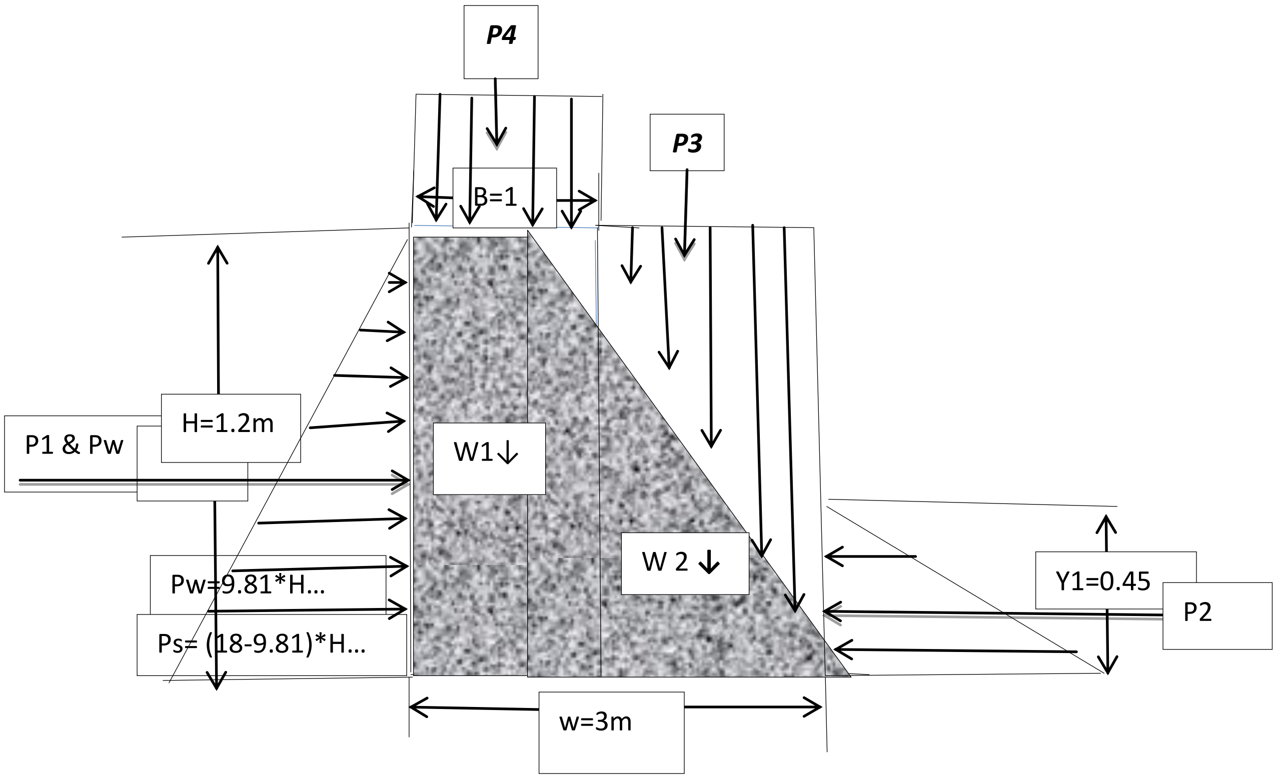

Figure 3-4 Loading arrangement for dynamic condition on weir body

Table 3-7: Weir stability analysis

Code	Dimension	Load			Lever arm about the toe	Moment	
	Width	Depth	Vertical	Horizontal		Positive	Negative
P1	3.00	1.202	0.00	17.69	0.40	0.00	7.09
P2	0.45	0.447	0.00	0.98	0.15	0.15	0.00
P3	2.00	3.839	37.66	0.00	0.67	25.11	0.00
P4	1.00	2.637	25.87	0.00	2.50	64.67	0.00
W1	1.00	1.202	28.85	0.00	2.50	72.12	0.00
W2	2.00	1.202	28.85	0.00	1.33	38.46	0.00
U1	3.00	1.202	35.37	0.00	1.50	0.00	53.06
U2	3.00	0.500	7.36	0.00	2.00	0.00	14.72
Ps	9.84	1.202	0.00	2.40	0.40	0.00	0.96
Sum	25.29	13.43	78.49	19.11	11.45	200.50	75.83

Factor of safety against,							
Over turning = (M+ve/M-ve) >1.5				Fo	2.64	>1.5	OK
Sliding = (µxFv/Fh),--- µ=0.7 >1.5				Fs	2.88	>1.50	OK
Tension: X= (Net Moment/Sum Fv), e=x-B/2, e<B/6				X	1.59		
		B/6=	0.50	e	0.09	<B/6	OK

From stability analysis, the designed weir section is over safe. To be economical, Provide 1m top width and 3.0m bottom width.

3.5 Design of Divide wall, retaining wall Under Sluice, and Canal outlet

Divide wall is designed in order to create separation between outlet canal and natural river course. The divide wall allows safe and stable base flow to the canal outlet. Flow turbidity created by current flow impact over the weir/intake body is reduced.

3.5.1 Retaining Wall Design

As recommended in geologic sub-section of this document, at the head work all the u/s and d/s left-right banks of the river have a chance to be subjected to erosion and the provision of protection structure has been indicated. Hence, masonry retaining wall along with main canal is selected as protection work structure. It is designed as a gravity wall type. Its stability is also checked against overturning, sliding, and tension developed within the body of the structure. The height of Maximum design flood governs the height of the wall with some free board provided

The existing topographical condition at the weir axis and HFL are considered to be most governing parameters for fixing the wall height for upstream and downstream of the weir including free board. Whereas the wing walls that are keyed to the upstream hills to contour to upstream high flood level are masonry retaining walls having varying heights. Below the recommended of the foundation height there is from 1 m to 1.5m cut depth and 0.1 m lean concrete .The structural dimensioning and stability analysis are set in table below. Stability has been done for both upstream and downstream retaining walls.

