THE NATIONAL REGIONAL STATE OF OROMIA
Wangur SMALL SCALE IRRIGATION PROJECT

IRRIGATION & DRAINAGE SYSTEM DESIGN DRAFT FINAL REPORT

CLIENT

OROMIA BUREAU OF AGRICULTURE AND NATURAL RESOURCES
IRRIGATION DEVELOPMENT BRANCH
P. O. Box
Tel +251
Email:
Finfinne/Addis Ababa, Ethiopia

CONSULTANT

Oromia Water Works Design Supervision Enterprise
P.O.Box. 870/1250
Tel +251 11 4392162/2470
Email: owwdse@telecom.net.et
Finfinne/Addis Ababa, Ethiopia

Nov, 2019
Finfinne

Table of Content

LIST OF FIGURES II

LIST OF TABLES II

CHAPTER 1 : INTRODUCTION 3

1.1 GENERAL 3
1.2 BACKGROUND TO THE PROJECT 3
1.3 PROJECT LOCATION AND ACCESSIBILITY 3
1.4 OBJECTIVE AND ORGANIZATION OF THE STUDY 4
1.5 SCOPE OF THE STUDY 4

CHAPTER 2 : PROJECT LAYOUT 6

2.1 GENERAL 6
2.2 TOPOGRAPHIC MAP AND THE LAND FEATURES. 6
2.3. GEOLOGY OF THE AREA 7

CHAPTER 3 : HEAD WORK DESIGN 8

3.1 GENERAL 8
3.2 HYDRAULIC DESIGN OF THE WEIR 9
3.2.1 HEADWORK SITE SELECTION 9
3.2.2 TYPE AND SHAPE OF WEIR 9
3.2.3 WEIR GEOMETRIC PARAMETERS 10
A) DETERMINATION OF WEIR HEIGHT..10
BARRAGE WIDTH: 11
REGIME OR SCOUR DEPTH 11
B) WEIR CREST LENGTH 12
C) TOP AND BOTTOM WIDTH OF THE WEIR...12
D) U/S & D/S SLOPES 13
3.2.4 HYDRAULIC ANALYSIS 13
3.2.5 HYDRAULIC JUMP 15
3.2.6 TYPE OF FLOW 16
3.2.7 DETERMINATION OF CUT-OFF DEPTHS AND ITS THICKNESS..........................16
3.2.8 HIGH FLOOD LEVEL COMPUTATION 18

CHAPTER 4 :DESIGN OF IRRIGATION CANAL 26

4.1 GENERAL 26
4.2 IRRIGATION WATER SOURCE 26
4.3 METHODOLOGY...26
4.4 CHOOSING OF IRRIGATION METHODS ....**ERROR! BOOKMARK NOT DEFINED.**
4.5 IRRIGATION SYSTEM LAYOUT ....**ERROR! BOOKMARK NOT DEFINED.**
4.6 COMMAND AREA..27
4.7 HYDRAULIC DESIGN OF CANALS..27
4.7.1 MAIN CANAL 27

CHAPTER 5 : ENGINEERING WORKS & COST **ESTIMATE 29**

LIST OF FIGURES

FIGURE 1-3: DIVERSION SITE OF WANGUR SSI PROJECT 4

FIGURE 3-1: STAGE DISCHARGE RELATION (RATING CURVE) AT HEADWORK SITE 14

LIST OF TABLES

TABLE 1:DESIGN DATA FOR THE WEIR 12

TABLE 2:HYDRAULIC JUMP CALCULATION ....**ERROR! BOOKMARK NOT DEFINED.**

TABLE 3:THE HYDRAULIC PARAMETERS FOR CANALS WERE FIXED IN SIMILAR WAY AND TABULATED ...28

TABLE 4: SUMMARY OF ESTIMATED ENGINEERING COST OF PROJECT 29

TABLE 5: SUMMARY OF ESTIMATED ENGINEERING COST OF PROJECT 31

Chapter 1 : INTRODUCTION

1.1 General

Agriculture is the major income sector which plays a decisive role in the economy of Ethiopia. However, the poor performance of the sector and insufficient and erratic rainfall condition of the country exacerbated by the rapid population growth (the rapidly increasing demand for food), result in problem of food security. The major factors behind the poor performance of subsistence farming of the Ethiopian Agriculture are: diminishing farm size, severe soil degradation, and inadequate and variable rainfall. Although available reports suggest that insufficient rainfall combined with variability in the onset and duration of rain is an important reason for low production, there are a number of other factors that needs close examination like; soil erosion or degradation, poor design and construction technology, lack of essential inputs, low prices and lack of incentives, poor marketing, weak agricultural support services and overstocking. Unless remedial measures are taken, the standard of living of the farmers will further decline, especially in view of the current rapid population growth rate per year.

The best strategies to overcome such problems are promotion of conservation-based farming systems; expansion of irrigated farm land; and improving the irrigation management practice which are achieved through properly design and construction of irrigation system.

In this regard, the client has proposed the Feasibility Study & Detail Design of Wangur Small Scale Irrigation projects. The implementation of the irrigation project will improve the living condition of the target kebeles within the vicinity of the project.

1.2 Background to the Project

This Draft Feasibility Study and Detail Design Report, as required in the consultancy agreement, contains a description of start-up activities, work plan for various discipline, approach and methodologies for performing the assignment, and an estimate of the time allocation for the study and feasibility study of the scheme. The overall objective of the feasibility study report is to convey the consultant's perception of the project and the approaches that the consultant would be taking in actually carrying out the project that would serve as a means of establishing common understanding of the project between the Client and the Consultant.

1.3 Project Location and Accessibility

The project area is located in Doba Woreda of West Haraghe Zone of Oromia National Regional State which is found in the eastern part of the country some 410 km from the center. Access to the project area is possible using the 370 km Finfinne-Hirna highway and using gravel road that turns left after 3.6 kilometers from Hirna, passes through Doba and runs towards east to reach the site covering a total of about 40 kilometers.

Figure 1-1: Diversion site of Wangur SSI project

1.4 Objective and Organization of the Study

The objective of the study is to undertake feasibility study and detailed designs of headwork for Wangur Small Scale Irrigation projects with main target to enhance the utilization of surface water resources to improve the livelihood of the user communities. The aim of enhancing the implementation of these irrigation schemes is to increase agricultural production per unit area and thus improve the living standard of the local poor farmers and bring about better socio-economic conditions and ultimately achieve food security.

The work consists of furnishing engineering services complete in all respects including all field and office works in accordance with standard of engineering profession and applicable guidelines and requirements. The study approach and methodology will incorporate attitudes preferences, skills and previous experiences of the local farmers in the course of the studies and designs through active and full participation of the end users of the schemes during all process.

1.5 Scope of the study

The main scope of the study includes:

- Carrying out detail design, for the headwork components and its pertinent structures;

- Carrying out detail design of hydraulic structures;
- Preparing Engineering Bill of Quantities, Cost estimates including break down of unit prices;
- Preparing Detail Design Reports
- Preparing detail drawing of each structures that need to be constructed while implementing the project;

Chapter 2 : PROJECT LAYOUT

2.1 General

The principal approach adopted for the preparation of the project layout has been bringing the maximum area possible under irrigation. In line with this, the maximum head at the Main Canal intake (maximum operating level) was allowed to command all potential areas by the gravity contour Main Canal. This arrangement will enable more lands identified as suitable for irrigation to be included in the command area. The preparation of the project layout has considered the following but not limited to:

2.2 Topographic map and the land features.

The topography and the land features identified during the field survey are important factors for layout preparation. These features, shown in Figure below which includes, land slope, shape, and brokenness, rugged areas, are the general landscape considered in the preparation of the layout of the command area.

