

# **Oromia National Regional State Irrigation Development Authority**

## **Wataba Baddesa Small Scale Irrigation Project Feasibility And Detail Design Report**



**Arsi Zone, Oromia Regional State**

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HM Development Consulting PLC

P.O. Box 1281, 2<sup>nd</sup> floor, Seblewengel Building,

Tiruffat to Alamura Avenue, Hawassa, ETHIOPIA

Phone: +251 46 212 4401/+251 916 451 962



## Table of Contents

LIST OF TABLES .....	iv
LIST OF FIGURES .....	iv
SALIENT FEATURE .....	v
1 INTRODUCTION.....	1
1.1 Background.....	1
1.1 Project Location and Accessibility .....	1
1.1.1 Location.....	1
1.1.2 Accessibility .....	2
1.2 Objective of the Project .....	2
1.2.1 General Objective .....	2
1.2.2 Specific Objectives .....	3
1.2.3 Scope of the study .....	3
1.2.4 Methodology.....	3
2 HEADWORK DESIGN.....	5
2.1 GENERAL.....	5
2.2 Methodology.....	5
2.3 HEADWORK SITE SELECTION.....	6
2.3.1 Assessments of Head Work Site.....	6
2.3.2 Weir Type selection.....	7
2.4 Geology of the head work.....	8
2.5 Components of Head works.....	9
2.6 General Assumptions .....	9
2.7 Hydraulic design of weir and Appurtenant structures .....	9

2.8	Structural design of Weir and Appurtenant structures.....	22
2.9	Backwater effect .....	32
2.10	Temporary river diversion during construction .....	32
3	IRRIGATION AND DRAINAGE DESIGN .....	34
3.1	General.....	34
3.2	Command Area .....	34
3.3	Irrigation System Design .....	34
3.3.1	Layout system.....	35
3.3.2	Water Distribution system.....	37
3.3.3	Naming of canal units.....	41
3.3.4	Crop Water Requirement.....	42
3.3.5	Topographic and soil survey .....	42
3.4	DESIGN OF IRRIGATION CANALS .....	42
3.4.1	General .....	42
3.4.2	Main Canal .....	43
3.4.3	Secondary Canals .....	47
3.5	Tertiary Canals.....	47
3.6	Field Canals .....	48
3.7	Design Of Irrigation Structures.....	49
3.7.1	General .....	49
3.7.2	Design of Drop Structures .....	49
3.7.3	Design of Division Boxes.....	51

3.7.4	Off-takes .....	53
3.7.5	Gates .....	55
3.7.6	Crossing Structures .....	56
3.7.7	Road crossings Culverts .....	58
3.8	DESIGN OF DRAINAGE SYSTEM .....	60
3.8.1	General .....	60
3.8.2	The objectives of surface drainage measures are to: .....	60
3.8.3	Existing Natural Drainage System .....	61
3.8.4	Description of the Drainage Network Layout of the project .....	61
3.8.5	Drain Design Discharges .....	61
3.8.6	Hydraulic Design of Drainage Channels .....	62
4	PHYSICAL AND SOCIAL INFRASTRUCTURES .....	65
4.1	ACCESS ROAD .....	65
4.2	Camping .....	65
4.3	Foot Bridge .....	65
4.4	Washing Basin .....	66
4.5	Cattle Trough / Animal Water Point .....	66

## LIST OF TABLES

TABLE 2-1: WEIR HEIGHT DETERMINATION .....	10
TABLE 2-2:- WEIR DESIGN RESULT SUMMARY .....	11
TABLE 2-3:- SUMMARY FOR HYDRAULIC DESIGN OF UNDER SLUICE .....	13
TABLE 2-4:- SUMMARY FOR HYDRAULIC DESIGN OF INTAKE/ HEAD REGULATOR .....	14
TABLE 2-5:- SUMMARY OF WEIR SECTION AFTER STABILITY ANALYSIS RESULT .....	15
TABLE 2-6:- HYDRAULIC JUMP CALCULATION SUMMARY .....	17
TABLE 2-7:- MOMENT AND LOAD CALCULATION RESULT IN MINIMUM WATER LEVEL.....	26
TABLE 2-8:- MOMENT AND LOAD CALCULATION RESULT IN MINIMUM WATER LEVEL.....	27
TABLE 3-1: PERTINENT CANAL FEATURES AND HYDRAULICS PARAMETERS ALONG MAIN CANAL.....	45
TABLE 3-2: HYDRAULICS PARAMETERS OF SECONDARY CANAL .....	47
TABLE 3-3: HYDRAULICS PARAMETERS OF TERTIARY CANALS .....	48
TABLE 3-4: DESIGN OUTPUT OF DIVISION BOXES AND TURNOUTS ON MAIN AND SECONDARY CANALS	54
TABLE 3-5 FLOOD RETURN PERIODS FOR CRESS-DRAINAGE STRUCTURE DESIGN .....	57
TABLE 3-6 SUMMARY OF CROSS DRAINAGE STRUCTURES BY SUPPER PASSAGE .....	58
TABLE 3-6 DRAINAGE CANALS HYDRAULIC DESIGN OUT PUT .....	62
TABLE 4-1:- SUMMARY OF FOOT BRIDGE IN THE MAIN CANAL .....	65

## LIST OF FIGURES

FIGURE 1-1:- PROJECTS LOCATION MAP .....	2
FIGURE 2-1: HEAD WORK SITE.....	7
FIGURE 2-2:- PHOTO. SHOWS VARIOUS FEATURES OF WATABA BADDESA RIVER AROUND HEAD WORK AREA. ....	9
FIGURE 2-3: FLOW OVER WEIR BODY. ....	16
FIGURE 2-4:- LOAD DISTRIBUTION DURING MINIMUM WATER LEVEL .....	25
FIGURE 2-5:- LOAD DISTRIBUTION DURING MINIMUM WATER LEVEL .....	26
FIGURE 3-1 FURROW SHAPE DEPENDING ON SOIL TYPE .....	38
FIGURE 3-2: LINED AND EARTHEN CANAL SECTION.....	45
FIGURE 3-3: TYPICAL DIVISION BOX .....	52
FIGURE 3-4: TYPICAL CROSS SECTION OF THE MAIN CANAL AT FOOT PATH CROSSING .....	60

## SALIENT FEATURE

- a. Project name: **Wataba Baddesa Small Scale Irrigation Project**
- b. Name of the stream: **Wataba Baddesa River**
- c. Location of the Head Work Site:-
  - East : 565362.446
  - North : 847984.861
  - Zone: **Arsi**
  - District: : **Shirka**
  - Average Altitude: 1694 m.a.s.l
- d. Hydrology
  - Design flood: 196.34 m<sup>3</sup>/s
  - Design base flow: 727.1lit/se.
- e. Irrigation and drainage systems Infrastructure
  - Net Command area size : **63 ha**
  - Type of soil group of the command area is dominantly Luvisols and Cambisols.
  - Design discharge of the main canal = **119.7l/sec**
  - Irrigation system layout consists of **one/01/(3.59km)** lined main canal, **Five/05/(0.81km)** lined Secondary canal **Twenty two /22/(5.6km)** earthen tertiary canals
  - Main irrigation structures designed are; Road crossing structures, Drainage Crossing Structures, and Division box/Turnout
- f. **Project cost**

Net project construction cost without management , contingency and VAT

Bill No.	Description	Amount (ETB)
Bill Part-1	General Item Cost	2,013,020.87
Bill Part-2	Head Work	3,050,589.64
Bill Part-3	Main Canal and Canal Structures	10,142,314.70
Bill Part-4	Irrigation Infrastructure	2,179,619.38
<b>Total Carried to Summary</b>		<b>17,385,544.59</b>
<b>15% Vat</b>		<b>2,607,831.69</b>
<b>Grand Total</b>		<b>19,993,376.27</b>
<b>Cost per hectare</b>		<b>317,355.18</b>

# 1 INTRODUCTION

## 1.1 Background

Agriculture accounts for half of the Ethiopian gross domestic product, 80% of its exports, and 80% of total employment (USAID, 2004). More than 85% of the Ethiopian population engaged in agriculture as a major source of livelihood.

Irrigation helps to lower the incidence of poverty better than in other farming systems and absolute number of the poor are small. According to (UNHDR, 2006) there is a potential of irrigating up to 3.7 million hectares of land in Ethiopia however the irrigated land so far accounts only 300,000 hectares or slightly greater than to 10% of the total potentials.

On the other hand, Oromia Regional State has adequate resource endowment with respect to the natural resources for irrigated farming. An agreement was signed on February 2018 between the HM Development Consultant PLC (here after the Consultant) and Oromia Irrigation Development Authority (OIDA) (Client) to conduct study and design of Lot 4 irrigation system at Munesa, Digelu & Tijo and Shirka Woredas in Arsi zone, Oromia National Regional State. This draft feasibility report, as required in the letter of agreement, contains the draft design of the head work and the irrigation systems, Overall project cost, and recommendations.

HM Development Consulting PLC deployed multidisciplinary study crew for the feasibility study. Wataba Baddesa Small Scale Irrigation Project is one of the irrigation scheme identified for further study and design at Shirka District, Arsi Zone in Oromia Region.

## 1.1 Project Location and Accessibility

### 1.1.1 Location

The project located in Oromia Regional State, Arsi zone, Shirka woreda, Eleliwalena kebele.. The command area also covers similar kebele.

The proposed head work located 565362.445m Easting, 847984.861m Northing, the proposed possible gross command area covers 565257.243m to 568803.604 East and 848756.295m to 847741.346m North. and an elevation range of from 1656 to 1736 meters above sea level (masl)

### 1.1.2 Accessibility

35km all weathered road from Gobesa, (Shirka woreda capital), 1.5km defective dry weather road, which require rehabilitation and 4km inaccessible road. Gobesa is 256km far from Finfinnee.

Currently the project site is not accessible by vehicles the study team under take the survey by foot, **8km access road with one small bridge** is mandatory for further project implementation.

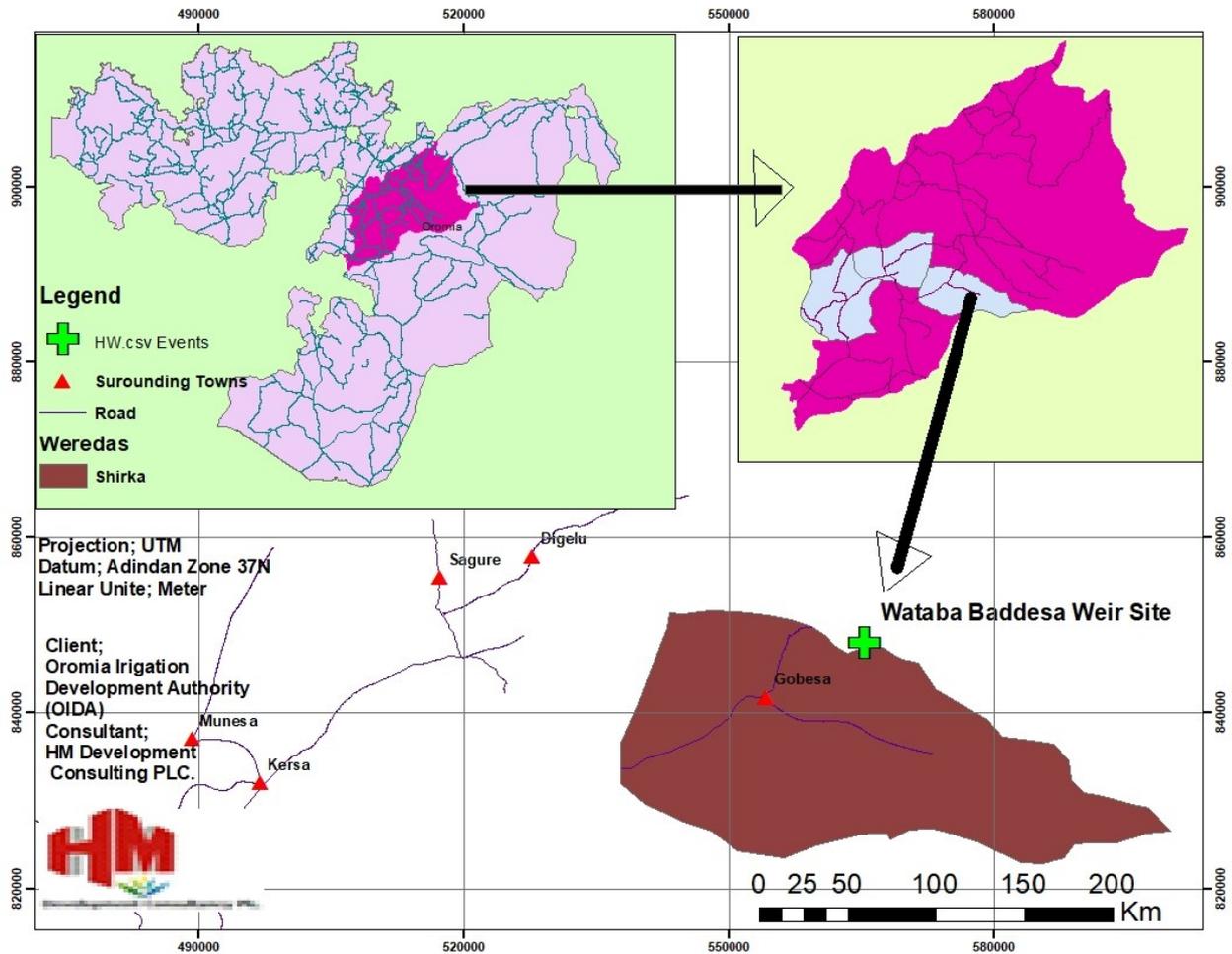


Figure 1-1:- Projects Location Map

## 1.2 Objective of the Project

### 1.2.1 General Objective

The prime objective of enhancing the implementation of small-irrigation project is the starting point for securing better livelihood for the rural poor population. To make this happen, several attempts are underway, of which studying and designing of small-scale irrigation scheme by the regional officials and private firms is considered as a short cut for increasing the pace of irrigated agriculture development.