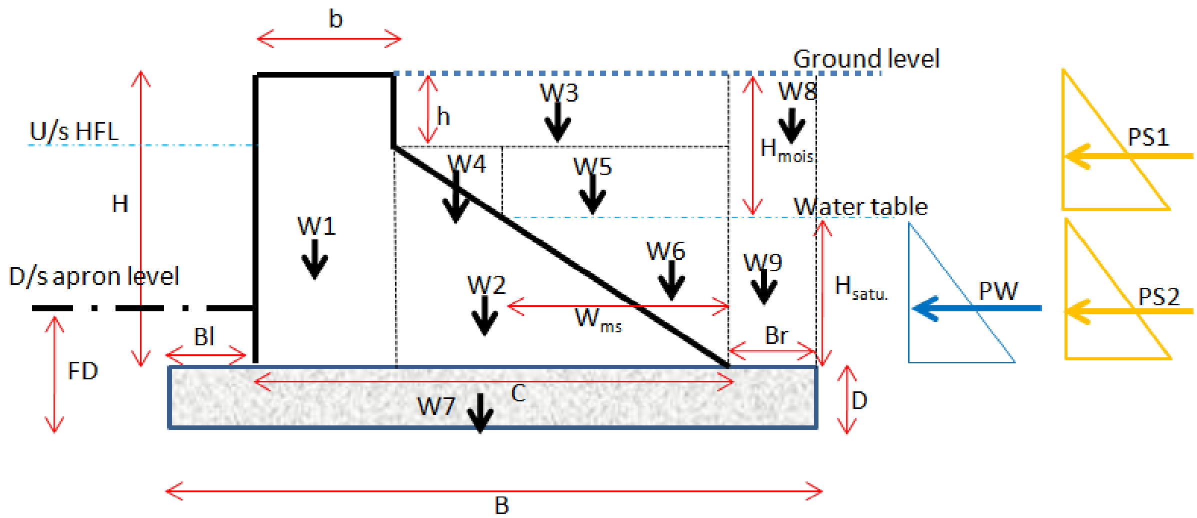

Figure 3-5 Loading arrangement on retaining wall

Table 3-8: Stability analysis of upstream retaining wall

A) Data		Value	Unit
1.Hudraulic data			
U/s apron level	=	1546.49	masl
U/s HFL	=	1549.49	masl
Free board	=	0.50	m
Foundation depth, FD	=	0.50	m
Depth of saturated bottom soil, Hsatu.= 2/3*H	=	2.33	m
Depth of moistened upper soil, Hmois. = H - Hsatu.	=	1.17	m
Width b/n moist and saturated soil, Wms.	=	1.94	m
2.Material data			
Unit weight of masonry (gmasn.)	=	24.00	KN/m3
Unit weight of bedding concrete (gconc.)	=	23.00	KN/m3
Moist unit weight of backfill (gmois.)	=	18.00	KN/m3
Saturated unit weight of backfill (gsat.)	=	20.00	KN/m3
Submerged unit weight of backfill (gsub.)	=	10.20	KN/m3
Unit weight of water (gwat.)	=	9.80	KN/m3
Angle of internal friction (∅)	=	30.00	Degree
Active internal friction coefficient (Ka)	=	0.33	
The friction angle b/n masonry & concrete	=	33.00	Degree
The friction angle b/n concrete & soil	=	20.00	Degree
Maximum allowable compressive strength in masonry	=	1.00	N/mm2
Maximum allowable tensile strength in masonry	=	0.10	N/mm2

Maximum allowable compressive strength in concrete	=	20.00	N/mm^2
Maximum allowable tensile strength in concrete	=	3.00	N/mm^2
Maximum soil bearing pressure	=	150.00	kN/m^2
B) Required dimensions of the retaining wall for stability consideration			
Masonry			
Top width, b	=	0.50	m
Bottom width, C	=	3.00	m
Top section height, h	=	0.50	m
Total height, H=U/S HFL- D/s apron level + FB + FD-D	=	3.50	m
Concrete base			
Thickness of masonary base, D=H/8 to H/6	=	0.50	m
Width of left side concrete base, Bl=D/2 to D	=	0.00	m
Width of right side concrete base, Br=10 to 15 cm	=	0.00	m
Total width of concrete base, B=C+Bl+Br	=	3.00	m

C) Load calculation

No.	Code of load	Load (KN)	Moment arm formula	Moment arm (m)	Moment (KNm)
1.0	**Self weight**				
1.1	W1	21.00	b/2	0.25	5.25
1.2	W2	75.00	b+((C-b)/3)	1.33	100.00
2.0	**Soil (vertical)**				
2.1	W3	22.50	b+((C-b)/2)	1.75	39.38
2.2	W4	3.33	b+(2/3)(C-b-Wms)	0.87	2.90
2.3	W5	11.67	$(C-W_{ms})+(W_{ms}/2)$	2.03	23.66
2.4	W6	45.37	$(C-W_{ms})+((2/3)W_{ms})$	2.35	106.70
3.0	**Soil (horizontal)**				
3.1	PS1	4.08	$H_{satu.}+(H_{mois.}/3)$	2.72	11.12
3.2	PS2	9.26	$H_{satu.}/3$	0.78	7.20
4.0	**Water (horizontal)**				
4.1	PW	26.68	$H_{satu.}/3$	0.78	20.75
5.0	**Uplift (masn/conc)**				
5.1	PU1	5.15	(2/3)C	2.00	10.29

D) Stability analysis at the level of masonry/concrete base interface

		Value	**Unit**
i. Overturning			
Sum of stabilizing moment	=	277.89	KNm
Sum of destabilizing moment	=	49.35	KNm
Factor of Safety	=	5.63	Safe
ii. Sliding			

Sum of vertical force	=	173.73	KN
Horizontal sliding force	=	40.02	KN
Horizontal resisting force	=	112.82	KN
Factor of Safety	=	2.82	Safe
iii. Tension			
Net moment	=	228.53	KNm
Net vertical force	=	173.73	KNm
X	=	1.32	m
e	=	0.18	m
C/6	=	0.50	Safe
iv. Direct compressive/tensile stress at the concrete surface			
Direct compressive stress (at the heal) base of concrete	=	0.08	N/mm^2
Direct compressive/tensile stress (at the toe) base of concrete	=	0.04	N/mm^2
		+ve, therefore no tension	
5. Bending stress at the concrete bottom surface			
$W=g_{mason.}H$	=	84	KN/m
M	=	94.50	KNm
b	=	1.00	m
D	=	0.50	m
$I=bD^3/12$	=	0.01	m
y=D/2	=	0.25	m
Z=I/y	=	0.04	m
σ = M/Z	=	2268.00	KN/m^2
σ=	=	2.27	N/mm^2