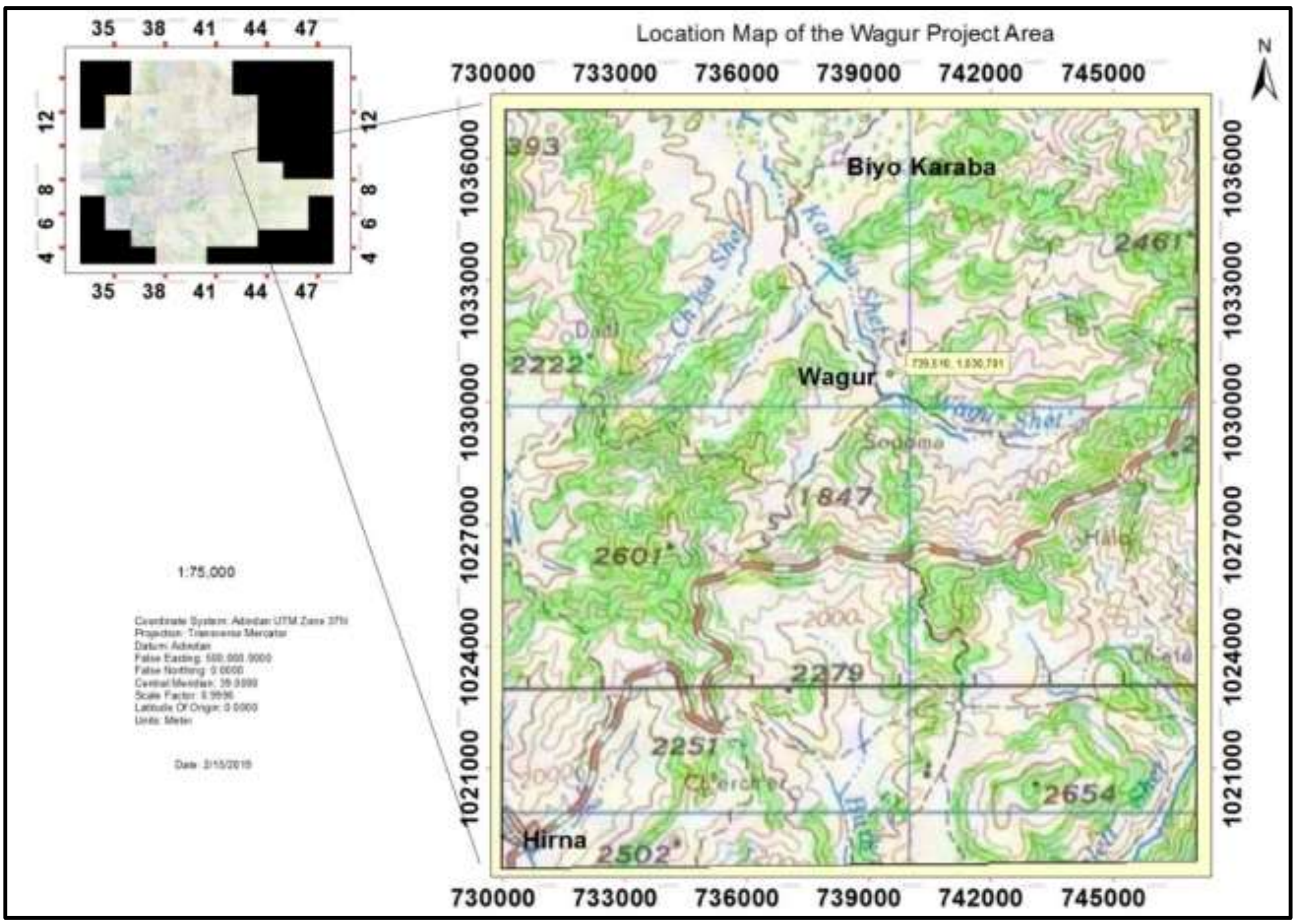


Figure 2.1 presents the topographic map of the project area.

2.3. Geology of the area

These are “Presumptive bearing capacity values based on experience with other structures already built. As presumptive values are based only on visual classification of surface soils/rocks their values can be improved by improving important factors affecting the bearing capacity such as the shape, width and depth of footing. That is widening the longitudinal section, deepening of cut off trench and utilizing structure shape that can transfer live load during flood into the abutments.

Natural construction materials for all purposes are found in very close range to all the structural sites. The limestone in the area is very suitable for masonry stone due to its horizontal bedding that made it of easy workability. The basalt of the Alajae formation can serve as aggregate source through manual or mechanical crushing. The only material, that needs to be transported from Miesso that is about 110 kilometers from the site is sand for mortar preparation. Another option of sand source is the Diredawa sand which is of better quality since it derived from rocks of high quartzo-feldspatic composition.

Chapter 3 : HEAD WORK DESIGN

3.1 General

Abstraction of water from the source, mostly rivers is possible either by using diversion or storage mechanisms. In planning any irrigation project, two major criteria need to be well assessed in relation to the water source. These are availability of sufficient water and or sufficiency of deriving head to distribute water to the command area. If the available water source in the river is not sufficient enough to satisfy the water demand of the irrigation water, storage structure is proposed. But if there is sufficient amount of water from the source, most commonly there is limitation of head to derive the water from point of abstraction to the distribution area on field. In such situation a diversion structure is recommended to give head to the source water.

A diversion structure is mainly designed to raise the water level to a certain height. The components and hydraulic and structural design of the headwork structure is very crucial. The safety of the diversion structure depends on the magnitude of peak flood expected in the river and the strength of the river bed material. In the subsequent sections, the design procedure of a diversion structure for Wangur small scale irrigation project are presented.

Flood damage and silt deposition have aggravated recently owing to artificial structures in high flow rare areas such as small rivers, which can lead to secondary damage. In this regard, studies are required to examine the conventional design criteria formulas to secure the stability of structures such as barrage. Although studies on the stability of these structures have been conducted through small-scale experiments, few empirical studies have investigated the hydraulic phenomena occurring near actual artificial structures. In this study, we fabricated real-size models of barrage at the spring and investigated the flow patterns around the structures by applying the particle image velocimetry analysis technique with a flow tracker. Measuring the scour length in the waterspout section when the structures are overflowing, and compared it with the values calculated using the formula. Consequently, as the supply flow increases, the result is different from the value calculated using the formula given in the existing design standard, and it is judged to be inappropriate for a small stream area with high flow rate. Thus, it is necessary to consider the design factors such as energy gradient and the flow amount per unit width into weir and drop structure as well as the existing design factors in designing an apron section for a barrage.

3.2 Hydraulic Design of the Weir

3.2.1 Headwork Site Selection

Following field investigation: topographic survey, and geotechnical / geological studies investigation two headwork sites have been selected, the first headwork site (pervious headwork which was damaged) alternative was proposed by the client and the second alternative is proposed by the consultant upstream of the first site, on the same stream.

The general factors and the particular local situations need to be considered in selecting a headwork site. Initially two headwork sites were selected and compared through different factors. The first headwork was located just at downstream where very narrow and rocky river channel. It was expected that headwork was damaged due to high pressure from narrow to wide channel where previous headwork located. The second alternative weir site of the project which is located just upstream of very narrow river channel has been selected once a detail physical survey and assessment carried out mainly based on the following basic considerations:

- River reaches preferably with straight, wide and well-defined channel.
- With respect to the adjoining land surface, the elevation of water surface upstream of the weir should not be so low as to require an excessively high weir to divert the water at the intake;
- The geological formation of the river cross section at the barrage axis and Width and shape of the river at selected headwork site
- Easy arrangement of flow diversion during construction; and
- Availability of construction material at the nearest place.
- Economic consideration (width of river at the barrage axis, required length of main canal and appurtenant structures, required protection works).

Accordingly, the barrage axis has been fixed at relatively stable reach of the stream with geographical coordinate of 739509.24m Easting and 1030706.07m Northing. The selected headwork site as compared to most of its sections has a narrower reach. In addition, the intake canal can be laid and taken easily and safely away from the river reach.

3.2.2 Type and Shape of barrage

Barrage type selection mainly based on the availability of construction materials where a barrage body and its accessory structures can be constructed with no material deficit, local skills to manage the overall construction work more easily and economy of the proposed barrage. Considering the economical design, availability of construction materials, easy construction, nature of the foundation and stream bank material, a sharp Crested barrage type has been selected.

As far as possible, it should be aligned at right angle to the direction of the main river current. This ensures lesser length of the barrage, better discharging capacity and lesser cost. This right-angled alignment is better and, therefore, common, especially when the river bed is silty or sandy.

The height of the barrage required to irrigate the higher elevated command area is *0.5m*. Besides the bed of the river d/s is composed of a rocky which is liable to scouring if energetic flow occurs (the 50 years return period design discharge is about *81.43* m^3/s). Therefore, a barrage that can dissipate the energy of water falling from this height needs to have better energy dissipation efficiency.