### 1.2.2 Specific Objectives

The specific objectives of the project studies are:

- To conduct feasibility study of the project and investigation for optimum use of land and water resources of the area under sustainable, technical, social, financial, and environmental condition.
- To prepare detail design, cost estimate and tender documents of the most feasible option for implementing the project, and
- To develop operation and maintenance manual guideline for the smooth and efficient running of the irrigation farms.

### 1.2.3 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.

- Computation of the actual evapo-transpiration, crop water requirement, irrigation demand/duty using agronomic data, climatologic and soil data using more appropriate methodologies.
- Design proper irrigation system compatible with local conditions and management capabilities,
- Planning and layout of the irrigation system, which include irrigation canals, drainage channels, 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,

### 1.2.4 Methodology

In the study and design procedure, the following steps are used.

- Specific Site identification:
  - Review of the reconnaissance survey
  - 50,000 scale top map and GIS information
- Local farmers interview and discussion
- District and Zone OIDA
- On foot travel along the river and farm areas.
- Topographic survey:
  - Surveying the headwork site and the Command area with sufficient radius, using Total station

## 2 HEADWORK DESIGN

### 2.1 GENERAL

The source of water for the scheme is the Wataba Baddesa River. The potential resources of the site for irrigation development were identified during the pre-feasibility study. As it was shown in the hydrological report, the 50 year design flow for 191 km<sup>2</sup> is about 196.34 m<sup>3</sup>/s .

Engineering aspects related to project study and design is addressed under this section. All influential factors that determine project sustainability are accounted to come up with reasonable results of study. In so doing, idea from different disciplines was given due attention to make the scheme smart. The project is gravity diversion system on Wataba Baddesa River. One-way intake is suggested to serve the district. Manageable systematic layouts were adopted to minimize costs and to enable easier application of water. A total of 94.65ha of land were surveyed and net irrigable area of 63 ha of land was planned for development.

Prior to final selection of the best possible headwork site, a general survey to determine the feasibility of the project, topographical features of the area, possible sites for locating the proposed weir and which can command the available irrigable area were carried out. The survey works were done by the use of precise power set surveying instrument.

### 2.2 Methodology

The following procedures followed to come up with the detail design of diversion weir on Wataba Baddesa Small Scale Irrigation Project.

- Selection of suitable type of headwork,
- Appropriate site selection made according to weir site selection criteria.
- Topographic surveying of the headwork site and plotting to 1:100 scale maps.
- Taking a cross- section at downstream of the weir axis, a tail water depth is calculated.
- Average stream slope is determined.
- Determination of appropriate weir structure type which is hydraulically efficient and economical.
- Hydraulic design of the weir body adopted and other structures like retaining walls sluice and Intake designed.

Finally, a plan view or layout of the weir with its accompanying structures will be prepared on a headwork contour map in 1:100 scales.

## 2.3 HEADWORK SITE SELECTION

The diversion head works are generally located in the boulder stage or trough stage of the river at a site which is close to the command area of the off taking canals. If there are a number of sites which are suitable, the final selection is done on the basis of cost. The site which gives the most economical arrangement for the diversion head works and the distribution works (canals) is usually selected. The ideal head work site will be selected on the basis of the following criteria stated at the approved design criteria report :

- The geological condition of the foundation and the abutments
- Downstream and Upstream protection works cost of the weir site
- The hydraulic crest level of the weir
- The width of the river
- The length of the main canals
- The maximum irrigated command area level
- The straightness of the reach of the river
- Defined channels and banks
- Narrow and stable banks of the river
- Relative cost of weir.
- Access to the site
- Area simpler for temporary diversion during construction

### 2.3.1 Assessments of Head Work Site

Like other water resource schemes the selection of head work site was assessed based on feasibility to address the water from Wataba Baddesa River to the entire irrigable area which identified at reconnaissance study. And also consider the geology of the banks and the river bed, the possibility to escape the main canal from the river bank and the optimal length of the main canal. The feasibility study team has assessed both u/s and d/s looking for the best diversion site to address the proposed command area at Elele Walena Kebele for construction of scheme at a reasonable cost and technically feasible structure.

The headwork site is selected in accordance with attainment sufficient head for the entire proposed command area and considering geology of the river. The assessed options of the head work site are as shown as the image shown below.



*Figure 2-1: Head work site*

During the period of field assessment, it was tried to critically observe all possibilities by walking upstream and downstream from the initial options. Considering river bank and bed stability, slope, suitable escaping position for the outlet canal. And highest location from the command area the head work site selected.

### **2.3.2 Weir Type selection**

The nature of the river that transport gravel and boulder size is given a special emphasis to select the weir type. As a result, water flowing with high kinetic energy with boulder and gravel deposition should pass the weir without crushing the structure. And also the weir height influence the type. Thus the weir type that will be proposed should be able to minimize this all defect as much as possible.

Classification of weir could be due to stabilizing factors; construction material, control surface, function and geometry of control section. Usually the economics is the main reason for the selection of weir type, which is influenced by the availability of suffice construction material at

close proximity and availability of skilled and unskilled labor at site and duration of the construction time. It is to the designer to select the best one satisfying the maximum mentioned conditions.

Considering the above reasons, Broad crested weir is preferable for **Wataba Baddesa Small Scale Irrigation Project** mainly because the weir height is small. And considering the stability because the area is prone to high flood the weir would be constructed of cyclopean concrete externally covered with 250mm thick reinforced concrete to protect from cracking and shearing.

## 2.4 Geology of the head work

Geology along the diversion weir axis, up and down stream beds and banks of **Wataba Baddesa River** is determined based on visual observation of outcrops and by pit excavation. The proposed head work site is located at relatively deep cut section of the wide valley where the river course is not well defined and matured so that high erosion and deposition processes were undergoing. The river bank is found to be steep at the left and gentle at the right side from the river flow direction. The local Geology of the left bank the center and the right side peripheries of the river of weir axis are covered by outcropped columnar fracture of Aphanitic basalt. But the right bank Aphanitic basalt is overlain by in-situ sandy silt. This basaltic material feature continues along the main canal since the canal route follows the toe of the cliff. Most of the command area and the remaining main canal rout is surfaced by fluvial of sandy silt. The command area right of the main canal mainly characterized by alluvial of weathered and decomposed basalt from the high land. The river course is covered by different sizes of alluvial deposits. Most of them are well rounded boulders, gravels and blocks of different rocks. The bedrock exposed at the left side of the river bank and the pit excavation result shows the presence of sound foundation rock at shallow depth. In short, the foundation material is found to be stable and water tight after removing the most top weathered material and alluvial deposits.



*Figure 2-2:- Photo. Shows various features of Wataba Baddesa River around Head Work area.*

## 2.5 Components of Head works

The essential components of head works are:

- i. Weir;
- ii. Under sluices;
- iii. Canal head regulator;
- iv. Divide wall;
- v. Piers and abutments;
- vi. Protection works;

## 2.6 General Assumptions

The hydraulic design of the weir that consists of height, crest length, flow depth, jump effect and others would basically consider:

- ❖ Maximum river flood discharge in 50 Years return Period.
- ❖ Maximum Command elevation.
- ❖ Bank level at weir site.
- ❖ Head loss.
- ❖ Permissible afflux

## 2.7 Hydraulic design of weir and Appurtenant structures

The hydraulic design of weir consists of Flood level at different section of the weir, water depth, afflux, and hydraulic energy level. Considering the natural width of the river, the total over flow

depth over the crest and the height of the crest, the weir crest length of the overflow section will have been recommended. Based on the natural river width, other parameters are determined.

The actual river width is 31m. Considering the natural width of the river, the total over flow depth over the crest and the height of the crest, 24m of the length of the crest of the overflow section has been recommended. Based on the recommended parameters, other parameters are determined.

The hydraulic design would basically consist of the following steps:

(i) **Fixing of design flood discharge**

From hydrology report, the 50-year return period discharge is 196.34m<sup>3</sup>/s and the respective tail water depth is 2.06m

(ii) **Weir Hydraulic Design**

Design Discharge over weir is given by

$$Q = CL_e H_e^{3/2}, C = 1.7,$$

➤ Approaching velocity is given by

$$Va = \frac{Q}{L(P+h)} \text{ Or } Va = (2g * (H - h))^{0.5}$$

The weir height is fixed based on the natural river bank level and maximum command area in the first command outlet i.e. at 0.397km from headwork. The weir height is used to compensate the head loss due to slope and different minor loss and to protect entry of silt in to the proposed canal. The weir is designed using 24m weir width. The following Table shows the detail weir height calculations.

**Table 2-1: Weir height determination**

Description	Value	Unit	Remark
Maximum flood in 50 years return period	196.34	m <sup>3</sup> /sec	
Maximum Command area Elevation (@ 0+400)	1713.00	m	
Width of river at this maximum flood in specific X-section	24	m	
Main Canal Length	397	m	
Main Canal Slope	0.001	m	
Head Loss at Head regulator and turn out	0.15	m	
River bottom elevation (lowest point )	1713	m	
Head Loss due to slope	0.397	m	

Description	Value	Unit	Remark
Head loss across head regulator	0.1	m	
Head loss at the Turn out	0.05		
Operational head losses	0.1		
Water depth in the conveyance canal	0.4		
Weir Crest level	1714.1	m	
Depth of flow in maximum discharge	2.6	m	
Discharge over weir	82.01	0.000	$Q= C*Le*He^{(3/2)}$
Flood on the right abutment	114.33		
Total Discharge	196.34		
Maximum height flood level	1716.68	m	
Weir height from Command Elevation Respect	1.1	m	

**Finally, the weir height is fixed to be 1.1m and the weir crest level is fixed to be 1714.1m.**

***(iii) Fixation of pond level: -***

Pond level in the under sluice pocket upstream of the canal head regulator and upstream of weir portion is generally obtained by adding the working head to the designed full supply level in the canal. The working head includes the head required for passing the design discharge into the canal, the head losses in the regulator and head loss through the trash - rack and for possible rise of FSL in the canal due to silting in the head reach of the canal. The pond level will be fixed in such a way that with the available working head (Pond level-F.S.L of the off taking canal) and provided water way of the head regulator, the design discharge of the main canal flows in to the canal. In the Wataba Baddesa Small Scale Irrigation Project, the driving head will have taken as 0.28m.