E) Stability analysis at the level of concrete/soil base interface

No.	Code of load	Load (KN)	Moment arm formula	Moment arm (m)	Moment (KNm)
1.0	**Self weight**				
1.1	W1	21.00	Bl+b/2	0.25	5.25
1.2	W2	90.00	Bl+b+((C-b)/3)	1.33	120.00
1.3	W7	34.50	B/2	1.50	51.75
2.0	**Soil (vertical)**				
2.1	W3	22.50	Bl+b+((C-b)/2)	1.75	39.38
2.2	W4	3.33	Bl+b+(2/3)(C-b-Wms)	0.87	2.90
2.3	W5	11.67	$Bl+(C-W_{ms})+(W_{ms}/2)$	2.03	23.66

2.4	W6	45.37	Bl+(C-W_{ms})+((2/3)W_{ms})	2.35	106.70
2.5	W8	0.00	B-(Br/2)	3.00	0.00
2.6	W9	0.00	B-(Br/2)	3.00	0.00
3.0	**Soil (horizontal)**				
3.1	PS1	4.08	D+Hsatu.+(H mois./3)	3.22	13.16
3.2	PS2	9.26	D+($H_{satu.}$/3)	1.28	11.83
4.0	**Water (horizontal)**				
4.1	PW	26.68	D+($H_{satu.}$/3)	1.28	34.10
5.0	**Uplift (conc/soil)**				
5.1	PU2	41.65	(2/3)B	2.00	83.3

F) Stability analysis at the level of masonry/concrete base interface

1. Overturning		**Value**	**unit**	**Remark**
Sum of stabilizing moment	=	349.64	KNm	
Sum of destabilizing moment	=	142.37	KNm	
Factor of Safety	=	2.46		Safe
2. Sliding				
Sum of vertical force	=	186.72	KN	
Horizontal sliding force	=	40.02	KN	
Horizontal resisting force	=	67.96	KN	
Factor of Safety	=	1.70		Safe
3. Tension				
Net moment	=	207.27	KNm	
Net vertical force	=	186.72	KNm	
X	=	1.11	m	
e	=	0.39	m	
B/6	=	0.50		Ok!

4. Direct stress at the soil surface (bearing and tension)

Direct compressive stress (at the heal) base of soil	=	110.78	KN/m^2	
	Which is < P_{allow}=150KN/m^2			Ok!
Direct compressive/tensile stress (at the toe) base of concrete	=	0.01	N/m^2	
	+ve, therefore no tension			

3.5.2 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 to be 0.5 m higher than the minimum bed level. Hence the sill level of the under sluice = 1546.8. 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 towards it. Considering this, the opening size of the gate is 1m*0.7m 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 following formula.

- The capacity should be at least five times the canal discharge to ensure proper scouring.
- The 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. The size of the under sluice is 1m$\times$ $0.7m$ (height *width)

3.5.3 Canal outlet level

The head regulator is provided on both the left and right side .The sill level of this head regulator is fixed from different angle observations. The main conveyance system is more than 1km which passes more gullies and undulating alignment. Hence this level is fixed based on the optimum route alignment and the maximum irrigated command level including minor and major losses criteria. Based on this condition, the sill level is fixed to be 1547.19m.

3.5.4 Outlet capacity

The minimum command area is determined by the minimum flow of the river. But the canal capacity should be determined for the maximum command area and the corresponding discharge. In this case the outlet capacity is fixed considering maximum duty and command area and 1.5 correction factors are considered to account the variation of duty.

$\Rightarrow$ Outlet capacity = Duty x command area x correction factor

Where, maximum duty for 12 hr irrigation = 1.53 L/s/ha

Command area = 80ha.

$\Rightarrow$ Outlet capacity = 1.53 L/s/ha x 80ha = 122.4L/sec,

- **Outlet size**

From the weir discharge formula the outlet size is determined as follows

$Q = CLHe^{3/2}$

Where; C = coeff. of discharge = 1.7

L = Length of water way (m)

He = head above sill level (neglecting the velocity head) = 0.5m

$$L = \frac{Q}{CHe^{3/2}} = \frac{0.113}{1.7x0.5^{3/2}} = 0.19m$$

⇒Adopt water way length = 0.5m

Hence, provide an outlet size of 0.5m x 0.5m (length x height) .The gate of the off takes canal is to be vertical sheet metal of 0.5m x 0.50m for the closure of the opening space. Provide some extra dimensions for groove insertion. Gross area of sheet metals for the off take canal gate will be 0.6m x 0.80m (allowing 5cm insertion for grooves and above the weir crest level). The grooves are to be provided on the walls using angle iron frames at the two sides of the gate openings.

Trash racks of diameter 14mm with c/c spacing of 10cm has to be provided u/s of the gate to prevent entry of debris to the canal.

3.5.5 Breast Wall and Operation Slab

To avoid spilling of water during HFL over intake gate, a R.C.C wall is provided from the gate top level up to the HFL (i.e. Known as breast wall). A vertical raised gate is designed for the head regulator. These gates are sliding over the breast wall-using a spindle during opening and closing. For the operational purposes, operation slab is also provided at the top of breast wall. The thickness of the breast wall & operation slab is simply determined from recommendations (point of construction) rather than the imposed load. The thickness required for the imposed load is less than this nominal value 20cm. For the breast wall & operation slab, the minimum reinforcement area is taken as 0.15% in both directions unit widths.

Hence area of steel per one meter width,

A_{steel}=0.0020*W*t,

Where, w= width =1m &t=thickness both in cm = 200mm and taking a 0.2% minimum reinforcement area

A_{steel}=0.0020*1000*200 =400mm^2

A_{steel}=400mm2, Provide ϕ 12 @ C/c 200 mm

I. **Checking the appropriateness of space** : -Considering the cover thickness of 50 mm, effective depth, de=t-(50+ϕ/2) =200-(50+12/2) =144 Hence spacing of reinforcement should be less of the following values:-

Three times the effective depth =3*144=432mm

450mm

The spacing of bars, which is 200mm, is less of the above listed values. **Hence, it is acceptable!**

II. **Checking the adequacy of steel bar thickness** A_{steel}=3. 14*12^2/4*5= 678.24 mm2

Therefore the actual provided steel area per meter width is 678.24 mm2/m>400 mm2/m Ok!

Hence, thicknesses of 20cm for the breast wall & operation slab are adequate.

Therefore, provide the reinforcement bar of ϕ12mm @200mm c/c spacing in both directions with reinforcement covers of 40mm for the breast wall.