The proposed barrage is to be constructed by cyclopean concrete with reinforced concrete capping. The cyclopean concrete consists of 60% C-25 concrete and 40% graded stone of size less than 10cm diameter, with external cover of single reinforced C-25 concrete.

3.2.3 Weir Geometric Parameters

a) Determination of barrage Height

The following criteria will be considered to fix barrage crest elevation:

The site must have a good command over the area to be irrigated and also must not be at too far distance to avoid long feeder channels.

- The width of the river at the site should be preferably minimum with a well-defined and stable river approach.
- A good land approach to the site will reduce expenses of the transportation and the ultimate cost of the project.
- There must be easy diversion of the river after construction
- Existence of central approach of the river to the barrage after diversion, this is essential for proper silt control.
- If it is intended to convert the existing inundation canals into the perennial canals, site selection is limited by the position of the head-regulator and the alignment of the existing in-undation canals.
- A rock foundation is the best but in the alluvial planes the bed is invariably sandy. The common practice in the world has been to build the barrage on dry land in a bye river and after completion to divert the river through it.

- This gave an oblique approach and created many problems. The following guidelines have now been proposed by the Irrigation Research Institute, Lahore. Their recommendations are based on extensive hydraulic model experiments for each individual case.
- If the river axis is to the right of the headwork axis, the concentration of flow is generally on the left side with the consequent tendency to form an island on the right and vice versa.
- When a barrage is located below the confluence of two rivers, it should be located sufficiently far below the confluence and consideration must be given as to which of the rivers dominate the confluence.
- The barrage should be located as far as possible in the centre of the flood plain. Asymmetry of location increases the likelihood shoal forming and calls for expensive training works.
- The most suitable site for a barrage when constructed on dry land, is below the outer side of the convex bund which is followed by the straight reach of the river.
- The main canals at the head reach should not be too deep in order to avoid large excavation work

Barrage width:

River in alluvial plains while in flood spread over miles in width and in dry weather flow in channels. For optimum width Lacey's Equation, related to wetted perimeter to discharge wetted perimeter in case of shallow channel is almost equal to the bed width of the channel. The barrage width must be sufficient to pass the design the flood safety. The present trend is to design barrage for a 50-100 years' frequency flood. The minimum stable width of an alluvial channel is given by Regime Eq.

Regime or Scour Depth

Due to high flow, the river bed is scoured both on the upstream and downstream sides of the weir, large scour holes develop progressively adjacent to the concrete aprons, the weir foundations may slip into these scour holes, thus undermining the weir structure. The regime scour depth Rs may be estimated by following formula.

If actual waterway provided is greater or equal to the regime width and

If waterway provided is less than regime width and f = 1.75 under root d
d is mean diameter of bed material in mm.

The desired objective of a diversion is to deliver the required water to the maximum possible point in the irrigated field. Hence, the height of the barrage will be fixed in relation to irrigated field level, expected losses and head on the main connivance canal as shown below:

Table 1: Design data for the weir

Average Level of The Highest Field in The Command Area	**1651**
Average River Bed Level (RBD), m	**1650**
Head loss in main Canal, m	0.2
Head loss in canal head regulator, m	0.15
Off take canal invert level, m	1650.5
Main canal water depth, m	0.40
Driving head required, m	0.20
Crest Level, m	1650.5
barrage Crest Height, m	0.5
Crest Height adopted, m ~	**0.5**
Crest Elevation adopted, m ~	**1650.5**

Weir Crest Length

From the Lacey's regime width, the crest length of the over flow weir section can be determined as follows:

From Lacey's formula

$$P = 4.75\sqrt{Q} = 4.75\sqrt{(81.43)} = 42.86m \ ,$$

For rivers flowing within well designed banks a water way width of 7 m is acceptable:

Stream morphology has been evaluated with respect to hydraulic stability conditions and accordingly the weir axis has been fixed. Considering the actual site natural feature conditions of the river banks, and average width of the river channel, hydraulic stable stretch identification, and the crest length of the weir is considered as 7.0m

a) Top and Bottom width of the Weir

According to the Bligh's formula top and bottom width of the weir body is determined using the following empirical equations:

Top width, BT = 2/3*He

Where P - Height of the weir

He - Specific Energy head (Overflow depth +approach velocity head)

ρ - Specific Unit weight of the Weir Body (Cyclopean Concrete)

He - is estimated using Broad Crested Weir formula

$$\mathbf{Q_T = 1.84\ L\ He^{3/2}}$$

Where L - weir crest length = 7m

Q - Peak design discharge = $81.43 m^3/s$

He = $(Q/1.84 * L)^{2/3}$ = 3.41m

Top width, BT = 2/3*He=2.79, 1m was taken

b) U/S & D/S Slopes

Adopt downstream side slope of 1H:2V and 1:1 upstream face is recommended.

3.2.4 Hydraulic Analysis

Discharge over weir

From the surveying data collected during field work, the width of the river (waterway) was estimated to be 7m. It was already determined in hydrological investigation that the peak design flood is 81.43 m^3/sec. each bay is 1.6 and pier is 1.1m. However, to be on the safe side mainly due to the stochastic nature of the hydrological process, the total discharge considered to be accommodated by a weir (overflow).

The depth water on of the crest of the weir is computed using:

$Q = CLH_e^{3/2}$, => He= $\{Q / (C.L)\}^{2/3}$

Where: -

Q = Peak Discharge (m^3/sec) = 81.43m3/s

C = Discharge Coefficient for broad crested Weir and sharp crested Weir (C = 1.71 & 1.84)

t=2/3*He.

He = Total Energy Head (m)

L = Crest Length (m) = 7m

H_e = 3.41 m

Velocity of approach, $V_a = Q/A = \{Q/[L ((h-P)+Hd)]\}$,

Where h = Weir Height = 0.5m

For Maximum Flood (50 Year Return Period)

(i)	U/S HFL = HFL before construction + Afflux	=	1655	m
(ii)	D/S HFL for 50 years retun period	=	1654.5	m
(iii)	Average discharge intensity = 81.43 / 7.10	=	16.96	cumec/m
(iv)	Scour depth R = 1.35 (q2/f)1/3	=	8.26	m
(v)	Velocity of approach Va = q / depth of water	=	2.05	m/sec
(vi)	Velocity head = Va2 / 2g	=	0.214	m
(vii)	U/S TEL = U/S HFL + Velocity head	=	1655.214	m
(viii)	D/S TEL = D/S HFL + Velocity head	=	1654.714	m
(x)	Head over the weir crest	=	4.5	m

Tail Water Depth

The water profile at downstream of the weir is required to carry out the stability analysis of the weir as well as to design the d/s wing wall and other river bank protection.

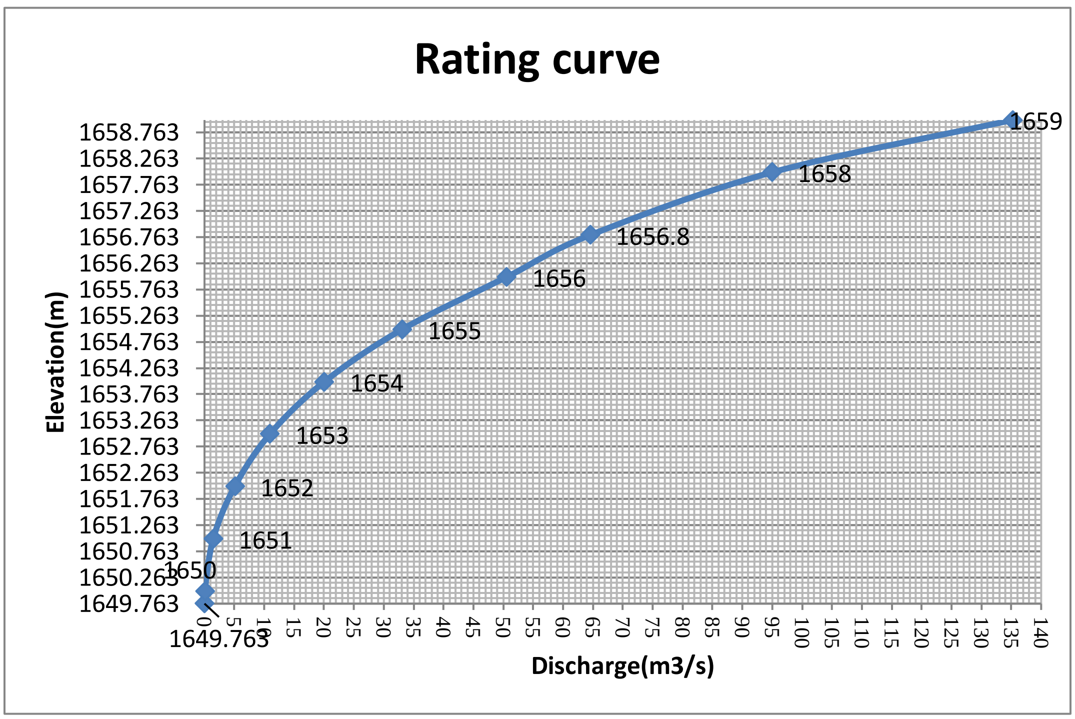


Figure 3-1: Stage Discharge Relation (Rating Curve) at headwork Site

From the stage discharge curve, tail water elevation equivalent to the flood discharge of 81.43m^3/s is found to be 1657.3m and tail water depth equal to 7.3m.