***(iv) Determination of optimum waterway and Afflux: -***

The length of water way, corresponding discharge per meter and afflux are co - related. By providing higher afflux the length of the weir can be reduced but the cost of weir and training works may increase due to increased head of water. These parameters are decided after consideration of many practical aspects such as effect of back water on the existing structures and submergence of land. Afflux is generally limited to 1.2m but may be kept higher if permissible. In Wataba Baddesa case the afflux is 0.823.

***Table 2-2:- Weir Design Result Summary***

Description	Value	Unit	Remark
Design discharge	196.34	m <sup>3</sup> /sec	

Weir crest elevation	1714.1	m	
Weir crest length	24	m	
Approach Velocity ( $V_a = Q/(L*(P+H))$ ), take the water depth (P=Weir Height)	2.593	m/s	$V_a = Q/L*(P+H)$
Velocity head ( $h_v$ )	0.343	m	
Tail water depth	2.06	m	From discharge stage curve
U/S HFL	1716.678		
U/S TEL	1716.899	m	
D/S HFL	1715.855	m	
D/S TEL	1716.197	m	
Afflux	0.823	m	Afflux should be less 1.2m

(i) **Under Sluices: -**

The under sluices are the gate controlled openings in the weir with crests at lowest level along the weir axis. It is located on the same side as off taking canal. The usual functions of the under sluices are:

- To preserve a clear and defined river channel approaching the canal regulator
- To scour silt deposited in front of canal regulator and control silt entry in the canal
- To lower the highest flood level by providing greater discharge per meter length

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

- It should be capable of passing at least five times the base discharge to ensure good capacity.
- It should be capable of passing about 10-20% of the maximum flood discharge during high floods.
- It should be wide enough to keep the approach velocities sufficiently lower than the critical velocities to ensure maximum settling of suspended silt load.

The crest level of the under sluices is usually kept near the bed level in the deepest channel where it is practically possible. The under sluice crest is kept low to attract a deep current in front of regulator so that dry weather current may remain near the regulator. It would be desirable to keep the crest and upstream floor level in front of under sluices at the same level.

Having tentatively decided the crest levels as well as the water way of the under sluice and the weir proper, adequacy of the water way is checked such that the maximum flood discharge passes down the works (weir and under sluices) without excessive afflux.

The hydraulic design of weir and under sluices have been carried out for the following two conditions;

- In the rare case, when the under sluices are not operational and all the design flood passes over the weir; and
- During the floods, the design flood passes over the weir crest, as well as through the under-sluice bays, when the gates of the under-sluice bays are fully open.
- The capacity of the sluice decided based on passing five times the base discharge, because the 20% design flood is too high which will require a very large under sluices.
  - Base flow :0.722 m<sup>3</sup>/s → Sluiceway capacity: 2 x 0.722 = 3.61 m<sup>3</sup>/s
  - 50 year design flood over the weir: 196.34m<sup>3</sup>/s → Sluiceway capacity: 0.2 x 196.34 = 29.45 m<sup>3</sup>/s (which is too much)
- Thus minimum sluiceway capacity has to be **3.61m<sup>3</sup>/s**.
- The bottom elevation is fixed in the minimum river bed level. The headwork has one side command area the under sluice is located in the left bank considering the command area capacity and bank relative stability.

*Table 2-3:- Summary for Hydraulic Design of Under sluice*

Description	Value	Unit	Remark
Clear scour sluice width	1.10	m	Scour sluice to take at least 5*base flow
Number of Gates	2.00		
Scour sluice total width, Bt	2.20	m	
Gate Height, H	0.80	m	
Discharge through scour sluice	3.86	m <sup>3</sup> /sec	Q=CA(2gh) <sup>0.5</sup> , C = 0.6
The inverted elevation of scoring sluice	1713.00	m	

The size of under sluice is fixed to pass the discharge of 3.86m<sup>3</sup>/s in two bays, each bays has dimension of 1.1mx0.8m opening.

*(v) Intake/ Canal Head regulator:*

An Intake/Head Regulator is provided at the entrance to the off taking main canal at the diversion head works. The higher is the crest of the head regulator, the better it is, from

the point of view of the prevention of entry of silt in to the canal. The crest level of the Head Regulator has been kept at above the crest level of the under sluices, this would greatly help in preventing entry of any coarser silt from the river to the main canal. The head regulator should be capable of passing the design discharge when all the gates are open and the water level in the river is at pond level. The head regulator is design to pass the required discharge of 0.12m<sup>3</sup>/s. The head regulator/off taking canal has size of 0.6mx0.68m with bed elevation of 1713.4m.

- Discharge through Under sluice and Head regulator is given by Orifice formula

$$Q = 2/3 C_d \sqrt{2g} * LH^{3/2}$$

*Table 2-4:- Summary for Hydraulic Design of Intake/ Head regulator*

Description	Value	Unit	Remark
Depth of Canal at Head Regulator, D	0.4	m	
Width of Canal at Head Regulator, W	0.6	m	
Discharge through Head regulator	0.403	m <sup>3</sup> /sec	Q=CA(2gh) <sup>0.5</sup> , C = 0.88
The inverted elevation of Head Regulator	1713.4	m	

*(ii) Top and bottom width of weir*

According to the Bligh's formula, the basic section of the weir body can be determined as follows:

$$\text{Bottom width, } L = \frac{H+H_e}{\sqrt{\rho-1}}$$

$$\text{Top width, } B = \frac{H_e}{\sqrt{\rho-1}}$$

Where, H: Height of weir (m)

H<sub>e</sub>: Specific Energy head (sum of overflow depth and approaching velocity

head(m)

σ : specific weight of weir body (=2.2-2.3)

If the weir body is not submerged completely by the downstream water, (ρ) should be used instead of (ρ-1). The Weir is submerged during 50 years design flood case.

*Table 2-5:- Summary of Weir Section after Stability Analysis result*

<b>Description</b>	<b>Calculated Value</b>	<b>Unit</b>	<b>Adopted Value</b>
He: specific energy head	2.85	m	
P: Height of weir	1.1	m	
$\sigma$ : Specific weight of weir body	2.3		
Top Width, B	2	m	2
Bottom Width, L	3	m	3.54

*(iii) Divide wall:*

The divide wall will be provided to separate the main weir from the under sluice and allows a comparatively less turbulent pocket near the canal head regulator and this in turn helps in the entry of silt free water in to the canal. On all important works the width of the under sluice portion and the length of the divide wall are fixed on the basis of model experiments. If indicated by model studies, long submerged spurs are provided to keep any parallel flows far away from the protection works. The following guide lines are normally adopted for fixing the length of the divide walls:

- It shall not extend beyond the upstream end of head regulator
- Generally satisfactory results are obtained if it covers half width of the head regulator,
- Downstream divide wall shall extend up to the end of the downstream weir body.

A 0.6m masonry divide wall is provided where top level is above the pond level, extending up to the Middle of head regulator on the upstream and up to end of downstream weir has been provided.

The height of the divide wall is fixed based on equal level of the weir, but some part of the wall shall extend up to the level of under sluice pier level for carrying operation slab.

*(iv) Hydraulic Jump Calculation*

The length of wing walls is determined based on the length of Jump, and it will be calculated as shown below.

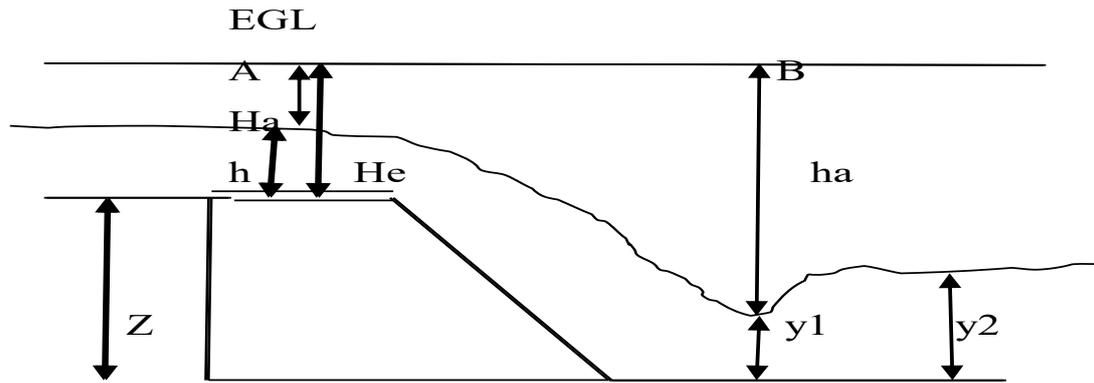


Figure 2-3: Flow over weir body.

- Weir crest length = L (m)
- Weir height = z (m)
- Pre-jump depth =  $y_1$
- Post -jump depth =  $y_2$

Neglecting losses between point A and B and considering similar datum

$$z + H_e = y_1 + h_a$$

$$q = \frac{Q}{L}$$

$$h_a = \frac{q^2}{2 * g * y^2}$$

$$V_1 = \frac{q}{y_1}$$

$$F_r = \frac{V_1}{\sqrt{g y_1}}$$

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

So, from calculation and graph on USBR hydraulic jump length (L) using trial and error the final result is presented in the following table.

Table 2-6:- Hydraulic Jump Calculation summary

REQUIRED DATA	VALUE	UNIT
Up stream river bed level (U/S RBL) =	1713.000	m
Weir height (H) =	1.080	
Weir crest level (WCL) =	1714.080	
Thus tail water depth (TWD) in meter(D=Y2)	2.057	m
Design discharge, QD =	196.340	m
Effective Length $L_{eff}$ =	24.00	m
Down stream river bed level (D/S RBL) =	1713.000	m
$H_e = (Q/Cle)^{2/3}$	2.850	m
design discharge per meter width (q)	8.181	m <sup>3</sup> /s/m
$H_a = (Va^2/2g) = (Q/Y1)^2/2g$	0.221	m
Hd	2.598	m

**Trial-1** Applying Bernoulli's Equation b/n u/s and at the toe of spillway >>>>  $E_1 = W_2$

$$Q = AV_1 \gg \quad q/Le = AV_1 = V_1 = q/Y_1$$

$$H + H_e = Y_1 + (V_1^2/2g)$$

$$3.93035584933715 = Y_1 + ((q/Y_1)^2/2 * g)$$

$$Y_1 + ((q/Y_1)^2/2 * g) = 3.930$$

$$Y_1 = 1.0973$$

$$3.930$$

Incoming froude number ( $Fr_1$ ) =  $V_1 / \sqrt{gy_1}$

$$V_1 = q/Y_1 \quad 7.455098468$$

$$Fr_1 = 2.27$$

$$H_{a1} = 2.8327$$

from  $Y_1/Y_2 = 1/2 * (-1 + \sqrt{1 + 8Fr_1^2})$

$$Y_2 = 3.02$$

$$V_2 = 2.709$$

$$H_{a2} = 0.374$$

$$\text{Head loss (HL)} = (Y_2 - Y_1)^3 / (4Y_1 Y_2) = 0.536$$

CHECK FOR THE DIFFERENCE(D) B/N Y2 AND TWD i.e (Y2-TWD)= 0.9630

**Trial-2**

Assuming the depth of apron is lowered by  $D_1 = 0.3$

Applying Bernoulli's Equation b/n u/s and at the toe of spillway

$$H + H_e = D/S RBL - 0.6 + Y_1 + (V_1^2 / 2g)$$

$$4.2304 \quad D/S RBL - 0.6 + Y_1 + ((q/Y_1)^2 / 2 * g)$$

$$Y_1 + ((q/Y_1)^2 / 2 * g) = 4.230$$

$$Y_1 = 1.0329$$

$$4.230$$



Incoming froude number ( $Fr_1$ ) =  $V_1 / \sqrt{gy_1}$

$$V_1 = 7.920$$

$$Fr_1 = 2.5$$

$$Ha_1 = 3.1970$$

from  $Y_1/Y_2 = 1/2 * (-1 + \sqrt{1 + 8Fr_1^2})$  TWD

$$Y_2 = 3.15$$

$$V_2 = 2.593$$

$$Ha_2 = 0.3428$$

$$\text{Head loss (HL)} = (Y_2 - Y_1)^3 / (4Y_1Y_2) \quad 0.7$$

$$\text{Difference B/n } Y_2 - Y_3 \quad 0.8$$

**Position of the jump**

The discharge per unit width =  $q = CdH^{3/2}$

Where  $H = U/S TEL - WCL = 2.819$   
 $q = 8.044317476$   
 $Cd = 1.7$

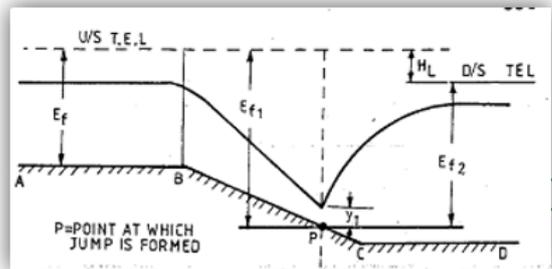
$$U/S TEL = 1716.899$$

$$D/S TEL = 1716.197$$

$$HL = 0.701$$

$$E_{f2} = 3.7$$

$$\text{Location of the Jump} = D/S TEL - E_{f2} \quad 1712.50$$



Length of D/S apron

$$L = 5 - 6 * (Y_2 - Y_1)$$

$$L = 6 * (3.15 - 1.1)$$

$$L = 12.3m$$

According to Bligh's

$$L = 2.21C\sqrt{(HL/10)}$$

Where C=7 Based on foundation material in our case the foundation is sound rock deep after the weathered part

$$L = 2.21 * 7\sqrt{(7/10)}$$

$$L = 2.21C\sqrt{(HL/10)}$$

$$L = 4.1m$$

Since the foundation is rock take the average length  $L = (12.3+4.1)/2 = 8.2m$

**So the downstream apron length will be 9m**

U/S Apron length considering the river foundation take A Nominal length of **1m**