SECTION III: IRRIGATION INFRASTRUCTURE

4. IRRIGATION AND DRAINAGE SYSTEMS DESIGN

4.1 Irrigable 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 sloping type. Its elevation range is from 1394 to 1548 meters above sea level. The slope gradient also ranges from flat (1%) to gently sloping (5%). 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 the right and left side of Burka River (to the North East and North West side of the river). The natural topographic feature of the command area has inclined from the South-East to the North-West direction.

4.1.2 Climate

As per the hydrological analysis and on the basis of the traditional Ethiopian Agro-Ecological Zones (MOA, 2001), the UGDWIP area is basically classified as Moist Woina Dega (sub-moist cool) agro-ecological zone, indicating better moisture condition in the area in wet seasons. There is no belg rain season in the project area. Despite the fact that the *Meher* rains are considered adequate, there is notable variation in terms of onset, distribution and withdrawal from year to year affecting crop production in general and crop productivity in particular.

As the project site has no its own meteorological station, Bokeksa meteorological Station (for rainfall and minimum and maximum temperature) and Bokeksa meteorological Station (for relative humidity, wind speed and sunshine hour) meteorological stations data were used for the project study as long as these stations are relatively near to the proposed command area. In general, the sources of meteorological data are the National Meteorology Service Agency (NMSA).

4.1.3 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 soils which can be classified as imperfect 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 (for further detail see the Agronomy Study of the same project.

4.1.4 Existing Irrigation Practices in the Project Area

The pressure of survival and the need for additional food supplies to meet the demands of the increasing population is necessitating a rapid expansion of irrigation schemes. Thus, irrigation is becoming a basic part of well-developed agriculture wherever there is water and irrigable land potential. Accordingly, traditional irrigation practices are under taken by individual farmers that use the river flow to the right side is with laborious temporary canal. So, the farmers in the project area are very much interested in the idea of upgrading the traditional scheme to modern scheme.

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”.

The corresponding values of the crop water requirements of the proposed crops of the project are presented in the Agronomy Study of the same project.

4.2.2 Irrigation efficiency (Ep)

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).

On the basis of these factors, the project has planned to impose surface irrigation method (using furrows). The canal system is unlined other than the main canals. Hence, the conveyance efficiency has been estimated to be 90%, distribution efficiency 85%, and field application efficiency 80%. As a result of these the overall irrigation efficiency has been estimated to be 45.9%. According to soil Lab result, soils of the command area are predominantly characterized as heavy clay soils.

4.2.3 Irrigation 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 Burka diversion weir irrigation project has showed a maximum net irrigation water requirement (NIWR) in the month of March with the amount of 6.24mm/day for 12 working hours (for overall proposed crops).

However, for the designing of the irrigation water application and the flows in the entire canal systems, from the overall proposed crops the average NIWR was used for irrigation duty calculation. Accordingly, 6.54mm/day is NIWR for dry season crops and hence taken for the irrigation project duty calculation as indicated here below:

For Burka River Diversion Irrigation Project, it decided to adopt 80% field application efficiency, 85% distribution efficiency, and 90% conveyance efficiency as the soil is heavy clay soil and the canal systems are estimated to be unlined except the main canals. Hence, the overall/project efficiency for the selected surface irrigation method has been estimated to be 45.9% (80/100*90/100*85/100) which is rounded to 46%.

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

$$GIWR = 3/0.46 = 6.24 \text{ [mm/day]}$$

The GIWR, 6.24 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:

$$\text{Duty (D)} = GIWR \times 1000 \times 10 / (t \times 80 \times 80)$$

Where; Duty – the duty [l/s/ha]

GIWR – Gross Irrigation Requirement [mm/day]

t – Daily irrigation or flow hours [hrs]

The duty for the GIWR of 6.24mm/day and 12hours of daily irrigation time (t = 12), is supported to be used with furrow irrigation method. Hence, Duty for 12 working hours, as the site is nearer to farmers' village and local farmers have experiences in irrigation, is computed as follows:

D = (6.24 x 1000 x 10) / (12 x 3800)=1.4l/ha

4.2.4 Irrigation methods

Among the different irrigation systems furrow irrigation system will be used for the project area; and the irrigation water will be obtained from Burka River and by constructing diversion weir and convoying the water commonly through the two lined main canals, two flumes & earthen canals (SC, and TC) 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.

4.3 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 comprises two main canal, two secondary canals and tin tertiary canals as shown on the layout Drawings. The main canal runs for most of its length parallel to the contours and several changes of direction are necessary to follow the topography. It crosses two main gullies, one foot path. The right and lift main canal are masonry lined for a length of 1296 meters and 471meters starting from the weir outlet respectively to make maintenance easier since this part of the canal may be subjected to flooding during high flood flows.

4.3.1 Conveyance System

The conveyance system consists of two Main canals to irrigate total command area of 80 ha. The main canal starts from Water abstraction site on right and left sides to convey water for a length of 1296 m and 471m respectively.

The two main canals are aligned along contours and the right main canal supplies to one secondary unit and five tertiary canals. The left main canals also supplies to one secondary unit and two tertiary canals.

4.4 Design of the Canal System

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:

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

Where Q= discharge (m^3/s)

R= Hydraulic radius (Flow area/wetted perimeter)

S= Hydraulic gradient= Manning's roughness coefficient, n=0.024 is adopted for the earth channels and n=0.018 for the masonry lined part of the main canal

4.4.1 Main Canal

Depending on the site specific condition, appropriate slope is provided. Hydraulic parameters of the main canal are shown below. Also on right main canal at a chain age of 773.49m, 1050.81m and 1834.15m, flume structure is provided where the canal crosses the gullies.

Table 4-1: Hydraulic Parameters of left main canal

Reach	Value of n	Bed Slope	BW (m)	FSD (m)	FB (m)	V (m/sec)	WP (m)	SS	Q_R cumecs	$Q_D$cumecs
0 to 335	0.018	500	0.30	0.13	0.2	0.42	0.56	0:1	0.042	0.017

Table 4-2 Hydraulic Parameters of Right main canal

Reach	Value of n	Bed Slope	BW (m)	FSD (m)	FB (m)	V (m/sec)	WP (m)	m	Q_R cumecs	QDcumecs
0 to 1296	0.018	850	0.5	0.27	0.20	0.55	0.96	0:1	0.08	0.11

BW= Canal bottom width QR= Required discharge

FSD= Full supply depth QD= Designed Discharge

FB= Free board

m = Side slope

V= Velocity

4.4.2 Secondary Canals

To secondary canals are provided in the lay out system one emerges from right main canal and the second from left main canal. Since the secondary canals run across contours, provision of successive drops is inevitable to fulfill the hydraulic requirements.