3.2.5 Hydraulic Jump

Hydraulic jump is formed to dissipate energy on D/S glacis. A suction pressure occurs on the floor due to hydraulic jump. For the determination of the suction pressure, the location of hydraulic jump and the profile of water surface on the U/S and D/S of the point at which the jump is formed is required.

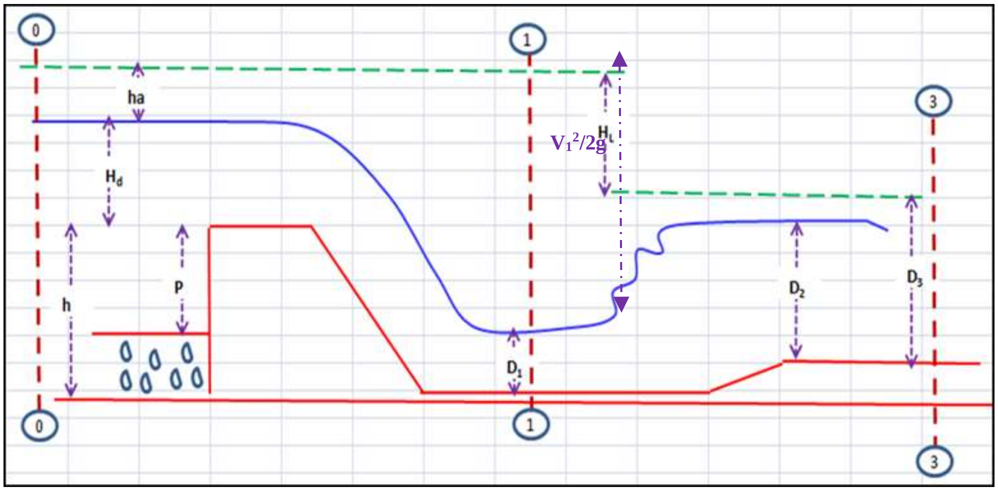


Figure 3 2: Flow profile sketch between section 0-0 and 1-1

As the downstream flow condition is turbulent and aggressive, to fix the corresponding dimensions of stilling basin and pervious protection, the position of jump have to be known. The position of jump on horizontal surface can be determined by applying Bernoulli’s equation between the weir crest and weir toe applied for section o-o and 1-1.

Pre-jump depth determination, $\mathbf{D_1}$

By equalizing the energy between upstream and downstream of the weir, the pre jump depth is determined as follows:

$Zo + h + Hd + ha = Z1 + D_1 + V_1^2/2g$

$h + Hd + ha = D_1 + V_1^2/2g$

$V_1 = q / D_1$, $q = Q/L$

And the critical depth dc is equal to:

$dc = (q^2/g)^{1/3} =$

$D_1 < dc$, implies that the flow takes place from super critical to sub critical flow

The post /Sequent jump depth determination, $\mathbf{D_2}$

The post jump depth or conjugated depth, D_2, is calculated from:

$V_1 = q/D_1$

$Fr = V_1/\sqrt{g}D_1$

$D_2 = (D_1/2)\,[(1+8Fr^2)^{1/2}-1]$

$$H_L = \frac{(D_2 - D_1)^3}{4D_2D_1}$$

The difference between d_2 and d_3 must be zero for best hydraulic condition. However, in general case the difference can be increased up to the range of 0.2m to 0.4m. In case of Hursa Weir, since the difference is out of the ranges or limits, the downstream basin shall be again increased by certain values till it comes to reasonable ranges. Thus, it is better to increase the stilling basin level by 0.5 m.

The jump length is found from L/D_2 versus Froude number graph in USBR 1987 design of small dams. Also, from Ministry of Water, Design Guide line of Diversion, September 2001 recommends:

Fr	2	3	4.5	5
L/D_2	4.3	5.3	5.8	6

3.2.6 Type of Flow

The ratio H_2/H_1 determine the type flow, the flow is free (modular) or Submerged (non-modular). If the ratio $H_2/H_1 < 0.75$, where H_1 and H_2 are the u/s and d/s head above the weir crest, the flow is modular, i.e. not affected by submergence. Otherwise ($H_2/H_1 > 0.75$), the flow is Submerged (non-modular).

3.2.7 Determination of Cut-off Depths and Its Thickness

Properly designed stilling basin dissipates the great majority of the turbulent energy in the flow. At the outflow from the basin there remains a certain proportion of energy in the flow that scours the downstream of the basin, bed and banks.

The depth of scour can be calculated using Lacey's equation.

Hydraulic mean depth(R), $R = 1.35 \times \left(\frac{q^2}{f}\right)^{1/3}$

Where: R = hydraulic mean depth

q = discharge per meter length = 48.6 $m^3/s/m$

Taking ds = 0.51mm particle size in mm for medium sandy soil type, the corresponding Lacey's silt factor, f = 1.76√ ds = 1.26 (See Engineering Geology and Geotechnical Report).

$$R = 1.35 \times \left(\frac{q^2}{f} \right)^{1/3} = 3.47$$

Cutoff walls are installed to prevent the piping under the structure, and to limit the intensity of the uplift so that the stability of the structure will not be threatened.

Upstream Cutoff Depth

Highest flood level at the upstream = 1657.95m

River bed level =1650.0m

Required depth of scour below HFL at upstream = 1.5 R

Upstream cutoff level = U/S UFL - (1.5 × R) = 1647.5masl

U/S depth of Cutoff = 1.5R-(h +Hd) = 2.5m

Minimum depth of U/S cutoff below bed level = $y_d/3 + 0.5$,

(where y_d is water depth in m at U/S HFL – River Bed Level = 3.471m)

Therefore, Minimum depth of U/S cutoff below bed level = $y_d/3 + 0.5$

But, provide average depth of U/S cutoff wall of = 2.5m

Provide U/S Cutoff Wall line down to elevation = 1647.5m

Downstream cutoff depth

Highest flood level at the Downstream = 1657.3m

Required depth of scour below HFL at Downstream = 1.75 R

Downstream cutoff level = D/S HFL- (1.75 × R) = 1646.5m

D/S depth of Cutoff = 1.75R-TWD.