*(v) Determination of scour/ Cut off Depth*

The uplift pressure of seepage water through the bed of the weir body tends to overturn the weir. The passage of water towards downstream through the bed of the weir body tends to bring piping effect. This will cause the silt particle to exit in the downstream of the weir and form a hollow section, which causes the failure of the weir, u/s and d/s cutoff is provided to prevent this effect. The upstream and downstream cut offs should generally be provided to cater for scours up to 1.5R and 2R respectively where R is the depth of scour below water level and is given by:

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

Where  $f = 1.76 * \sqrt{d}$ , Lacey's silt factor

d = is average particle size in (mm)

q = is discharge per unit length

- *Upstream cut off level = upstream HFL-1.5R*
- *Downstream cut off level = Downstream HFL-2R*

Description	Value	Unit
River Bed Level	1713	m
U/S HFL	1716.68	m
D/S HFL	1715.85	m
River bed formation	Large Bolder	
Unit Discharge, q	8.181	m <sup>3</sup> /s/m
River bed material average diameter	183	mm
Lacy's silt factor, $f=1.76*d(0.5)$	23.81	
Normal Scour Depth, $R=1.35*(q^2/f)^{1/3}$	1.91	m
U/S Cut off depth	1.5	m
D/S Cut of Depth	1.6	m

*When the river bed formation goes to rock formation during excavation it is better to be anchored.*

*(vi) Upstream and downstream weir protection works*

Protection work is provided in order to relieve the uplift Pressure and to protect the piping of the underlying soil as well as the dislocation of particles due to subsurface flow. Natural aprons normally protect the upstream and downstream floor of the weir.

***D/s impervious floor Length (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:**

$$H_s = \text{crest level} - \text{bed level}$$

**2) Maximum flood case:**

$$H_s = \text{U/s HFL} - \text{D/s HFL}$$

$$L_d = 2.2 * C_b * \sqrt{\frac{H_s}{10}} L_d$$

Bligh's constant,  $C_b$  is depending on the type of the foundation.

***U/S Impervious Floor Length, (Lu)***

The u/s impervious floor,  $(L_u) = LT - (2 * U/s \text{ cut off} + \text{bottom Width} + L_d + 2 * d/s \text{ cut off})$

Description	Value	Unit
River Bed Level u/s	1713.00	m
River Bed Level d/s	1713.00	m
Weir Crest Level	1714.10	m
U/S HFL	1716.68	m
D/S HFL	1715.85	m
Weir bed width	3.54	m
Weir Height	1.1	m
River bed formation	Large Bolder	
U/S Cut off depth	1.50	m
D/S Cut off depth	1.60	m
D/S Water Depth/Tailwater	2.06	m
U/s Water Depth	3.7	m
Head Loss (Pond Level) = Weir Crest Level - D/s River bed Level	1.1	m
(HFL) = U/s HFL - D/s HFL	0.823	m
Adopted HL (Maximum)	1.1	m
D/S Apron Length, L1 Adopted	9	m
U/S Apron length, $L_2 = LT - (L_1 + 2 * d_1 + 2 * d_2 + B)$	0.00	m
U/S Apron Length, Adopted	1.00	m
No of d/s apron partitions (different thickness apron)	3	m
1st apron length	3	m
2nd apron length	4	m
3rd apron length	2	m

***(vii) Breast Walls***

In the under sluice bays, the required discharge during the flood shall pass only with a small opening. Therefore, to reduce the height of gate, the breast wall has been provided in all the under-sluice bays. The bottom of the breast wall has been kept at the top of the required opening and the top of the breast wall has been kept above the HFL for the design flood. Similarly, in the head regulator also, provision of the breast wall has been made to reduce the height of gates. The bottom of the breast wall has been kept at the top of the opening required to pass the full supply discharge at the pond level in the river, and the top has been kept above the HFL for the design flood.

***(viii) U/Stream and Downstream Flood Protection***

Guide banks are provided in both banks in order to train water to flow axially through the trough without flanking the structure. In addition, the guide banks are provided in pairs and HFL and free board govern the top level of the banks. Hence, the downstream and the upstream guide banks are treated separately.

The general consideration in design of guide walls is that the masonry section of the guide wall must have enough self-weight to resist the thrust due to earth pressure and water pressure for its rear without overturning, sliding, tension and compressive stress developed within the body of the structure.

The height of the flood jump in the downstream governs the height of the guide wall with some free board provided.

## **2.8 Structural design of Weir and Appurtenant structures**

Structural design consists of the following:

- a. Stability of the weir,
- b. Design of capping of weir,
- c. Design of Operation Slab,
- d. Design of divide wall,
- e. Design of Breast walls,
- f. Stability of abutments and retaining walls,

### **a) Stability of the weir body**

Once a section of the weir has been designed, it has to be analyzed and checked, whether it satisfies the safety requirements. Gravity method (or two dimensional methods) has been used for the analysis of the weir. In this method of analysis, the weir is considered as a two dimensional structure. A unit length of the weir is considered for the analysis. The weir is assumed to consist of a series of vertical cantilevers of unit length and fixed at the base. These cantilevers are assumed to be independent of one another. The loads acting on the cantilevers are transferred to the foundation through the cantilever action. The stability of these cantilevers will be checked against all possible modes of failure for all possible forces acting on it.

The stability analysis will be carried out taking a unit length of the weir and taking into account the geology of the river bed. Therefore, the most dominant forces identified are

- *Static water pressure of the surface water*
- *Uplift water pressure*
- *Soil reaction at the weir base*
- *Friction forces at the base which develop to balance the horizontal forces*
- *Weight of weir and water wedges*

*Usually in structural analysis of weirs the dynamic force is neglected, since water behind the weir is built up gradually, and the uplift pressure which results from the arrival of a new wave does not develop instantly.*

The following procedure is used for checking the stability of the weir.

- i. All the forces, vertical and horizontal, acting on the weir are determined
- ii. Find the algebraic sum of all the horizontal forces( $\Sigma H$ ) and vertical forces( $\Sigma V$ )
- iii. Determine the moments of all the forces components about the downstream edge or toe. Find the algebraic sum ( $\Sigma Mr$ ) of resisting moments and algebraic sum ( $\Sigma Mh$ ) of overturning moments. Also determine the net moment ( $\Sigma M$ ) about the toe. Thus,
 
$$\Sigma M = \Sigma Mr - \Sigma Mh$$
- iv. Determine the distance  $x$  of the point where the resultant  $R$  strikes the base.
 
$$X = \Sigma M / \Sigma V$$
- v. Determine the eccentricity  $e = 0.5B - x$ , ensure that the eccentricity is within the middle one third of the base width.
- vi. Determine the Factor of safety against overturning ( $\Sigma Mr / \Sigma Mh$ )
- vii. Determine the factor of safety against sliding ( $\mu \Sigma V / \Sigma H$ )
- viii. Determine the vertical stresses at the toe and heel of the weir

The overall stability analysis involves checking for the design margin for eccentricity, sliding, overturning and bearing capacity. The eccentricity of loads worked out is found to be within one third of the base implying thereby that the pressure at the base shall always be positive and there shall be no tension.

1. ***If Factor of safety against overturning,  $F_o = \frac{\Sigma M (+)}{\Sigma M (-)} > 1.5$ , the structure is safe against overturning.***

2. *If Factor of safety against sliding,  $F_s = \mu * \frac{\Sigma V}{\Sigma H} > 1.5$ , the structure is safe against sliding.*

3. *Tension for checking*

$$X_{ave} = \frac{(\Sigma M(+)) - \Sigma M(-)}{\Sigma V}$$

$$\text{The eccentricity, } (e) = e = \left(\frac{B}{2} - X_{ave}\right)$$

*For eccentricity (e)  $< \frac{B}{6}$ , shows the resultant lies within the middle third hence no tension developed.*

4. *Bearing pressure development*

The weir may fail by the failure of its materials due to compression or crushing. Thus the compression stress developed must not exceed the allowable stress. Maximum base bearing pressure developed due to the weir section.

$$\text{Max. Base Pressure} = \frac{\Sigma V}{B} * \left(1 + \frac{6 * e}{B}\right)$$

Stability analysis will be done for two critical conditions, the first case is minimum river flow case and the second case is at maximum flow of the river.

***Case 1: - When Water is at Pond Level***

Forces acting in base flow condition are Self-weight, Uplift pressure, silt load and upstream water load. The unit weight for concrete is 23KN/m<sup>3</sup> and for water is 9.81KN/m<sup>3</sup>. All pressures are estimated by assuming unit width. The load is estimated dividing the weir in to several parts to get best result. The total load acting on the weir is presented in the following figure. Table below shows the computed values and factors of safety are checked accordingly.