Table 4-3: Hydraulic Parameters of secondary canals

Name	Reach (m)	Value of n	Bed Slope	BW (m)	FSD (m)	FB (m)	V (m/sec)	WP (m)	SS	QR cumec	QR cumec
RSC-1	0 to 502	0.024	40	0.3	0.10	0.20	1.10	0.58	1:1	0.01	0.04
LSC-2	0 to 185	0.018	20	0.3	0.05	0.20	1.45	0.41	0:1	0.015	0.02

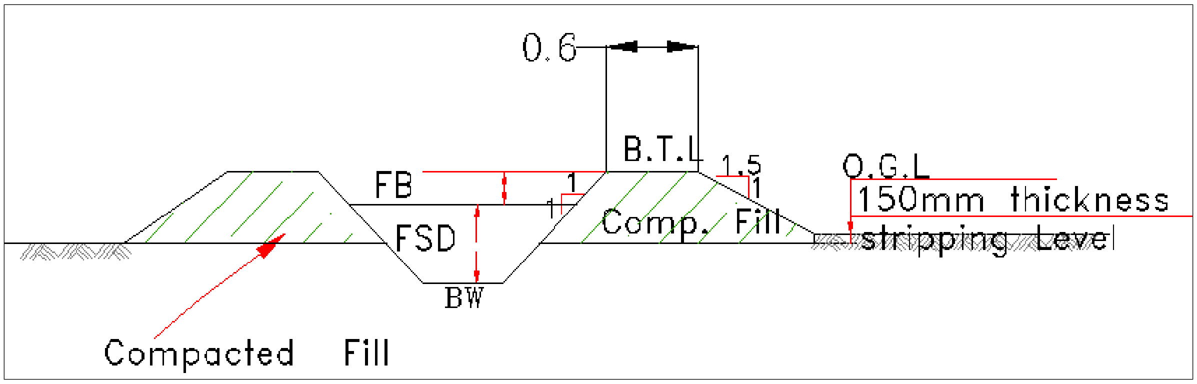

Figure 4-1: Typical Cross Section of secondary canals

4.4.3 Tertiary canals

In the layout system there are seven tertiary canals, the designed discharge is determined based on the duty of irrigation. The sections of the canals are determined by using manning's formula, and they are trapezoidal section. The hydraulic characteristics is Presented in Table below.

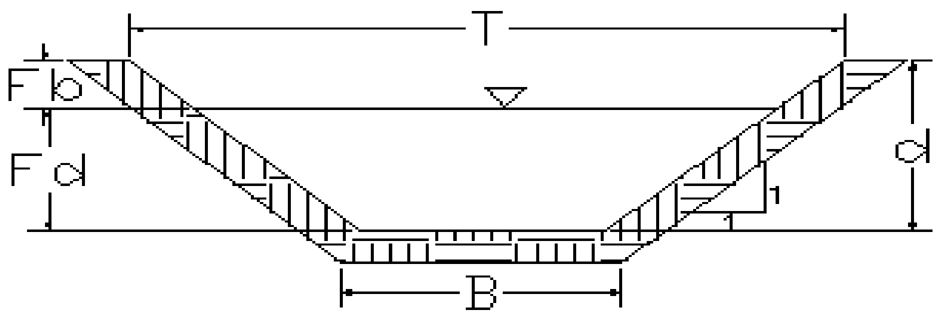

Figure 4-2: Typical Cross Section of tertiary canals

Table 4-4: Hydraulic Parameters of tertiary canals

Type of canal	Chain age (m)	Design discharge Q(m3/s)	Bed width (m)	Flow depth (m)	velocity (m/s)	Free board (m)	Total canal depth (m)	Top width (m)	Canal length (m)
RTC1	0+834	0.022	0.30	0.15	0.34	0.20	0.35	1.00	834
RTC2	0+201	0.015	0.30	0.10	0.40	0.20	0.30	0.90	201
RTC3	0+185	0.018	0.30	0.20	0.20	0.20	0.40	1.10	185
RTC4	0+181	0.011	0.30	0.10	0.30	0.20	0.30	0.90	181
RTC5	0+237	0.018	0.30	0.10	0.44	0.20	0.30	0.90	237
LTC1	0+132	0.026	0.30	0.10	0.64	20	0.30	0.90	86
LTC2	0+100	0.043	0.30	0.10	1.40	0.20	0.30	0.90	55

4.4.4 Field Canals

As shown in the layout, field canals run across the contours and hence face relatively gentle gradient. The discharge of most of the field canals is very small. Figure 15 below shows a typical field canal x-section. As much as possible field canals shall be made in fill in order to easily irrigate the adjacent command area. As can be seen from the layout, majority of the filed canals can be used to irrigate one sides of the command area.