Minimum depth of D/S cutoff below bed level = $y_d/2 + 0.5$,

(where y_d is water depth in m at D/S HFL – River Bed Level)

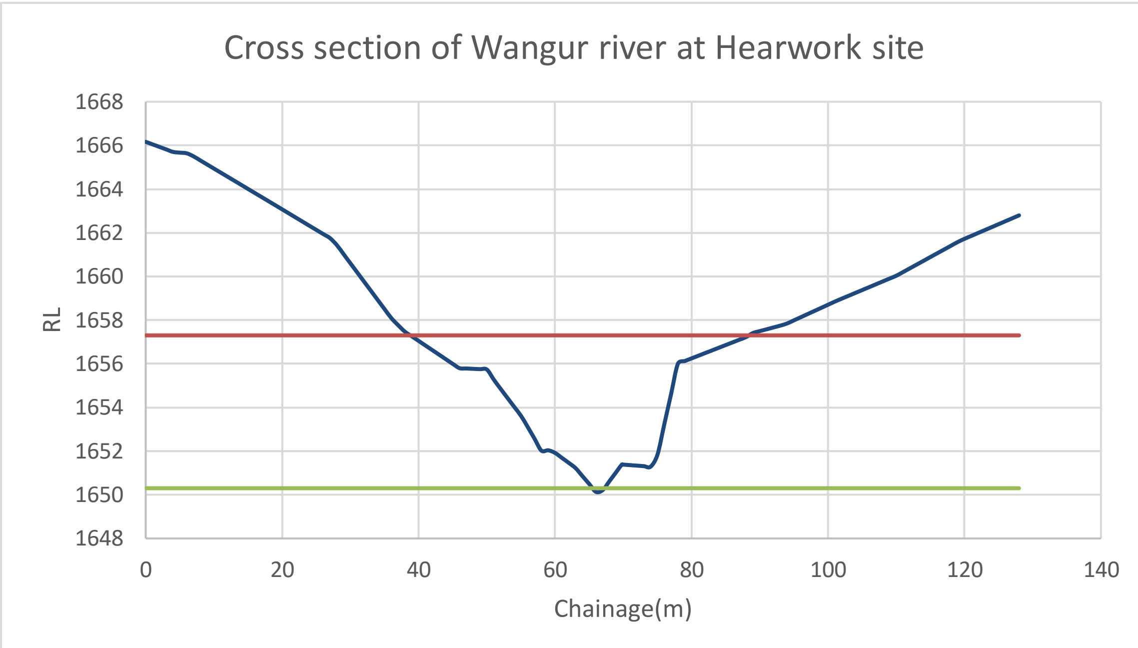
Therefore, Minimum depth of U/S cutoff below bed level = $y_d/2 + 0.5$

But, provide average depth of U/S cutoff wall of = **2.5m**

Provide D/S Cutoff Wall line down to elevation = **1646.5m**

3.5.0m height and 0.5m thickness of downstream cutoff is provided. But if uniform layer happened before the recommended depth, we recommended to anchor strongly the structure to the foundation.

3.2.8 High Flood Level Computation



Cistern Elevation

Downstream Tail water elevation - D2	=	1649.14	m
The cistern level is increased by 0 % of D2	=	0	m
Cistern level as per calculations	=	1649.15	m
Provided depth of cistern	=	0.85	m
Provided cistern level	=	1649.15	m

Since d/s river bed level is 1650m, cistern elevation depressed by 0.85 m, becomes 1649.15 m

Provide cistern length = 5.00

∆ Provided 5.00 m length of cistern, to be joined with D/S bed width an inverse slope of 2:1.

∆ D/S Floor length = Cistern length + Inverse slope + Hor. Floor (0.5m) = **5.50**

Vertical Cut-offs

The minimum depth of U/S & D/S Cut-off shall be provided below deepest foundation level.

(i) Provide U/S Cut-off of 2.50 m depth below U/S bed level = 2.5

Bottom elevation of = 1647.8

	upstream pile		
(ii) Provide D/S cut-off of 3.50 m depth for safe exit gradient		=	3.5
	Bottom elevation of downstream pile	=	1645.9

The Total length will be provided as below: -

(i)	D/S Horizontal floor length	=	0.5	m
(ii)	D/S Inverse slope (2:1)	=	1.7	m
(iii)	Cistern length	=	2.4	m
(iv)	D/S Glacis length (1:1 slope)	=	0.85	m
(v)	U/S floor length	=	2.7	m
(vi)	Total length of floor Provided	=	8.15	m

Total Floor Length and Exit Gradient

From Khosla's theory, the exit gradient can be expressed by the relation as

Exit gradient, GE =

1 / p x sqrt (l) X (H / d)

Where, GE

d = Depth of d/s cutoff

H = Maximum static head = Pool Level - D/S bed level

Substituting the values

1 / 6 = 1 / p x sqrt (l) X (H / d)

or

1 / p x sqrt (l) = d / (5 x H)

From Khosla curve for 1 / p x sqrt (l)

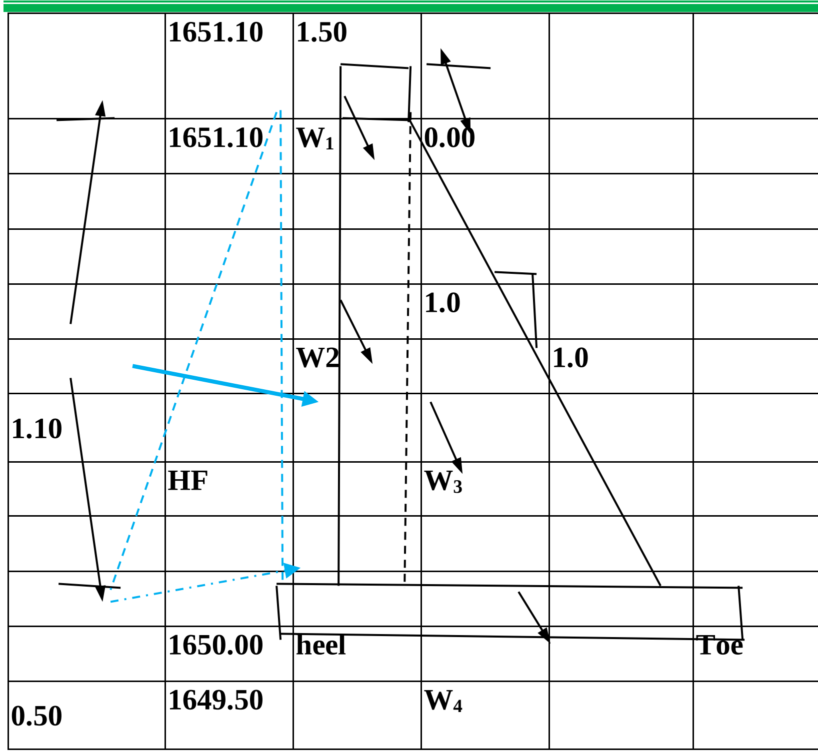
a

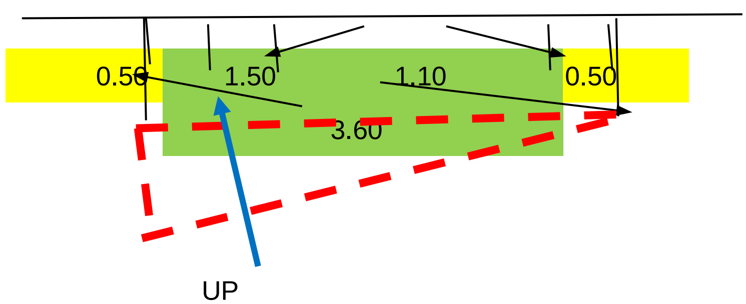
Floor length, b = a x d

Length Correction

Minimum total floor length

STABILITY ANALYSIS OF WEIR


1651.10
1.50
1651.10
W1
0.00
1.0
W2
1.0
1.10
HF
W3
1650.00
heel
Toe
0.50
1649.50
W4


0.50
1.50
1.10
0.50
3.60
UP

FORCES DUE TO WEIGHT

SN	Item	Calculations	Force (tonnes)		Lever arm	Moments (t-m) about the Heel	
			Vertical	Horizontal	(m)	(+ ive)	(- ive)
1	W_1	1.50 x 0.00 X 2.40	0.000		1.250	0.000	
2	W_2	1.10 x 1.50 X 2.40	3.960		1.250	4.950	
3	W_3	1/2 x 1.10 x 1.10 X 2.40	1.452		2.367	3.437	
4	W_4	3.60 x 0.50 X 2.40	4.320		1.800	7.776	
7	EW_1	1/2 x 1.10 x 1.10 X 1.80	0.000		3.733	0.000	
8	EW_2	1.10 x 0.50 X 1.80	0.000		3.350	0.000	
9	EW_3	1.60 x 0.50 X 1.80	0.000		3.850	0.000	
10	HF	0.50 x 1.10 x 1.10 x X 1.00		0.605	0.367	0.222	
11	UP	0.50 x 1.00 x 1.10 x X 3.60	-1.980		2.400		4.752
			$\sum V_1$	$\sum H_1$		$\sum M_1(+)$	$\sum M_1(-)$
			7.752	**0.605**		**16.385**	**4.752**