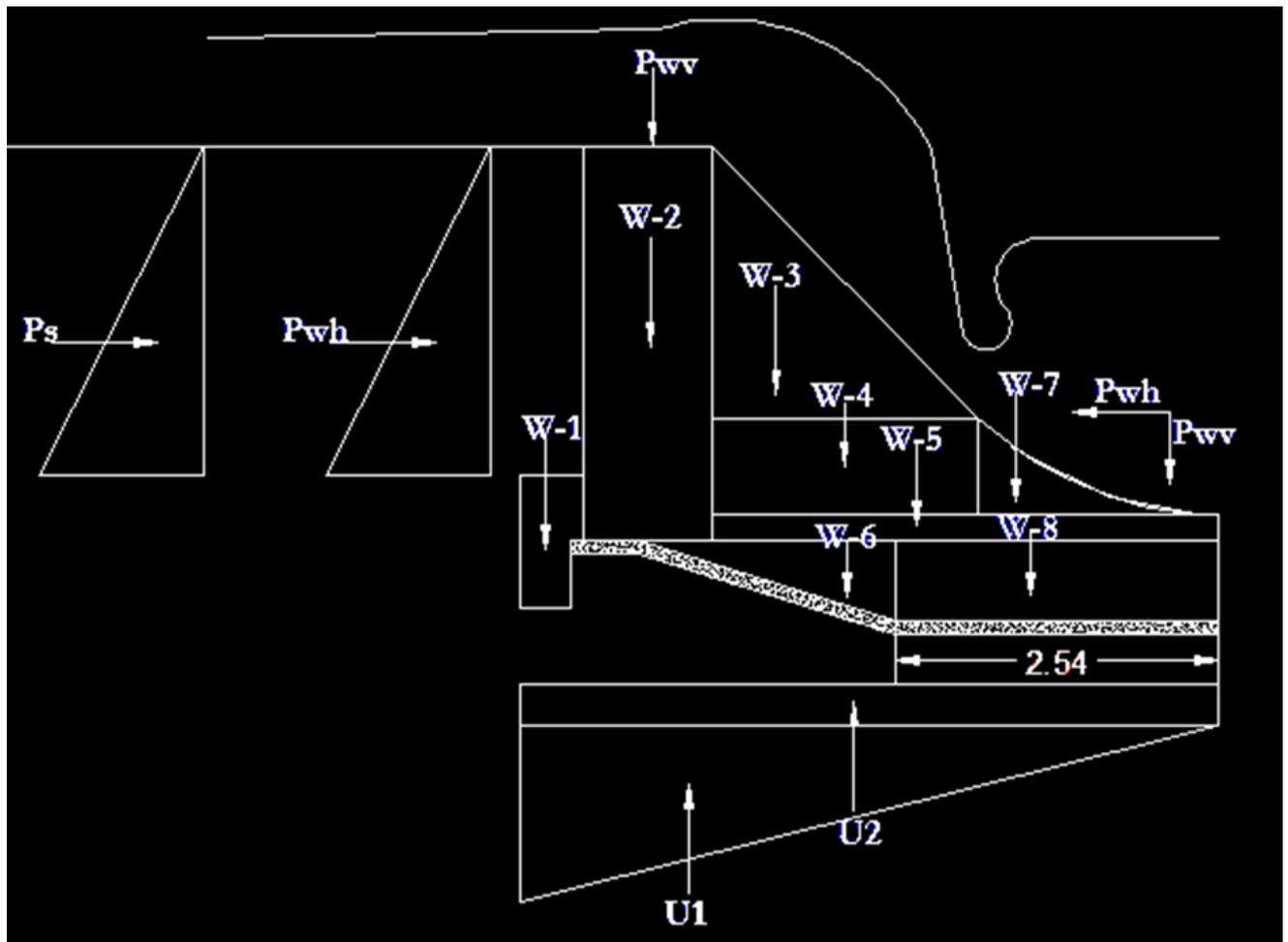


Figure 2-4:- Load Distribution during Minimum Water level

Table 2-7:- Moment and Load Calculation result in Minimum water level

FOR STATIC CASE							
Name of forces	Symbol	AREA(m2)	Magnitude of forces (KN)		Lever arm in m	Moment at "o"(KN.M)	
			Vertical	Horizontal		Resisting	Disturbing
<b>1. Vertical Forces Down ward</b>							
<b>WEIGHT OF WEIR BODY</b>	W1	0.75	17.25		3.75	64.69	
	W2	1.60	36.80		2.50	92.00	
	W3	1.10	25.30		1.33	33.73	
	W4	1.00	23.00		1.00	23.00	
	W5	0.25	5.75		3.25	18.69	
	W6	1.50	34.50		1.00	34.50	
	W7	0.00	0.00		0.63	0.00	
	W8	0.00	0.00		1.27	0.00	
<b>2. Silt pressure</b>							
Psilt				-2.11	0.36		-0.76
<b>4. Hydro static preasure</b>							
PW u/s-h				-5.72	0.36		-2.06
<b>5. Uplift pressure</b>							
U			-15.89		2.00	2.00	-31.78
sum			<b>126.71</b>	<b>-7.83</b>	<b>17.45</b>	<b>268.61</b>	<b>-34.60</b>
Checking for sliding	$\eta \frac{\sum F_v}{\sum H_F} > 1.5$	<b>9.71</b>	ok!				
Checking for overturning	$e = \frac{\sum M(+)}{\sum M(-)} > 1.5$	<b>7.76</b>	ok!				
Tension	$e = \left  \frac{\sum M}{\sum F_v} - \frac{L}{2} \right  < \frac{L}{6}$	<b>0.34681352</b>	ok!	<b>0.5</b>			
Checking for Bearing Capacity	$\Sigma V/B * (1+6*e/B)$						
Maximum base pressure		<b>71.5319184</b>					
Minimum base pressure		<b>12.9399482</b>	<	350KN/m2			

Case 2: - When Water level is at High Flood Level

In this condition of analysis there are additional loads, i.e. water level is above weir, Uplift pressure considers tail water depth and tail water pressures. The detail calculation is conducted using excel and the result is as follows.

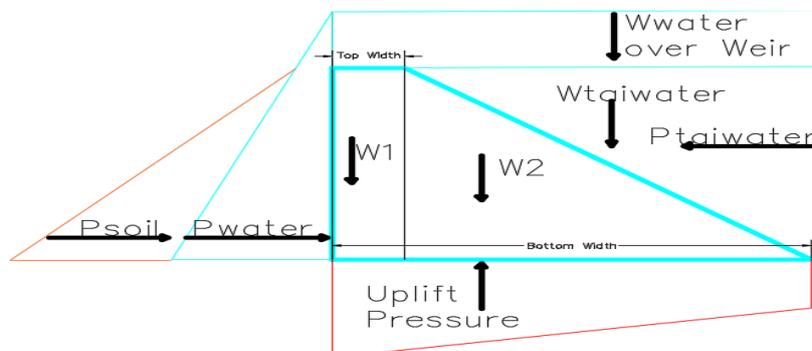


Figure 2-5:- Load Distribution during Minimum Water level

Table 2-8:- Moment and Load Calculation result in Minimum water level

FOR DYNAMIC CASE							
Name of forces	Symbol	AREA(m <sup>2</sup> )	Magnitude of forces (KN)		Lever arm in m	Moment at "o"(KN.M)	
			Vertical	Horizontal		Resisting	Disturbing
<b>1. Vertical Forces Down ward</b>							
	<b>W1</b>	<b>0.75</b>	<b>17.25</b>		<b>3.75</b>	<b>64.69</b>	
	<b>W2</b>	<b>1.6</b>	<b>36.80</b>		<b>2.50</b>	<b>92.00</b>	
	<b>W3</b>	<b>1.1</b>	<b>25.30</b>		<b>1.33</b>	<b>33.73</b>	
	<b>W4</b>	<b>1</b>	<b>23.00</b>		<b>1.00</b>	<b>23.00</b>	
	<b>W5</b>	<b>0.25</b>	<b>5.75</b>		<b>3.25</b>	<b>18.69</b>	
	<b>W6</b>	<b>1.5</b>	<b>34.50</b>		<b>1.00</b>	<b>34.50</b>	
	<b>W7</b>		<b>0.00</b>		<b>0.63</b>	<b>0.00</b>	
	<b>W8</b>		<b>0.00</b>		<b>1.27</b>	<b>0.00</b>	
<b>1. Water pressure(U/S)</b>							
	<b>PV1</b>		<b>25.48</b>		<b>4.26</b>	<b>108.56</b>	
	<b>PH1</b>			<b>-5.72</b>	<b>0.36</b>		<b>-2.06</b>
<b>2. Silt pressure</b>							
	<b>Psilt</b>			<b>-2.11</b>	<b>0.36</b>		<b>-0.76</b>
<b>4. Hydro dynamic prea (D/S)</b>							
	<b>Pwh</b>		<b>39.61</b>		<b>0.69</b>	<b>27.27</b>	<b>27.27</b>
	<b>Pwv</b>		<b>57.21</b>		<b>1.03</b>	<b>59.09</b>	<b>59.09</b>
	<b>Pv D/S(on apron)</b>		<b>318.30</b>		<b>0.89</b>	<b>281.70</b>	<b>281.70</b>
<b>5. Uplift pressure</b>							
	<b>U1</b>		<b>-92.84</b>		<b>1.50</b>		<b>-139.26</b>
	<b>U2</b>		<b>-7.70</b>		<b>2.00</b>		<b>-15.40</b>
	<b>sum</b>		<b>482.66</b>	<b>-7.83</b>	<b>25.82</b>	<b>743.23</b>	<b>210.59</b>
Checking for sliding	$\eta \frac{\sum F_v}{\sum H_f} > 1.5$	<b>36.99</b>	<b>ok!</b>				
Checking for overturning	$e = \frac{\sum M(+)}{\sum M(-)} > 1.5$	<b>3.53</b>	<b>ok!</b>				
Tension	$e = \left  \frac{\sum M}{\sum F_v} - \frac{L}{2} \right  < \frac{L}{6}$	<b>0.47616</b>	<b>ok!</b>	<b>0.5</b>			
Checking for Bearing Capacity	$\Sigma V/B*(1+6*e/B)$						
Maximum base pressure		<b>314.106</b>					
Minimum base pressure		<b>7.67002</b>	<b>&lt;</b>	<b>350KN/m2</b>			

## b) Weir capping

To protect the weir from the wear and tear due to boulders carried by the flood, a capping of RCC C-25 will be provided on the outer face of the weir. The weir cap is assumed as a beam, fixed at

the base and simply supported at the top of the weir. 25cm thick capping in C-25 concrete with suitable reinforcement has been provided. The minimum reinforcement shall be

$$\rho_{min} = \frac{0.5}{f_{yk}} = \frac{A_s}{bd'}$$

$$A_s = 0.5 * \frac{bd}{f_{yk}}$$

Where,  $A_s$  = Area of reinforcement,

$b$  = unit width,

$d$  = thickness of slab

$f_{yk}$  = characteristics yield strength of reinforcement, 276 for dia.  $\leq 16$ mm and 400 for dia.  $> 16$ mm.

**Hence provide single reinforcement of  $\Phi 12$  @ C/C 200mm for main bar and  $\Phi 12$  @ C/C 200mm for distribution bar.**

### c) Divide wall

The divide wall has been designed as a cantilever beam to resist both hydrostatic pressure and sediment loads. The overall stability of divide wall will be checked against overturning, sliding and also against the tension at the bottom of foundation. The critical forces are when one side of the divide wall is under force of silt while the other side is free. The divide wall is also used as a foot of operation slab for the access of the Gate in the head regulator and under sluice. The divide wall is recommended to be 0.6m thick masonry wall.

### d) Operation slab and Breast wall

Vertical gates will be provided for the under sluice and as well as for the head regulator. These gates are slide over the breast wall-using spindle during opening and closing, the operation shall be on operation slab.

For easy operation of these gates, operation slab will be provided. The size of the operation slab is fixed from the point of construction and its free movement. After the analysis, the reinforcement of the operation slab and breast wall will be provided and presented in the drawing part. The

operation slab and breast wall is recommended to be reinforced concrete, the detail is presented in the drawing.

### e) Stability of Abutments and Retaining Walls

Guide banks are constructed to train water to flow axially through the trough without flanking the structure. In addition, the guide banks are provided in pairs and HFL and free board govern the top level of the banks. The top elevations of both upstream and downstream wing walls were determined in the hydraulic design sections.

The general consideration in design of guide walls is that the masonry section of the guide wall must have enough self-weight to resist the thrust due to earth pressure and water pressure for its rear without overturning, sliding, tension and compressive stress developed within the body of the structure. The height of the flood jump in the downstream governs the height of the guide wall with some free board provided. Since the jump depth is higher than the tail water depth the downstream retaining wall is based on the level of the jump depth and length.

Gravity type wing/retaining walls are recommended to be constructed with stone masonry embedded in cement mortar.

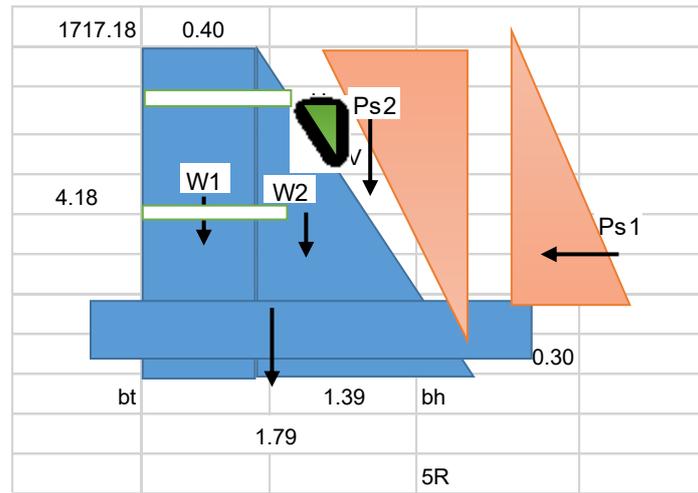
#### Conditions:

Retaining or wing walls are expected to be subjected to critical imbalances from side soil pressure under no flow case for the downstream wing and when water level is at WCL for the upstream walls. Otherwise, during high flood cases this condition is on safer side as soil pressure and water pressure balance each other. Thus stability is checked under these critical conditions as follows.