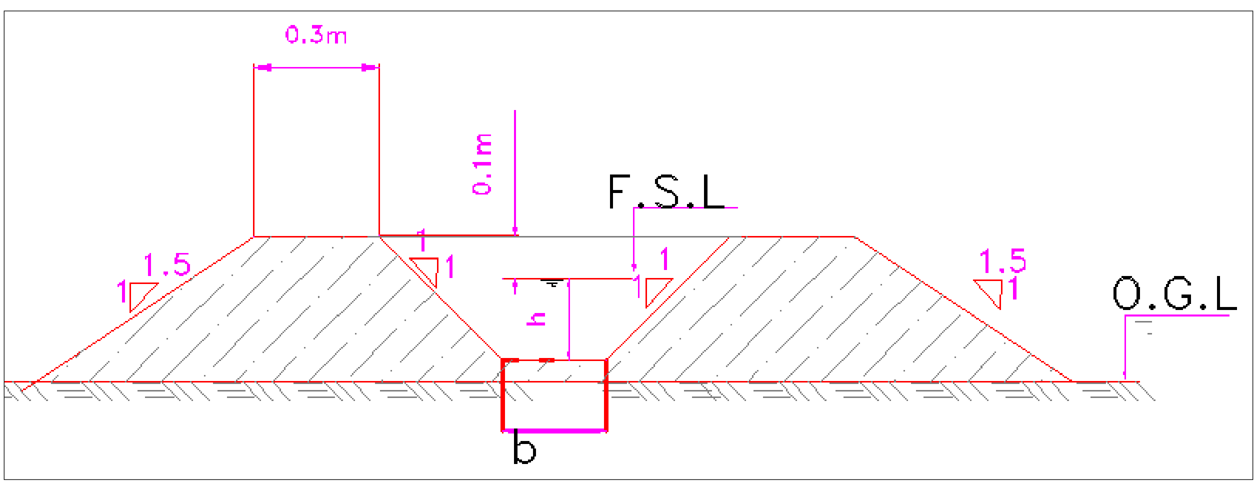

Figure 4-3: Typical field canal x-section

4.5 Canal Structures Design

4.5.1 Design of a typical flume

Hydraulic Characteristics of the canal

Length of the flume: 22m

Shape of the flume: Rectangular, Roughness coefficient, n =0.013

From the canal longitudinal profiles, u/s canal bed level (CBL) = 1546.90m

D/s Canal bed level (CBL) =1546.80m

U/s full supply level (FSL) = 1547.17m

D/s Full Supply Level = 1547.14m

Total head loss between the inlet & Outlet =0.3m

Table 4-5: Hydraulic Parameters of Flume

Flume on	**start Chainage (m)**	**Q(m/s)**	**V**	**L**	**H**	**T**
Flume 1	0+338	0.08	0.63	28	3.5	0.5
Flume 2	0+1300	0.2	0.65	15	2.5	0.5

4.5.2 Drop structure

This structure plays in energy dissipating in such a way that it reduces the problems encounter in the canal or prevents the canal from being eroded when the specific topography of land along the canal allows the usage of this structure.

For this particular project, vertical drop structure of U.S.B R type standard is selected to convey water safely along the canals without the cause of erosion within a canal. Using the recommended formula each dimension has been calculated in table below.

Design steps:-

A. Critical hydraulic

1. Design discharge, Q (m^3/s)

2. Height of drop, h(m)

3. Width of drop, bc = $\frac{0.734a}{d^{3/2}},(m)$

Where; d = water depth of the canal, m

4. Critical discharge, q = Q/bc

5. Critical depth, dc = $\left(\frac{q^2}{q}\right)^{1/3}$

B. Stilling basin

1. Basin width, B = $\frac{18.46\sqrt{Q}}{Q+991}, m$
2. Basin length, L= $2.5+\left[\frac{1.1dc}{h}+0.7\left(\frac{dc}{h}\right)^3\right]\sqrt{hdc}, m$
3. Lip height, a = dc/2, a ≥ 0.15

Table 4-6: Vertical Drop Parameters

the drop lays at	Chain age(m)	Discharge, Q (m^3/s)	Canal Bottom Width, b (m)	Height of drop, h (m)	Water Depth, d(m)	Free board, fb (m)	Width of Drop, wd (m)	Unit discharge, q (m^3/s/m)	Critical depth, dc(m)	Length of drop basin, L (m)	Lip height, a (m)	Width of drop basin, B (m)	Length of Protection, L_p (m)
RMC	0+82	0.07	0.50	1.2	0.27	0.20	0.50	0.21	0.16	1.17	0.3	0.49	1.4
	0+700	0.07	0.50	1.2	0.27	0.20	0.50	0.21	0.16	1.17	0.3	0.49	1.4
RSC1	0+125	0.04	0.30	0.9	0.10	0.20	0.30	0.09	0.10	1.5	0.23	0.42	1.00
	0+225	0.06	0.30	0.8	0.11	0.20	0.30	0.12	0.11	1.6	0.24	0.52	1.00
LSC1	0+25	0.02	0.30	0.9	0.05	0.20	0.30	0.07	0.08	1.4	0.24	0.30	1.0
RTC1	0+34	0.025	0.30	1.2	0.14	0.20	0.33	0.08	0.08	0.8	0.23	0.30	1.2
	0+112	0.025	0.30	0.9	0.14	0.20	0.30	0.08	0.08	0.80	0.18	0.33	1.0
	0+325	0.025	0.30	0.8	0.14	0.20	0.33	0.08	0.08	0.75	0.17	0.33	1.0

4.5.3 Design of Division box

At different points of the main and secondary canals division boxes are provided which regulates the head and facilitates the division of flow among the dividend canals. Gate should be provided at the outlet of the boxes. For detail refer the drawing. The hydraulic parameters are calculated and presented in table below

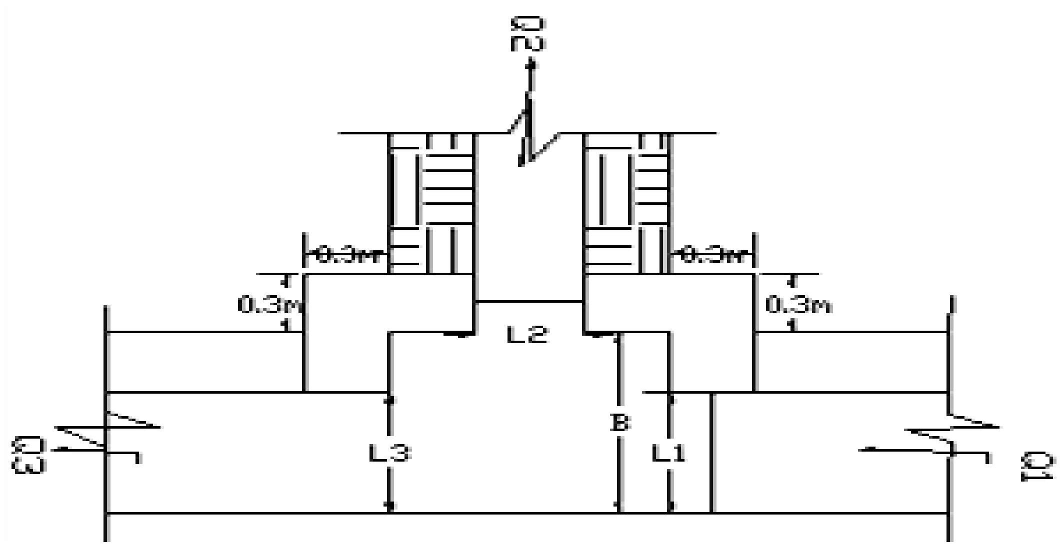

Figure 4-4: Typical Division Box plan

Using broad crested formula,

$$Q= CL\,(h)^{3/2}$$

Where; Q= discharge over rectangular weir/intake (opening), m^3/s

C = discharge coefficient, C= 1.7

L= effective length of crest form in m

h= over flow depth, m

Assuming equal discharge coefficient & sill height for two or three dividing canals, the proportion becomes.