EARTHQUAKE FORCES

Earthquake co-efficient (For zone IV) αv = 0.03 αh = 0.06

SN	Item		Calculations	Force (tonnes) Vertical	Force (tonnes) Horizontal	Lever arm (m)	Moments (t-m) (+ ive)	Moments (t-m) (- ive)
	VERTICAL							
1	$\sum V_2$	$\sum V_1$ x αv	= 7.752 * 0.03	0.233				0.492
	HORIZONTAL							
2	H_1	W_1 x αh	= 0.000 * 0.06		0.000	1.600		0.000
3	H_2	W_2 x αh	= 3.960 * 0.06		0.238	1.050		0.250
4	H_3	W_3 x αh	= 1.452 * 0.06		0.087	0.867		0.075
5	H_4	W_4 x αh	= 4.320 * 0.06		0.259	0.250		0.065
8	H_7	EW_1 x αh	= 0.000 * 0.06		0.000	1.233		0.000
9	H_8	EW_2 x αh	= 0.000 * 0.06		0.000	1.050		0.000
10	H_9	EW_3 x αh	= 0.000 * 0.06		0.000	0.800		0.000
11	H_{10}	HF x αh	= 0.605 * 0.06		0.036	0.367		0.013
12	H_{11}	UP x αh	= -1.980 * 0.06		-0.119	1.200		-0.143
				$\sum V_2$	**$\sum H_2$**		**$\sum M_2(+)$**	**$\Sigma M_2(-)$**
				0.233	**0.501**		**0.000**	**0.752**

$\sum V = \sum V_1 - \sum V_2$ = 7.519 t

$\sum H = \sum H_1 + \sum H_2$ = 1.106 t

$\sum M = [\ \sum M_1(+) + \sum M1(+)\ ] - [\ \sum M1(-) + \sum M2(-)\ ]$ = 10.881 t-m

Z = ∑M / ∑V = 10.881/7.519	=	1.447
e = b/2 - Z =3.6/2 - 1.447	=	0.353
b/6	=	0.600
e <= b/6	=	**SAFE**

Vertical stresses at Toe

Pmax = (∑V/b) x (1 + (6 * e / b))

= (7.519 / 3.6) *(1 + (6 * 0.353 / 3.6))	=	3.32
<= 45 t/m^2	=	**SAFE**

Vertical stresses at Heel

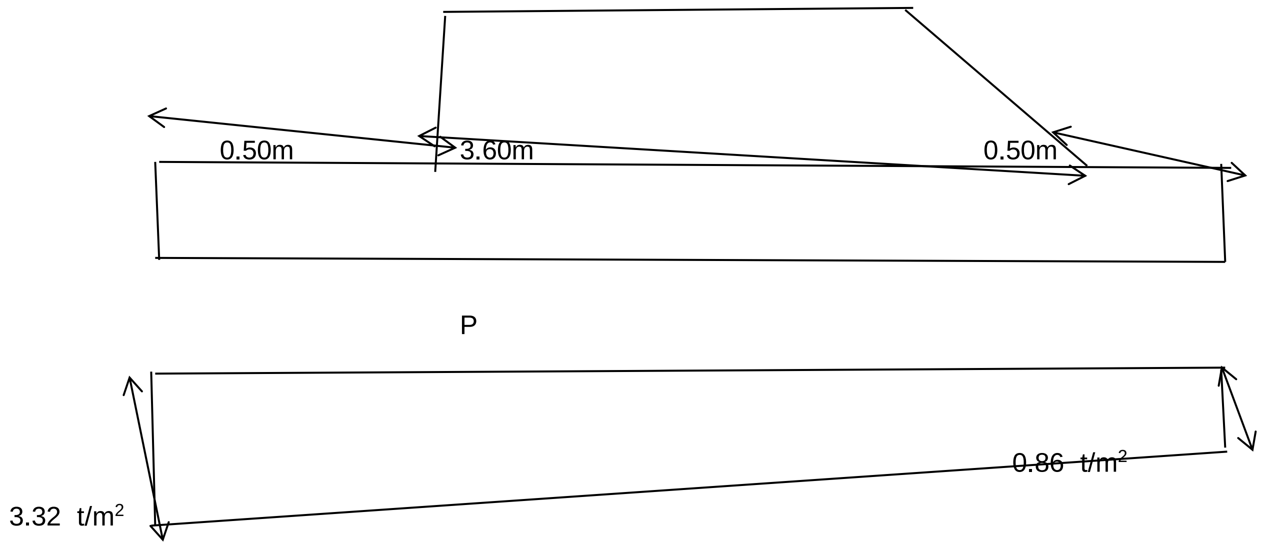
Pmin = (∑V/b) x (1 - (6 * e / b))

= (7.519 / 3.6) *(1 - (6 * 0.353 / 3.6))	=	0.86
>0 t/m^2	=	**SAFE**

Factor of Safety Against Sliding = SF = tan □ * ∑V / ∑H	=	6.80
SF > 1.5	=	**SAFE**
Check For Overturning = Total +ve Moment / Total -ve Moment	=	2.98
CFO > 2	=	**SAFE**

DESIGN OF FOUNDATION TOE

The pressure distribution on the foundation is shown below: -



Stress at toe	=	3.32 t/m^2
Stress at heel	=	0.86 t/m^2
stress at point "P"	=	3.05 t/m^2
Counterbalance weight of base concrete	=	0.95 t/m^2
Thus, the net reaction acting on the toe will be	=	2.37 t/m^2
And the net reaction acting at the point "P" will be	=	2.10 t/m^2

Now the Shear Force acting at point "P"			=	1.12 t
Maximum moment (M) at point "P"			=	0.28 tm
Section modulus (Z) at point "P"		$bd^2/6$	=	0.042 m^3
Tension at bottom	=	M / Z	=	6.83 t/m^2
Tension at bottom	0.68	kg/cm^2	<	7 kg/cm2 (allowed in masonry)

Tension at bottom is less than the 50% i.e. 3.5 kg/cm^2 of allowed tension in the Masonry.

Chapter 4 :DESIGN OF IRRIGATION CANAL

4.1 General

Irrigation development to raise the share of the area under irrigation and to bring sustainable agricultural productivity getting a series attention from time to time on the overall agricultural production improvement direction of the country. Upgrading and increasing the efficiency of traditional irrigation system through modern and well-designed system is the main objective of Wangur small scale diversion irrigation development project.

Traditional irrigation agriculture has been practicing on the project area under consideration mainly using stream and its tributaries indigenous intake exercise which often subjected to flood damage during rainy seasons.

From irrigation agronomy crop water requirement finding 0.98 lit/sec/ha under 24 hours of irrigation, base flow estimate from hydrological analysis (minimum flow data estimate ~58 liter/sec) and maximum command area size (~60ha), off take canal has been designed to carry a discharge of 58liters/sec.

4.2 Irrigation Water Source

Wangur spring is the source of irrigation water for commanding nearly 60ha of land that will be managed through direct irrigation both in dry season and supplementary irrigation seasons based on minimum flow consideration and required pondage at upstream side of the weir. A maximum flow of 58 liters/Sec proposed to be diverted to the off-take canal and subsequently distributed to secondary canals thereby covering a total of 60 hectares of land.

4.3 Methodology

The methodology employed to design the Wangur project involves the following steps:

- Preparation of Top Map:200ha which is under irrigation was surveyed and the top map was produced using 0.5m contour interval to develop the layout of the system. 60ha maximum area is found to be irrigated by the scheme based on duty which is 0.98l/s/ha for 24hr irrigation and base flow which is 58//s.

- Preparation of Layout: The components of the irrigation system should be aligned on the ridges and that of drainage in the valleys. Between two components of irrigation system, there should be one component of drainage system. The system should extend from main canals to field canals.
- Discharge Capacity: It is also required to fix the discharge capacities of different components, such that they are able to carry adequate irrigation water to properly serve the areas under their respective commands.
- Fixation of F.S.L: It is required to fix the full supply level (FSL) of each component such that it facilitates reaching of irrigation water to every corner of the command area by gravity flow.
- Hydraulic Design of Canals: Once the design discharge is fixed, it is required to complete the hydraulic design of the different components.
- Structures: Canal structures on different components are essential parts of the system; they are required for regulating, monitoring, and controlling the irrigation supplies. It is, therefore, also required to decide about the type, location, size, and number of structures and prepare their detailed designs.