The stability analysis will be carried out taking a unit length of the structure and taking into account the geology of the area. Therefore, the most dominant forces identified are

- *Static water pressure of the surface water*
- *Uplift water pressure*
- *Soil reaction at the base*
- *Friction forces at the base which develop to balance the horizontal forces*
- *Weight of structure and water wedges*

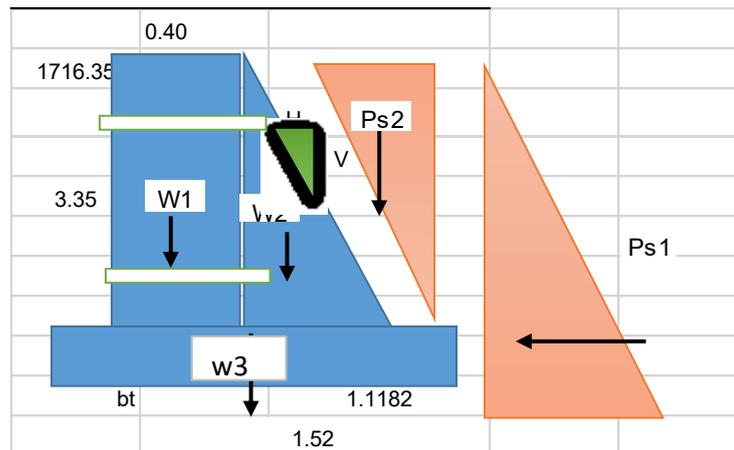
U/S Wing Wall



Stability Analysis of U/s wing wall			
Top width	B1=	0.40	m.
Bottom width	B =	1.79	m.
Height of wall	Hs=	4.18	m.
Foundation Thickness	t=	0.30	m.
$\gamma_m$ of masonry	$\gamma_m$ =	23	KN/m <sup>3</sup>
$\gamma_s$ of silt(Fill)	$\gamma_{silt}$ =	17	KN/m <sup>3</sup>
$\gamma_{co}$ of Concrete	$\gamma_{RCC}$ =	25	KN/m <sup>3</sup>
Angle of Repose	$\phi$ =	30	Deg.
$K_a = (1 - \sin\phi) / (1 + \sin\phi)$	K=	0.33	
bt =D/2-D		0.15	m
bh =10-15cm		0.15	m

Bearing capacity of Foundation material gravel material		200 KN/m <sup>2</sup>					
Item Description	Forces(KN.m)				Lever arm (m.)	Moment about toe(KN.m)	
	Vertical		Horizontal			Resisting	Overturning
	+ve	-ve	+ve	-ve			
1. Vertical force							
1.1. Self weight(W1)	38.43				0.2	7.69	
1.2. Self Weight(W2)	66.90				0.86	57.82	
1.3. Silt preasure(Ps2)	49.45				1.33	65.69	
2. Horizontal force							
2.2. Active preasure (Ps1)				-16.483	1.39		-22.95
2.3. Active preasure (Ps1)							
<b>TOTAL</b>	<b>154.79</b>	<b>0.00</b>	<b>0.00</b>	<b>-16.48</b>		<b>131.19</b>	<b>-22.95</b>
D) Overturning Stability	Fo=SMr/SMo	5.72	Safe	1.50			
II) Sliding Stability	Fss=SH/SV	9.39	Safe	1.5			
III) Overstressing Stability	ΣM=Mr+Mo	108.24					
$e = \left  \frac{\sum M}{\sum F_v} - \frac{L}{2} \right  < \frac{L}{6}$	SM/SFv	0.69927					
	L/2	0.90					
	e= 0.1970		safe	0.30			
IV) Bearing Capacity	P=SV/B(1+6e/B)						
Max compression stress at the toe	Pmax = 143.29		Safe	200	KN/m		
Tension develop at the heel	P = 29.41		Safe	200			

D/S Wing wall



Bearing capacity of Foundation material gravel material		200 KN/m <sup>2</sup>					
Item Description	Forces(KN.m)				Lever arm (m.)	Moment about toe(KN.m)	
	Vertical		Horizontal			Resisting	Overturning
	+ve	-ve	+ve	-ve			
1. Vertical force							
1.1. Self weight(W1)	30.86				0.2	6.17	
1.2. Self Weight(W2)	43.14				0.77	33.33	
1.3. Silt pressure(Ps2)	31.88				1.15	36.52	
2. Horizontal force							
2.2. Active pressure (Ps1)				-10.628	1.12		-11.88
2.3. Active pressure (Ps1)							
<b>TOTAL</b>	<b>105.88</b>	<b>0.00</b>	<b>0.00</b>	<b>-10.63</b>		<b>76.02</b>	<b>-11.88</b>
I) Overturning Stability	Fo=SMr/SMo	6.40	Safe	1.50			
II) Sliding Stability	Fss=SH/SV	9.96	Safe	1.5			
III) Overstressing Stability	ΣM=Mr+Mo	64.14					
$e = \left  \frac{\sum M}{\sum F_v} - \frac{L}{2} \right  < \frac{L}{6}$	SM/SFv	0.60579					
	L/2	0.76					
	e=	0.1533	safe	0.25			
IV) Bearing Capacity	P=SV/B(1+6e/B)						
Max compression stress at the toe	Pmax =	111.99	Safe	200	KN/m		
Tension develop at the heel	P =	27.49	Safe	200			

## 2.9 Backwater effect

Due to the new barrier it is obvious that the raise in flood height will cover extra banks and this was considered in the design to protect effects by flood protection dyke and keep flood height of 1.1m above crest not to result any upstream damages. The river morphology is more of valley and thus no side flooding is expected.

## 2.10 Temporary river diversion during construction

For this particular project temporary river diversion is normally required to facilitate construction of the head work structure (mainly the retaining wall and other works located on the river bed). Depending on the magnitude of river flows the design and construction of diversion works can be difficult and expensive. Construction of head works and related temporary river diversion works are weather dependent and constitute a key activity in any project construction schedule.

The following factors influence the design of temporary river diversion works:

- Duration of construction of in-river structures.

- Vulnerability to overtopping (masonry, concrete versus embankment works).
- Stream flow characteristics.
- Magnitude and duration of floods during construction period.

The climate in the project area is characterized by two distinct seasons, a wet season with high flows, and a dry season with low flows. The dry season provides the best conditions for construction of in-river works as the flows to be handled are much smaller than during the wet season. Accordingly, it would be advantageous to schedule construction of the intake and related temporary diversion works for the dry season. For the head work structure it possible to complete all vulnerable works within a single dry season.

### 3 IRRIGATION AND DRAINAGE DESIGN

#### 3.1 General

The irrigation design project needs to be simple so that users can understand and participate in the operation and maintenance. Complex designs are avoided as much as possible. Designing cost effective structures is taken as one of the approaches in this study and design work. The irrigation system and structures are designed to use the water as efficient as possible by minimizing the losses in conveyance, distribution and application system.

#### 3.2 Command Area

The command area is bounded by Wataba Baddesa River in the East, Elele Walena kebele settlement area in the West. The main canal follows the possible high-level ground and gentle canal slope is aligned to irrigate more area around the command area.

Slope is the most important site characteristics as it influences the suitability to irrigation and methods of irrigation and type and kinds of farm operations. In this regard, the majority of the irrigation command area is flat and gently sloping, still other slope classes constitute limited proportions.

In the project area the total area covered by the study is more than 94ha but due to water head and infrastructure work the net area is limited to about 63ha.

#### 3.3 Irrigation System Design

Surface irrigation is the most common method of irrigation in the world. Soils with high infiltration rate are commonly not suitable to surface irrigation, because the distribution of irrigation water is difficult to maintain without short furrows. As a result, loamy soils may be considered as marginally suitable, despite the potential optimum nutrient and moisture holding capacity.

As slopes increase to 20%, so too does the need for soil conservation measures to accompany irrigation; on slopes greater than 12% land forming for surface irrigation is seldom economically viable. The risks of erosion are potentially greater on increasingly sloping land so a sufficient minimum soil depth of 1.0m on slopes between 8% and 12% must be maintained to allow maximum root and soil structural development and to enhance infiltration and reduced run-off.

Vertisols are more unstable than other soils, so terracing is not feasible on slopes above 6%. On slopes up to 6% and so long as soil depth exceeds 1.0m, land can be safely formed to gently sloping benches with gentle and vegetated risers.

Where groundwater is high, the pressure irrigation may be preferable because percolation and runoff and hence the rate of groundwater rise can be minimized more easily than the case with surface irrigation, and any need for drainage can be deferred. Where drainage exists or planned alongside the irrigation development, the choice of irrigation method is not critical so long as the drainage system can handle the extra runoff water generated by surface irrigation.

In Wataba Baddesa Small Scale Irrigation Project, the dominant soil type is Sandy loam and the dominant command slope is greater than 12%. Surface Irrigation method is selected considering the traditional practice of the community and considering the economy.

### **3.3.1 Layout system**

In preparing the alignment of the conveyance system of Wataba Baddesa SSIP the following issues have been taken in to consideration.

- The alignment of most of the canal system is made to follow the existing traditional conveyance and distribution system as much as possible.
- The length of canal, mainly the tertiary canals and field channels, is made to be as economical as possible in such a way that the maximum area is irrigated with least length of channel and a good balance of cut/fill is be exercised.
- Curves have been made to be avoided as much as possible; however, in cases when a curve becomes inescapable, it has been made to be as smooth/gentle as possible the radius of curvature being made proportional to the discharge
- The average furrow length is made based on soil type and slope.
- The number and length of canals are tried to be minimized not to waste a valuable and productive land as land is scarce.
- Boundaries of the tertiary units are determined based on drainage lines and natural boundary of farm.

The irrigation system comprises four major components: the Main canal, Secondary canals, the on-farm distribution, and the drainage systems. The total Net command area is about 63 ha. The Main canal is proposed to irrigate required the left command area. The overall system is sub

divided in to 1 Main Canal, 5 Secondary canals and 22 tertiary blocks as indicated in the topographic map of the project farm system layout. The tertiary canals are aligned almost as contour channels the furrow length is kept almost 100 m on the average with average feeder ditches length of 100 m.

Before commencement of design of entire irrigation structure, the detailed irrigation and drainage system layout were prepared. This layout contains information on field configuration, canal networks, natural drainage channels network, field drains, access roads and service roads, etc. Key dimensions for all layout components and irrigation and drainage infrastructure are determined.

Ground level profiles of canal systems are also taken and analyzed in accordance with the acceptable field layout. The detail levels are then used for the longitudinal and cross section of the canals and drains and to determine the levels of canal control and regulating structures.

The design of the canal is concerned with the determination of the cross sectional dimension of the canal to convey the required discharge needed to meet the peak requirement of crops grown in the entire command area. The whole section of the canal is designed for adequate capacity, to provide sufficient capacity.

Main canal and secondary canals are designed to be lined canals and others are designed as unlined section, based on the criteria such that the canal is non-silting when conveying sediment-laden water, and non-scouring when conveying silt-free water. The canal flows were classified as steady and uniform and were designed using the Manning’s equation for open canals.

$Q = A * V$ .....Eq1

Where: Q = Discharge (m<sup>3</sup>/s)

A = Average of cross-section (m<sup>2</sup>)

V = Average velocity (m/s)

The velocity of flow was computed using Manning Formula as follows.

$V = \frac{1}{n} * R^{2/3} * S^{\frac{1}{2}}$ .....Eq2

Where: V = Average velocity (m/s)

$n$  = Rugosity coefficient, depends on canal material roughness

$R$  = Hydraulic mean radius (m)

$S$  = Bed slope of canal

### 3.3.2 Water Distribution system

Water is conveyed through the main canal from the river and distributed by division box in field. Each block has got water by field canals. The flow through the main canal is continuous type. The tertiary canals are branched from secondary canal in turn the field canals are branched from tertiary canals and offtake from main canal. Flow through tertiary is rotational base. For easy distribution division boxes are provided at the junction point of each tertiary canal head and simple turnouts arise from tertiary canals and main canals to divert water to the field canals which are the final minor canal in the system.

In surface methods of irrigation, water is applied directly to the soil surface from a pond located at the upper reach of the field by gravity. A flow is introduced at high point or along the high ridge of the field and allowed to cover the field by gravity.