$$Q_1/ Q_2= Q_2/ Q_3 =L_1/ L_2 = L_2/ L_3$$

Where Q_1= is flow in canal 1

Q_2 = is flow in canal 2

Q_3 = is flow in canal 3

L_1= is effective crest length of weir/intake sill across opening to canal 1

L_2= is effective crest length of weir/intake sill across opening to canal 2

L_3 = is effective crest length of weir/intake sill across opening to canal 3

$Q_1= CL_1\,(h)^{3/2}$,

$L_1 = Q_1/Ch^{3/2}$

$L_2 = L_1*Q_2/ Q_1$

$L_3 = L_1*Q_3/ Q_1$

The depth of (height of) the division box,

$D = d + f_b$

The width of the division box,

$B = b + 2*m*D$

Where b= base width of the incoming canal

D = total canal depth of the incoming canal

Table 4-7: Hydraulic parameters of Division Boxes

Chain age of Division Box	**Incoming flow (m^3/s)**	**Design Discharge to Sc (m^3/s)**	**Ongoing discharge (m^3/s)**	**MC bed width for ongoing discharge (m)**	**SC outlet width (m)**	**width of division box (m)**	**Depth of division box (m)**
RMC1 &RSC 1-1 (0+962)	0.069	0.050	0.040	0.50	0.30	1.0	0.45
LMC1&LSC1-1 (0+322)	0.02	0.012	0.006	0.30	0.30	0.50	0.35

4.5.4 Road crossing structure

One road crossing structures is provided on the left main canal, at the existing foot paths. The road crossing structures are rectangular reinforced concrete slab. The slab is reinforced with 12mm @150mmc/c the length of the slab is 1.m which is the same as the respective canal bed width, its width and thickness is 1.5m and 15mm respectively.

5. BILL OF QUANTITIES AND COST ESTIMATION

5.1 Bill of Quantity and Cost Estimation for General Item &Head Work

The quantities of the various items have been worked out as per the final design and final drawings prepared for the scheme. The unit rates analysis has been carried out based on the data available in the vicinity of the project area. The bill of quantities and cost estimation are presented in table below.

Table 5-1: Bill No. 1- General Items

Sr. Nr	Item of Work	Nr	Unit	Quantity	Cost (Birr)
1.1	Allow for mobilization	L.S	1	209,066.40	209,066.40
1.2	Allow for demobilization	L.S	1	188,159.76	188,159.76
1.3	Access road	L.S	2	170,146.73	340,293.45
1.4	Allow for Consultant's/contractor's camping facilities 4*5m2, Living room for contractors key personnel, CIS and internally painted clip wood wall, Masonry floor cement screened and well ventilated room complete with doors and windows.	L.S	1	450,000.00	450,000.00
1.5	Sign post/Indicator	L.S	1	9,225.90	9,225.90
1.6	Asbuil drawing	L.S	1	28,561.60	28,561.60
1.7	Dewatering of open trenches and excavation, temporary diversion of the river flow and pumps	L.S	1	175,000.00	175,000.00
	Total cost				**1,400,307.10**

Table 5-2: Bill No. 2- Head work

S/ N	Description	Unit	Quantity	Rate	Cost (Birr)
1	Weir body				
1.1	Earth Excavation	m3	319.8	43.17	13,804.71

1.2	Lean Concrete (C10)	m5	40.38	2,048.15	82,704.44
1.3	cyclopean concrete(C-20)	m3	582.4	2,466.35	1,436,401.97
1.4	Reinforced Concrete				0.00
1.5	Concrete (C20)	m3	403.78	2,171.82	876,937.34
1.6	Reinforcing bars Dia 14mm	Kg	2757.3	47	129,593.10
2	Both wing wall				0.00
2.1	Earth Excavation	M3	600.31	43.17	25,913.41
2.2	Back fill and compaction	M3	265.64	112.05	29,764.17
2.3	Lean concrete (C10)	M3	21	2,048.15	43,011.23
2.4	Masonry	M3	320.25	1,723.86	552,064.93
2.5	plastering with a 1:3 ratio mortar	M2	240	150.59	36,141.83
3	Divide wall				0.00
3.1	Lean concrete (C10)	M3	0.2	2,048.15	409.63
3.1	Masonry	M3	7.5	1,723.86	12,928.92

3.1	concrete c-20	M2	5.94	2,171.82	12,900.61
3.1	reinforcement bar14mmdia	kg	398.6	47	18,734.20
4	U/s cutoff				0.00
4.1	Earth Excavation	M3	166.4	43.17	7,182.94
4.2	Back fill and compaction		153.92	112.05	17,246.27
4.3	Concrete (C20)	m3	11.7	2,171.82	25,410.29
4.4	Reinforcing bars Dia 14mm	Kg	556.9	47	26,174.30
5	D/s cutoff				0.00
5.1	Earth Excavation	M3	270.4	43.17	11,672.28
5.2	Back fill and compaction		250.12	112.05	28,025.20
5.3	Concrete (C-20)	m3	11.7	2,171.82	25,410.29
5.4	Reinforcing bars Dia 14mm	Kg	796.9	47	37,454.30
6	under sluice gate material supply & instalation				0.00
6.1	Sheets metal 6mm thick	m2	1.98	400	792.00

6.2	Stiffening angle iron (30*30*4)	m	5	84	420.00
6.3	Angle iron for groove(40*40*4mm)	m	9.2	95	874.00
6.4	16mm reinforcement bar for handling	kg	2.31	73.52	169.83
6.5	Spindle for gate	LS	2	5000	10,000.00
7	head regulator Gate material supply & installation				0.00
7.1	Sheet metal 6mm thick	m^2	0.6	400	240.00
7.2	Stiffening angle iron (30*30*4)	m	4	84	336.00
7.3	Angle iron for groove(40*40*4)	m	7.2	95	684.00
7.4	16mm reinforcement bar for handling	kg	3.01	50	150.50
7.5	Concrete top floor wall C-20	m^3	0.42	2,171.82	912.16
8	Operation Slab (C-20 concrete)				0.00
8.1	C-20 concrete	m3	0.24	2,171.82	521.24
8.2	reinforcement bar14mmdia	Kg	33.34	47	1,566.98
9	Breast Wall (C-20 concrete)				0.00