4.4 Command Area

The command area of the project is determined based on the result of the topographic survey, hydrological study, and soil survey and land suitability evaluation. Accordingly, the survey has been conducted for entire command area measuring 200ha. From this, a maximum irrigable land of 60ha has been obtained. The reduced area is constraining only due to water scarcity

4.5 Hydraulic Design of Canals

4.5.1 Main Canal

Main canal is designed as a rectangular box canal using Manning's formula where masonry lining has been used to make the canal water tight and economical for about 260m which need somewhat extent needs deep excavation and as rectangular canal for about 160m. For ease of construction and to avoid dimensional irregularity, standard main canal size has been adapted. Main canal is aligned as contour canal with flat to moderate slope condition. The whole length is designed as lined rectangular canal using Manning formula and continuity equation as follows:

$Q = A*V;$

$$V = \frac{1}{N} R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

Where:

Q is Design Discharge (m^3/sec)
A is Cross-sectional area in (m^2)
V is Mean velocity (m/sec)
p is Wetted Perimeter(m)
R is Hydraulic mean depth (m)
S is Design bed slope (m/m)
n is Manning roughness coefficient

Using the above formula, the hydraulic parameters of the main canal are fixed as follows;

Bed width, b (m)	0.4
Full Supply Depth, h (m)	0.4
Manning roughness coefficient, n	0.018
Design bed slope, S	0.00033
Canal side slope, m (H: V)	0:01
Cross Sectional flow Area, A (m^2)	0.16
Wetted perimeter, P (m)	1.19
Mean Hydraulic Radius, R (m)	0.13
Velocity, V (m/sec)	0.37
Required Discharge, Q (m^3/sec)	0.058
Design Discharge, Q (m^3/sec)	0.058
A free board, F (m)	0.3

Table 2:The hydraulic parameters for canals were fixed in similar way and tabulated

Canal	Chainage(m)		b	y	Fb	D	1/S	b/d	TW	n	V	Q - Design	Remark
	From	To	(m)	(m)	(m)	m		m	m		m/s	l/s	
Inlet	0	260	0.40	0.40	0.30	0.70	3000	1.0	0.4	0.013	0.37	58	Box Lined canal
MC	260	420	0.40	0.07	0.33	0.40	29.5	5.5	0.4	0.013	2.01	58	Lined canal

Chapter 5 : ENGINEERING WORKS & COST ESTIMATE

Unit Rate Analysis

Before estimating bill of quantities of each item, rate build up is made for all bill items of the project in consideration of cost of current construction materials and approximating future inflation of input construction items (as contingencies). Based on these costs, the estimated investment cost and annual operation and maintenance costs are derived for budgetary purposes and financial viability evaluation.

Bill of Quantities & Cost Estimate

Estimated costs of the project construction, which are considered as the engineers estimate, are prepared based on three particulars: namely, the established design criteria of this project and bill of quantities and estimated current rate for construction and procurements of items. The estimated bill of quantities, rate and cost of each item are summarized in the following table.

Table 3: Summary of Estimated Engineering Cost of Project

S/No	Description of Activities	Total cost (Br)
1	Total cost of Camp	441,869.0
2	Total Cost of Road Works	72,100.00
3	Total Cost of Headwork	1,250,888.03
4	Total Cost of Canals	637,899.40
	Sub Total	2,402,756.5
4	Cost for maintenance of existing structures 25%	600,689.1
5	Management & Construction Supervision (7.5%)	180,206.7
6	Physical Contingency (10%)	240,275.6
	Subtotal	1,021,171.5
	Total	3,423,927.9
	VAT (15%)	513,589.2
	Grand total	3,937,517.14
S/No	Description of Activities	Total cost (Br)
	Total cost of Camp	441,869.0
6	Total Cost of Road Works	72,100.00
2	Total Cost of Headwork	1,250,888.03
3	Total Cost of Canals	637,899.40
	Sub Total	2,402,756.5
	Cost for maintenance of existing structures	480,551.3
	Management & Construction Supervision (7.5%)	180,206.7

	Total	3,063,514.5
	Physical Contingency (10%)	306,351.4
	S.Total	3,369,865.9
	VAT (15%)	505,479.9
	Grand total	**3,875,345.82**

Table 4: Summary of Estimated Engineering Cost of Project

SN	Description	Unit	Qty	Unit cost (Br)	Total cost (Br)
1	Camping (3m x 13.85m office & bed room, 4m x 6m kitchen & Cafeteria, 5m x 5m store, 4mx2m Toilet & Shower, 2m x 2m guard house				
1.1	Site clearing	m2	175	2.92	511.00
1.2	Excavation	m3	63.336	57.2	3,622.82
1.3	Cart away all excess excavated material for safe place with a radius of more than 500m	m3	88.52	31.2	2,761.82
1.4	25cm thick hard core	m3	89.7	36.4	3,265.08
1.5	Masonry work with 1:3 mortar mix	m3	38.404	858.54	32,971.37
1.6	5cm thick mass concrete (1:2:4 mix ratio)	m3	12.71	1698.84	21,592.26
1.7	2cm cement screed	m2	91	90.64	8,248.24
1.8	CIS walling G-32	m2	337	142.91	48,160.67
1.9	CIS roofing G-32	m2	194.5	211.94	41,222.33
1.10	Chip wood wall ceiling	m2	256	129.33	33,108.48
1.11	Supply, assemble and fix in position eucalyptus wall post of length 3 m with span length of 1.2m	No	161	225	36,225.00
1.12	Supply and fix purlin in Eucalyptus wood size 50 x 70 mm nailed into eucalyptus truss	m	586	69.375	40,653.75

1.13	Supply, assemble and fix in position eucalyptus roof truss	No	36	225	8,100.00
1.14	Supply and fix purlin in zigba wood size 50 x 70 mm nailed into eucalyptus truss including three coats of anti - termite external treatment	m	190	69.375	13,181.25
1.15	Supply and fix CIS doors size 1.0x2.10m with accessaries	No	14	3139.25	43,949.50
1.16	Supply and fix CIS windows size 1x1.2m	No	9	2150.25	19,352.25
1.17	Fence 2.0m height & 15cm φ eucalyptus poles placed every 2m with barbed wire at 20cm vertical interval & erected in 0.6m depth embedded with concrete	LS	1	84943.2	84,943.20
	Subtotal 1				**441,869.02**
2	Access Road Construction				
2.1	Access Road Maintenance cutting to an average depth of 0.2m, with 6m width	km	7	7000	49,000.00
2.2	Field Road construction cutting to an average depth of 0.15m with 3m width	km	6.6	3500	23,100.00
	Subtotal 2				**72,100.00**
3	Cost of Headwork				
3.1	Site Preparation				
3.1.1	Clearing & Grubbing	m2	716.08	8.38	6,004.01
3.1.2	Temporary Diversion Work	m3	1,134.00	364.13	412,920.08
3.2	U/S & D/S Launching Apron		-		-
3.2.1	Common soil Excavation	m3	37.23	47.92	1,784.17

3.2.2	Soft Rock Excavation	m3	55.85	118.20	6,601.30
3.2.3	Hard Rock Excavation	m3	93.08	364.13	33,892.48
3.2.4	Wett Riprap	m3	186.16	650.00	121,002.34
3.3	U/S & D/S CC Block		-		-
3.3.1	Common soil Excavation	m3	77.58	47.92	3,717.84
3.3.2	Soft Rock Excavation	m3	164.04	118.20	19,389.62
3.3.3	Hard Rock Excavation	m3	58.09	364.13	21,151.42
3.3.4	1.25x1.25 CC Block Laying	m2	126.40	250.00	31,600.80
3.3.5	0.60 thick Inverted Filter	m3	75.84	200.00	15,168.38
3.4	Weir		-		-
3.4.1	Common soil Excavation	m3	8.13	47.92	389.53
3.4.2	Soft Rock Excavation	m3	12.19	118.20	1,441.22
3.4.3	Hard Rock Excavation	m3	20.32	364.13	7,399.53
3.4.4	Cyclopean Concrete	m3	50.61	1,700.00	86,036.43
3.4.5	Concrete C-20	m3	29.03	2,961.00	85,959.01
3.4.6	Plastering	m2	37.97	149.79	5,686.82
3.5	Stilling Basin		-		-
3.5.1	Common soil Excavation	m3	9.43	47.92	451.85