The rate of coverage of land depends almost entirely on the quantitative difference between the inlet discharges and cumulative infiltration rate. Two general requirements of prime importance to obtain high efficiency in surface methods of irrigation are properly constructed water distribution system to provide adequate control of water to the fields to permit uniform distribution.

The common method of surface irrigation is furrow type. The furrow method of irrigation is used to irrigate row crops with furrows developed between the rows in the plan and cultivation process. Water in the furrows contacts only 1/2 to 1/5 of land surface, thus reducing puddling and crusting of soils and renders early cultivation. Water infiltrates into the soil and spreads laterally. It is more suitable method of surface irrigation for crops sensitive to pounded water. Furrows are most commonly made down the slope but when land slope exceed the safe limit soil erosion of soil appears, they are constructed nearly on contour or obliquely. Similarly, when rainwater is to be conserved, furrows act as an effective means to catch and conserve the rainfall. When irrigation water is very scarce, the system of alternate/skip furrow irrigation, results in considerable saving of water.

Surface (Furrow method) is the most common form of irrigation around the world and hence it is recommended for this particular project.

### Furrow Irrigation Design Considerations:

Efficient irrigation by the furrow method is obtained by selecting proper combinations of: shape, length, slope of furrows, and suitable size of the irrigation stream and duration of the water application.

#### (i) Furrow shape

The furrows are designed to have good permissible velocity with shape of either V or U-cross-sections as shown in the Fig. below. This design approach is based on the Recommendation of FAO-Paper-volume-II in module-7. The first section is common for sandy texture of deep and narrow furrows while the second is common for clay texture of wide and shallow furrows. Hence in Wataba Baddesa Small Scale Irrigation Project the soil is Sandy loam, the first cross-section type can be applied. The depth,  $d$ , varies from 10-30 cm.

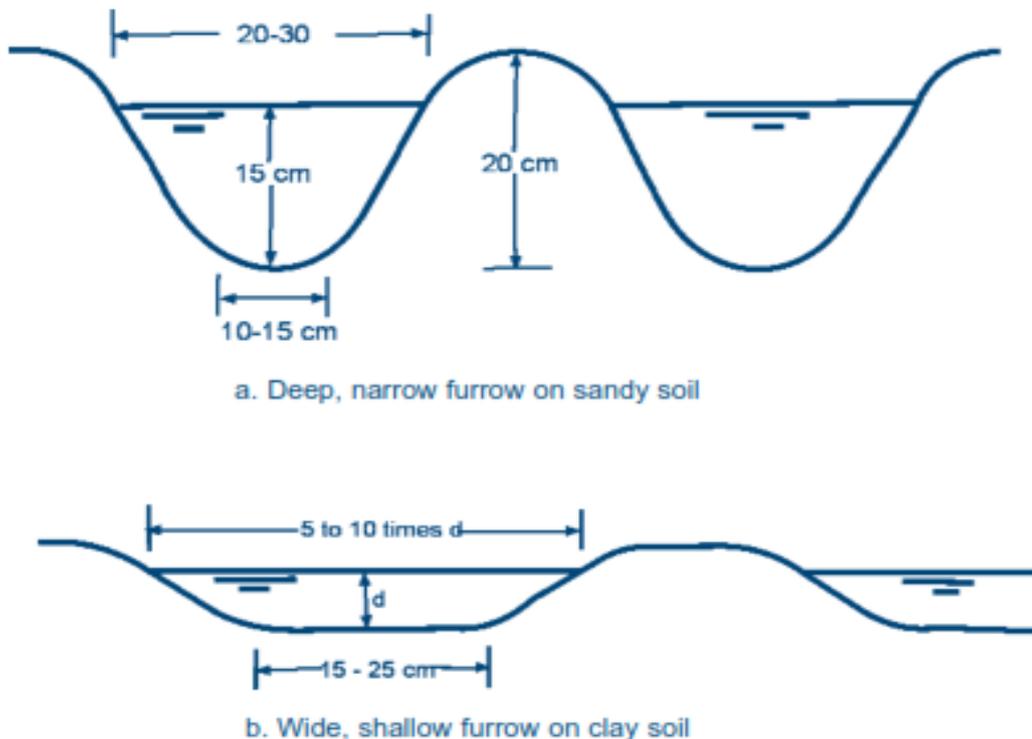


Figure 3-1 Furrow shape depending on soil type

#### (ii) Furrow Spacing

The spacing between furrows depends on the water movement in the soil, which is texture related, on the crop agronomic requirements as well as on the type of equipment used in the construction of furrows. In practice a compromise often has to be reached between these factors.

When water is applied to a furrow, it moves vertically under the influence of gravity and laterally by capillarity. Clay soils have more lateral movement of water than sandy soils because of their small pores, which favor capillary action. In this regard, larger spacing can be used in heavier soils than in light soils. In general, a spacing of 0.3m and 0.6m has been proposed, for coarse soils and fine soils respectively. For heavy clay soils up to 1.2m has been recommended. It should also be realized that each crop has its own optimum spacing and the ridges should be spaced according to the agronomic recommendations. In addition, the equipment available on the farm determines the furrow spacing, as this is adjustable only within limits. However, in all instances the furrow spacing adopted should ensure a lateral spread of water between adjacent furrows that will adequately wet the entire root zone of the plants.

*(iii)* Furrow length

The optimum length of a furrow is usually the longest furrow that can be safely and efficiently irrigated. Proper furrow length depends largely on the hydraulic conductivity of the soil. Furrows shall be shorter on a porous sandy soil than on a tight clay soil. The length of furrow which can be efficiently irrigated may be as short as 45m on sandy soils which take up water rapidly, or as much as 100m or longer on clay soils with low infiltration rates. The length of furrow may often be limited by the size and shape of the field. Since the proposed command area is owned by local farmers the maximum furrow length shall be up to 100m.

*(iv)* Furrow slope:

The slope or grade of the furrow is important because it controls the speed at which water flows down the furrow. A minimum furrow grade of 0.06 per cent is needed to ensure surface drainage. When the slope of the land is too steep, the furrows should be round the hill rather than straight down the slope; thus, the contour furrow method permits the use of furrows even on fairly steep land. For the project, all furrows aligned along contour and hence the minimum slope is 0.06%.

*(v)* Furrow stream:

The size of the furrow stream can be varied even after the furrow has been installed. The maximum size of the irrigation stream that can be used at the start of the irrigation is limited by considerations of erosion in furrows, overtopping of furrows and prevention of runoff at the downstream end. The maximum non-erosive flow rate in furrows is estimated by the following empirical equation:

$$q_m = \frac{0.60}{s}$$

Where;

$q_m$  = maximum non-erosive stream, l/s

$s$  = slope of furrow expressed as a percent

The average depth of water applied during irrigation can be calculated from the following relationship:

$$d = \frac{q * 360 * t}{w * L}$$

Where;

$d$  = average depth of water applied, cm

$q$  = stream size, l/s

$t$  = duration of irrigation (elapsed time), hours

$w$  = furrow spacing, m

$L$  = furrow length, m

The size of the furrow stream varies from 0.5 to 2.5 liters per second. To obtain the most uniform irrigation, the largest stream of water that will not cause erosion is used in each furrow at the beginning of irrigation. Its purpose is to wet the entire length of each furrow as quickly as possible, thus enabling the soil to absorb water evenly through the entire furrow length. After the water reaches the lower end of a furrow, the stream is reduced or cut back so that it will just keep the furrow wet throughout its length with a minimum waste at the end. This cut back stream flows until the required amount of water has been applied. With level furrows, however, the initial stream is continued from the beginning to the end of irrigation. The water is ponded in the furrow until it is absorbed by the soil.

Flow into furrows can be carefully regulated for uniform water distribution and efficient irrigation through difference method of regulators outlets. Furrow sizes and stream sizes can be easily

selected in the field for different soil and crop conditions, as the stream size can be easily manipulated by farmer.

### 3.3.3 Naming of canal units

In naming of the canals, Ethiopian humans naming system is adopted i.e. from child name to parental name. The naming of secondary units reflects the name of the canal that supplies it accept from the main canal.

#### Main Canal

The main canals directly off taking from river, have been named with out suffixes.

- MC = Main Canal.

#### Secondary Canals

The canals directly off taking from main canal have been named with One suffix. Which describes below.

SC1 = Secondary canal one off taking from Main Canal.

#### Tertiary Canals

The tertiary canals off- taking from main canal and secondary canal are named with one suffixes according to their order of appearance . Which describes below.

- TC1 = Tertiary canal one the first tertiary canal off taking from MC or SC considering starting point from the head work
- TC2 = Tertiary canal two the second tertiary canal off taking from MC or SC considering starting point from the head work.

#### Field Canals

The Field canals off- taking from each Canals are named with different suffixes. Which describes below.

- FC1 = Field canal one off taking from Directly from Main Canal.
- FC1-2 = Field canal one off taking from Directly from Secondary Canal Two.

- FC1-3-2 = Field canal one off taking from Tertiary Canal three in Secondary Canal Two.

### 3.3.4 Crop Water Requirement

The crop water requirement/*design supply for the project is 1.90 lit/sec/ha*, in the month of January is required with the assumption of a daily irrigation cycle of 12 hours and rotational flow in the main canals. Therefore, the total irrigation water required to satisfy net irrigation command area 63 ha of land will be calculated from the formula,

$$Q = \text{Duty} * \text{area}$$

Where Q is discharge in lit/sec

$$\text{Duty} = \text{flow in lit.sec/ha} = 1.90 \text{ lit/sec/ha} * 63 \text{ ha}$$

$$A = \text{area in ha} = 63 \text{ ha}$$

$$Q = 1.90 \text{ lit/sec/ha} * 63 \text{ ha} = \mathbf{119.7 \text{ lit/sec.}}$$

### 3.3.5 Topographic and soil survey

Prior to the preparation of the layout of the irrigation system, topographic survey has been carried out for the entire and potential command area of the project. These maps are prepared using software and CAD system. Based on the field survey data, major and minor contours are constructed in 1 m and 0.5 m vertical intervals respectively for detailed planning of irrigation system. On this topographic map the layout of the irrigation system has been designed. The topographic maps also show physical features, spot levels, bench marks and natural drainage, traditional irrigation canals, water logged area etc. on the command area.

The total boundary of the project area covers an area more than 94 ha gross command area and has net area of 63ha. The command area has gentle slope. The command area covers minimum and maximum elevation of 1656 and 1713 masl.

## 3.4 DESIGN OF IRRIGATION CANALS

### 3.4.1 General

Open canals are typically open geometric cross sections used to carry irrigation water to its point of use. These canals should be of adequate size and installed on non-erosive grades. Small, inadequate canals that do not have proper water control structures and maintenance probably are the source of more trouble and consume more time in operating a surface irrigation system than any other cause.

Open channels that carry irrigation water from a source to one or more farms are typically referred to as Main canals and Secondary Canals; and are generally permanent installations. Field or farm ditches convey and distribute water from the source of supply to a fields within a farm. Most are permanent installations except where they are used within a long field to shorten length of runs, where excessive sediment is in irrigation water, or where crop rotations require differing field layouts. In these cases they are installed at planting time and removed before or following harvest.

A canal cross-section can be any shape. But it is sensible to choose a profile that is easy to construct and does the job of carrying water for the least cost and with the best practical hydraulic efficiency. Unlined trapezoidal shaped canals are the most common and economic solution in most irrigation schemes in all situation of terrain. The flow of water in irrigation canal is classified as steady uniform flow. In case of canals running on cliffs/hills, rectangular section will be used so as to avoid extended embankment width and reduce land slide. The canal sections should be chosen ideal for construction and maintenance enabling cost effective & economical.

All the canals have been designed using Manning's formula:

$$Q = \frac{A * R^{2/3} * S^{1/2}}{n}$$

Where: Q = discharge (m<sup>3</sup>/s)

R = mean hydraulic radius (flow area / wetted perimeter)

S = hydraulic gradient

n = Manning's roughness coefficient

A velocity of 0.45 m/s is often quoted as a minimum velocity that will not induce siltation, reduce weed growth, and prevent schistosomiasis (bilharzia). This velocity, however, requires a steep longitudinal bottom slope which is hardly desirable in irrigation canals where loss of elevation usually has to be kept minimal. Some 0.30 m/s is, however, considered to be a minimum velocity in large earth canals, and a velocity of 0.10 to 0.15 m/s in small canals. Velocities below these limits result in uneconomically wide sections.