9.1	C-20 concrete	m3	0.64	2,171.82	1,389.96
9.2	reinforcement bar14mmdia	Kg	82.67	47	3,885.49
	Total headwork cost				**3,471,828.53**

5.2 Bill of Quantity and Cost Estimation for Irrigation Infrastructure

Table 5-3: Bill No. 3- infrastructures

Sr.no	Item of work	Unite	Quantity	Unite price	Cost (birr)
1	Main canal				
1.2	Clearing & grubbing	m2	2,638.38	9.23	24,344.38
1.3	Excavation (ordinary soil)	m3	3,306.80	43.17	142,743.68
1.4	Compacted back fill with selected material	m3	230.24	112.05	25,797.70
1.5	Masonry work (with1:3 mortar)	m3	960.69	1,723.86	1,656,091.37
1.6	Plastering	m2	6,113.15	150.59	920,585.08
2	Secondary canals				-
2.1	Earth work				-
2.1.1	Clearing & grubbing	m2	686.88	9.23	6,337.85
2.1.2	Excavation (ordinary soil)	m3	612.17	43.17	26,425.37
2.1.3	Compacted fill with selected material	m3	6.13	112.05	686.85
2.2	Masonry work (with 1:3 mortar)	m3	88.57	1,723.86	152,681.94
2.3	Plastering	m3	147.64	150.59	22,233.25
5	Flume quantity				-

5.1	Excavation (ordinary soil)	m3	43.20	43.17	1,864.80
5.2	Lean concrete ,C-10	m2	32.00	2,048.15	65,540.92
5.3	Masonry work (with1:3 mortar)	m3	12.37	1,723.86	21,324.10
5.4	Plastering(with1:3 mortar)	m2	35.00	150.59	5,270.68
5.5	Gabion (standard)	m3	15.00	1,181.98	17,729.67
5.6	Concrete(C-20)	m3	15.62	2,171.82	33,923.82
5.7	Reinforcement bar, φ14	kg	253.28	150.59	38,141.68
6	Tertiary canals				-
6.1	clearing & grubbing	m2	1,461.08	9.23	13,481.41
6.2	Excavation (ordinary soil)	m3	1,549.21	43.17	66,874.30
6.3	Compacted fill	m3	92.05	112.05	10,313.93
7	Drop				-
7.1	Excavation (ordinary soil)	m3	14.90	43.17	643.18
7.2	Masonry work (with1:3 mortar)	m3	27.79	1,723.86	47,905.96
7.3	50mmConcrete(C-10) for basin	m3	0.34	2,048.15	696.37
7.4	Plastering	m2	47.60	150.59	7,168.13
7.5	Stone pitching	m3	11.69	1,300.00	15,197.00
8	Turnouts				-
8.1	Masonry work (with1:3 mortar)	m3	10.13	1,723.86	17,462.66
8.2	cemented stone pitching	m3	20.04	1,300.00	26,052.00
8.3	50mm thick C-10 concrete bedding	m3	2.43	2,048.15	4,977.01
8.4	plastering	m2	32.16	150.59	4,843.01
9	Division box				-
9.1	Excavation (ordinary soil)	m3	28.00	43.17	1,208.67
9.2	Masonry work (with1:3 mortar)	m3			

			26.40	1,723.86	45,509.80
9.3	Plastering	m2	52.80	150.59	7,951.20
9.4	Stone pitching	m3	20.04	1,300.00	26,052.00
9.5	Gate material supply& installation				-
9.5.1	4mm thick sheet metal	m2	6.60	278.50	1,838.10
9.5.2	40*40*4mm angle iron	m	158.91	111.75	17,758.30
9.5.3	Concrete(C-20)	m3	110.76	2,171.82	240,550.74
9.5.3	ɸ14mm bar for handling	kg	20.27	47.00	952.69
10	Canal crossing @ MC (culvert)				
10.1	C-20 Concrete for slab 15mm thick	m3	1.44	2,171.82	3,127.42
10.2	Reinforcement bar 12mm	kg	34.34	47.00	1,613.98
	Total				**3,723,901.03**

6. CONCLUSSIONS AND RECOMMENDATION

- Burka irrigation project is upgrading traditional irrigation practice. The existing irrigation practice has more or less two basic problems. The route, along which main canal is aligned, is made of alluvial deposit as a result there is much water losses. Solving these problems is very essential for proper utilization of water & soil which in turn can improve the livelihood of peasants of the project area. That is why Burka irrigation project is being formulated.
- The head work structure of this project consists of retaining wall, d/s and u/s left - right bank protection works providing same scour depth along with canal, breast wall, & gate. The designs of each of these structures with their working drawings have been executed.
- Though the banks & bed of the river are designed to make them stable, continues removal of silt (may be annual) from the headwork on the entry line of water to the intake outlet has to be done by project beneficiary.
- The infrastructure of this project area is designed to irrigate about 80 ha of land by taking its supply from the Burka diversion weir irrigation project. The maximum duty of the command area for 12 hours per day irrigation with overall project efficiency of 46%. The method of irrigation of the project area is furrow surface irrigation in which the main and tertiary canals are working continuously where as the field canals within a tertiary block are working rotational system.
- As the dominant soil type is clay soil, the main canal system is designed to be masonry.
- The reason why the main canal is to be lined up to the end is to avoid the siltation problem, time saving to reach at the tail part, reduce maintenance cost.
- On the right secondary unit of the irrigation systems, some are associated with chute. They are designed as far as possible to be partially filled and cut. The layout is designed as far as possible to avoid cross-structures within them.
- The design of the canal dimensions of the irrigation canal is done by applying the manning's uniform flow equation. The variable of the hydraulic parameters are calculated using iteration or flow master program.
- As soils of the command area are predominantly clay textured; and hence water and soil management measures should be undertaken; and optimum moisture content should be maintained to improve workability of the soil during land preparation and planting time.

The following recommendations are drawn:

1. For better performance and long service year of the project regular inspection and maintenance is highly required.
2. Farmers training, how to operate and maintain the project structures as a whole and available and water resources has a paramount important.
3. The irrigation hours per day and per week should be flexible based on base flow amount of each week or month.

Close supervision of the construction should be made to modify (if needed) each Components of irrigation system based on specific site conditions.

7. REFFERENCES

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