3.5.2	Soft Rock Excavation	m3	14.14	118.20	1,671.81
3.5.3	Hard Rock Excavation	m3	23.57	364.13	8,583.45
3.5.4	Cyclopean Concrete	m3	17.56	1,700.00	29,857.77
3.5.5	Concrete C-20	m3	16.26	2,961.00	48,137.05
3.5.6	Concrete C-25	m3	16.26	3,015.00	49,014.93
3.5.7	RIB		-		-
3.5.8	Dia. 10mm	Kg	334.35	60.00	20,061.17
3.5.9	Plastering	m2	54.29	149.79	8,131.40
3.6	L&R Side Retaining Wall		-		-
3.6.1	Common soil Excavation	m3	2.48	47.92	118.68
3.6.2	Soft Rock Excavation	m3	3.71	118.20	439.10
3.6.3	Hard Rock Excavation	m3	6.19	364.13	2,254.43
3.6.4	Masonry	m3	53.49	1,842.18	98,544.21
3.6.5	Concrete C-15	m3	12.72	2,358.00	29,982.71
3.6.6	Plastering	m2	33.82	149.79	5,066.22
3.6.7	Back Fill & Compaction	m3	74.05	137.40	10,174.47
3.7	Divide walls		-		-
3.7	Common soil Excavation	m3	1.16	47.92	55.76

3.7	Soft Rock Excavation	m3	1.75	118.20	206.32
3.7	Hard Rock Excavation	m3	2.91	364.13	1,059.28
3.7	Masonry	m3	16.87	1,842.18	31,084.84
3.7	Plastering	m2	20.14	149.79	3,016.66
3.8	L&R Side Upper Concrete Plate		-		-
3.8.1	Concrete C-25	m3	0.81	3,015.00	2,448.01
3.8.2	Plastering	m2	6	149.79	859.43
3.8.3	RIB		-		-
3.8.4	Dia. 10mm	Kg	10.64	250.00	2,659.94
3.90	Head Regulators		-		-
3.9.1	For Left Side		-		-
3.9.2	Common Soil Excavation	m3	2.89	47.92	138.40
3.9.3	Soft Rock Excavation	m3	4.33	118.20	512.06
3.9.4	Hard Rock Excavation	m3	7.22	364.13	2,629.03
3.9.5	Concrete C-15	m3	0.54	2,358.00	1,262.11
3.9.6	Concrete C-20	m3	0.45	2,961.00	1,325.20
3.9.7	Concrete C-25	m3	1.58	3,015.00	4,759.26
3.9.8	RIB		-		-
3.9.9	Dia. 10mm	kg	34.45	60.00	2,066.74

3.9.10	Dia. 8mm	kg	14.33	60.00	860.03
3.9.11	Masonry	m3	1.66	1,842.18	3,065.25
3.9.12	Plastering	m2	1.25	149.79	186.62
3.9.13	Back Fill & Compaction	m3	2.27	137.40	311.61
3.9.14	Intake gate		-		-
3.9.15	(1.00m X 0.80m)	pcs	0.03	15,000.00	453.60
3.9.16	*(1.00m X 1.6m) for three bays*	pcs	0.09	18,000.00	1,632.96
3.10	L Side Protection Work		0.00		-
3.10.1	Back Fill & Compaction	m3	128.5	137.40	17,660.68
	Subtotal3		-		**1,250,888.03**
4	Cost of Canals in the System		-		
4.1	Cost of Main Canals		-		
4.1.1	Lined Left Main canal (LMC), L=420.04m		-		
4.1.1.1	clearing up to 20m cm depth soil	m^2	748	8	5,984
4.1.1.2	Excavation of ordinary soil depth	m^3	771	54	41,641
4.1.1.3	Excavation of Soft Rock	m^3	123	119	14,682
4.1.1.4	Excavation of Hard Rock	m^3	134	365	48,786
4.1.1.5	Masonry work with 1:3 mortar work	m^3	219	1,689	369,417
4.1.1.6	Plastering with 1:3 mix ,3 coats	m^2	477	150	71,477

4.1.1.7	5mm thick coping of C 15	m^3	22	2,358	52,435
4.1.1.8	Fill and Compaction of Normal Soil	m^3	243	138	33,478
	Sub Total of 2		-		**637,899**
	Total				2,402,756.45

REFERENCES

- American Society of Civil Engineers (ASCE) 1960. Design manual for storm drains. New York.
- Ankum, P 1992. Flow control in irrigation systems; Technical University of Delft, Delft; The Netherlands. lecture notes.
- Basak, N. N. 2007. Irrigation Engineering. 10th edition. Tata McGraw-Hill Publishing, New Delhi, India.
- Bos, M. G. 1989. Discharge measurement structure. International Institute for Land Reclamation and Managament (ILRI) Publication No. 20. Wageningen, The Netherlands. 3rd edition.
- Chow, V.T 1973. Open - Channel Hydraulic, Mc. Graw – Hill Book Company Ltd.
- Davis C. and K. E. Sorenson 1969. Canal and conduits: Handbook of hydraulics. McGraw-Hill books. New York. 3rd edition.
- Ethiopian Road Authority (ERA) 2002. Drainage design manual.
- Food and Agricultural Organization of the United nation (FAO) 2001. Irrigation water management: Irrigation methods. AGLW Training manual No. 5 prepared by International Land reclamation Institute and Consultant.
- Food and Agricultural Organization of the United Nation (FAO) 2002, Irrigation manual: Planning, development, monitoring and evaluation of irrigated agriculture with farmer's participation. Vol. I.
- Federal Highway Administration (FHWA) 1984. Hydrology: Hydraulic Engineering Circular No. 19. US Department of Transportation
- Federal Highway Administration (FHWA) 1990.HYDRAIN Documentation.
- Federal Highway Administration (FHWA) 2001. Urban drainage design manual. Hydraulic Engineering circular No. 22. National Highway Institute, US Department of Transport. 2nd edition.
- Framji, K K (1972); Design practices of irrigation canals in the world; ICID, New Delhi
- French, Richard H. 1986. Open-Channel hydraulics. McGraw-Hill.2nd edition.
- Indian Standard (IS) 1987. Guideline for planning and layout of canal system for irrigation.IS 5968-1987.
- Indian Standard (IS) 1991. Code of practice for earthwork on canal.IS: 4701-1982.2nd edition.
- Indian Standard (IS) 1995. Guidelines for planning and design of surface drain.IS: 8835-1987.2nd edition.
- Indian Standard (IS) 2002. Criteria for design of cross section for unlined canal in alluvial soil.IS 7112:2002.
- Indian Standards 2002.Criteria for design of cross section for unlined canals in alluvial soil.IS 7112:2002.1st edition.Bureau of Indian Standard.
- Kay, M. 1986. Surface irrigation: system and practice. Cranfield press, Bedford, UK.
- Khatsuria, Rajnikant2005.Hydraulics of spillway and energy dissipaters.
- McCuen, R. H., Johnson, P. A., and Ragan, R. M., 1995. Hydrology: Hydraulic Design Series No. 2, Federal Highway Administration (FHWA).
- Milenkovic, Dragana 1987: Seepage losses in irrigation canals: A literature survey. IHE, Delft, The Netherlands.
- RangaRaju, K. G. 1983. Flow through open channel. Tata McGraw Hill, New Delhi, India.

- Smerdon, E. T. and R.P. Beasley 1961.Critical tractive force in cohesive soils.J. of Agric. Engrg. American Soc. of Agric. Engineers.
- Smith, K. V. H. 1984. Design of stable channels. Lecture notes; IHE, Delft, The Netherlands.
- US Army Corps of Engineers 1970.Hydraulics design of flood control channel. Report EM 1110-2-1601.
- United State Bureau of Reclamation (USBR) 1967. Design standard Nr 3, Canals and related structures, 1967, United State Department of the Interior, Bureau of Reclamation, Office of Chief Engineer, Denver Colorado
- United State Bureau of Reclamation (USBR) 1978.Design of small canal structure. Denver, Co.
- USDA Soil Conservation Service 1973.A Method for Estimating Volume and Rate of Runoff in Small Watersheds.
- Vlugter, H 1962. Sediment transportation by running water and the design of stable alluvial channels; den Haag, The Netherlands.
- Wither, B. and S. Vipond 1974. Irrigation: design and practice. Batsford, London, UK.