### 3.4.2 Main Canal

The Main Canal (MC) is the largest size of the canal network, capable of conveying the flow of the system under favorable hydraulic conditions of flow velocity with minimum losses. The main

canal is aligned along contour with different slope. It takes off from the head regulator located at the head work to the tail end where the last secondary canal (SC5) off takes. The design discharge of the main canals at 12 hrs  $0.12\text{m}^3/\text{s}$  and has a total length of 3.59km. The longitudinal slopes of the main canal adopted are 0.5m/km and in some reach 1 in 100m.

The main canal is aligned in Gentle slope. Linings have been suggested throughout its length in order to reduce canal failure and water loss. The capacity of the canal is determine as follows.

#### Design parameters

Design Discharge,	$Q = 119.7\text{l/s}$
Longitudinal Slope,	$S = 1\text{m/km}$
Manning Roughness,	$n = 0.018$ (for Masonry Lined Canal)
Section Type	Rectangular section is chosen

The best hydraulic section of a rectangular lined canal is when the bed width is equal with the flow depth (i.e.  $b = d$ ). But the recommended width and depth should be workable and the width and depth shall not less than 0.3m and 0.35m respectively.

$$\text{Discharge, } Q = \text{Duty} * \text{Command Area}$$

$$\text{Flow area, } A = b * d$$

$$\text{Wetted perimeter, } P = b + 2d$$

$$\text{Hydraulic radius, } R = A/p$$

$$n = 0.018$$

$$S = 0.0005$$

Assuming Bed width is 0.6m Using manning's formula, D is calculated by trial and error until the provided canal section can convey the required discharge.

In lined canals the normal free board for the lining is  $f_b = 0.15 * Q^{0.35}$  (f in m, Q in  $\text{m}^3/\text{s}$ ) with a minimum of 0.2 m. The height of the canal bank above the water level should be  $fb = 0.4 * Q^{0.25}$  with a minimum of 0.2 m. These values should be increased when siltation, risk of strong winds and wave action, and possible large inflows of water resulting from cross-drainage flows or

deficient operation are expected. For this particular project for design discharge of  $0.099\text{m}^3/\text{s}$  the height of the canal bank above the water level is  $fb = 0.4 \times 0.059 = 0.024\text{m}$ . Therefore, adopted free board is  $0.25\text{m}$ .

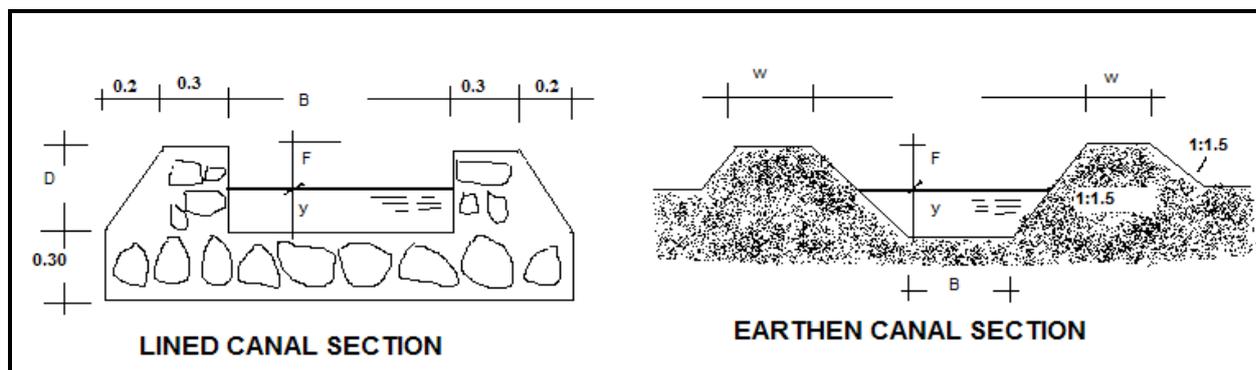


Figure 3-2: Lined and Earthen canal section

Table 3-1: Pertinent Canal Features and Hydraulics Parameters along Main Canal.

Chainage (m)	Structure	Irrigable Area (ha)	Q ( $\text{m}^3/\text{s}$ )	Slope	B (m)	FSD (m)	FB (m)	Total Depth (m)	V (m/s)
108.58	Foot Path	63.0	0.12	0.00100	0.60	0.38	0.3	0.68	0.53
403.54	FC1	63.0	0.12	0.00100	0.60	0.38	0.3	0.68	0.53
459.38	Foot Path	62.7	0.12	0.00067	0.60	0.44	0.3	0.74	0.45
582.78	FC2	62.7	0.12	0.00067	0.60	0.44	0.3	0.74	0.45
699.75	Flume 1	61.3	0.12	0.00067	0.60	0.43	0.3	0.73	0.45
845.52	FC3	61.3	0.12	0.00067	0.60	0.43	0.3	0.73	0.45
931.99	TC1	60.3	0.11	0.00067	0.60	0.42	0.3	0.72	0.45
1341.83	FC4	58.5	0.11	0.00067	0.60	0.41	0.3	0.71	0.45
1351.78	Pipe Culvert 1	57.6	0.11	0.00067	0.60	0.41	0.3	0.71	0.45
1495.07	TC2	57.6	0.11	0.00067	0.60	0.41	0.3	0.71	0.45
1519.03	Pipe Culvert 2	55.6	0.11	0.00067	0.60	0.40	0.3	0.70	0.44
1773.24	SC1	55.6	0.11	0.00067	0.60	0.40	0.3	0.70	0.44
1836.52	Foot Path	40.8	0.08	0.00067	0.50	0.38	0.3	0.68	0.41
2000.47	Super Passage 1	40.8	0.08	0.00067	0.50	0.38	0.3	0.68	0.41
2186.43	Flume 2	40.8	0.08	0.00067	0.50	0.38	0.3	0.68	0.41
2216.09	Drop	40.8	0.08	0.00667	0.30	0.27	0.3	0.57	0.95
2279.29	FC5	40.8	0.08	0.00667	0.30	0.27	0.3	0.57	0.95
2331.33	SC2	39.0	0.07	0.00667	0.30	0.26	0.3	0.56	0.95
2383.71	FC6	39.0	0.07	0.00667	0.30	0.26	0.3	0.56	0.95
2442.34	Flume 3	38.7	0.07	0.00667	0.30	0.26	0.3	0.56	0.94

Chainage (m)	Structure	Irrigable Area (ha)	Q (m <sup>3</sup> /s)	Slope	B (m)	FSD (m)	FB (m)	Total Depth (m)	V (m/s)
2638.94	SC3	25.7	0.05	0.02000	0.30	0.12	0.3	0.42	1.31
2671.29	Drop	25.7	0.05	0.02000	0.30	0.12	0.3	0.42	1.31
2751.46	Foot Path	25.7	0.05	0.02000	0.30	0.12	0.3	0.42	1.31
2984.40	Flume 4	25.7	0.05	0.00200	0.30	0.30	0.3	0.60	0.54
3026.27	Foot Path	25.7	0.05	0.00200	0.30	0.30	0.3	0.60	0.54
3061.46	FC7	25.7	0.05	0.00200	0.30	0.30	0.3	0.60	0.54
3116.45	SC4	27.9	0.05	0.00200	0.30	0.32	0.3	0.62	0.54
3177.94	FC8	24.4	0.05	0.00200	0.30	0.29	0.3	0.59	0.53
3205.49	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3210.42	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3217.01	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3226.17	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3234.47	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3240.83	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3247.20	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3253.73	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3262.17	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3269.57	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3274.88	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3279.76	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3284.64	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3289.52	Drop	23.9	0.05	0.05000	0.30	0.09	0.3	0.39	1.78
3294.40	Drop	23.9	0.05	0.01429	0.30	0.13	0.3	0.43	1.78
3302.09	Super Passage 2	23.9	0.05	0.01429	0.30	0.13	0.3	0.43	1.13
3315.91	Wash basin	23.9	0.05	0.00500	0.30	0.20	0.3	0.50	0.76
3329.09	Foot Path	23.9	0.05	0.00500	0.30	0.20	0.3	0.50	0.76
3590.20	SC5	23.9	0.05	0.00125	0.40	0.25	0.3	0.55	0.45

Note: HW- Head Work ,SC- secondary canal off taking point, FC – Field Canal, FP - Foot path, Road:- Road Crossing at Robe Gore Main Road.

There are 5 division boxes (DB) and 10 Turnouts on the main canal diverting the irrigation water to the respective off taking canals. Main canal is designed reach by reach. Secondary canals are designed to supply water to Tertiary canals. Tertiary canal are designed to supply water to all field plots in rotation during peak demand and hence have uniform cross-section.

### 3.4.3 Secondary Canals

Depending on the natural drainage within the project command area the entire areas is divided in different blocks and each block is served by one secondary canal. Totally Five secondary canals 0.81km are proposed for this scheme which is aligned across the contour. Each secondary canal has different canal capacity, length and area coverage which depends on the topographic nature of the command area. The hydraulic parameters for Secondary canals are shown Table below.

Table 3-2: Hydraulics Parameters of Secondary Canal

Chainage (m)	Canal Name	Structure	Q (m <sup>3</sup> /s)	Slope	B (m)	FSD (m)	FB (m)	Total Depth (m)	V (m/s)
3.94	SC1	TC3&TC4	0.03	0.100	0.30	0.10	0.25	0.35	1.84
92.45	SC1	TC5&TC6	0.03	0.100	0.30	0.10	0.25	0.35	1.84
27.18	SC2	TC7&TC8	0.003	0.200	0.30	0.10	0.25	0.35	1.10
58.28	SC2	TC9	0.003	0.200	0.30	0.10	0.25	0.35	1.10
4.55	SC3	TC10&TC11	0.026	0.100	0.30	0.10	0.25	0.35	1.90
93.35	SC3	TC12&TC13	0.026	0.100	0.30	0.10	0.25	0.35	1.90
72.71	SC4	TC14&TC15	0.009	0.143	0.30	0.10	0.25	0.35	1.41
154.31	SC4	TC16&TC17	0.009	0.125	0.30	0.10	0.25	0.35	1.41
3.61	SC5	TC18	0.033	0.067	0.30	0.10	0.25	0.35	1.79
133.98	SC5	TC20&TC21	0.033	0.067	0.30	0.10	0.25	0.35	1.79
222.60	SC5	TC22	0.033	0.040	0.30	0.10	0.25	0.35	1.79

### 3.5 Tertiary Canals

The entire command area of the project is planned to be irrigated using the tertiary canals that off take directly from the main canal and secondary canal and supplies irrigation water to field canals and run nearly as a contour canal. In the system layout there are 22 tertiary canals about 5.6km.

The designed discharge is determined based on the duty of irrigation and rotation criteria. The sections of the canals are determined by using manning's formula, and all of them are trapezoidal section.

The details of the tertiary canals with length, command area and discharge capacities are shown in Table below.

Table 3-3: Hydraulics Parameters of Tertiary Canals

S.No.	SC Name	Chainage	TC Name	TC Length	TC Q (l/s)	B (m)	FSD (m)	FB (m)	Total Depth (m)
1	MC	931.99	TC1	297.11	18	0.3	0.1	0.25	0.35
2	MC	1495.07	TC2	100.65	81	0.3	0.1	0.25	0.35
3	SC1	3.94	TC3	182.91	8	0.3	0.1	0.25	0.35
4	SC1	3.94	TC4	360.56	8	0.3	0.1	0.25	0.35
5	SC1	92.45	TC5	165.75	9	0.3	0.1	0.25	0.35
6	SC1	92.45	TC6	249.69	8	0.3	0.1	0.25	0.35
7	SC2	27.18	TC7	81.69	8	0.3	0.1	0.25	0.35
8	SC2	27.18	TC8	64.85	8	0.3	0.1	0.25	0.35
9	SC2	58.28	TC9	88.81	8	0.3	0.1	0.25	0.35
10	SC3	4.55	TC10	146.64	11	0.3	0.1	0.25	0.35
11	SC3	4.55	TC11	293.22	11	0.3	0.1	0.25	0.35
12	SC3	93.35	TC12	260.19	11	0.3	0.1	0.25	0.35
13	SC3	93.35	TC13	316.10	10	0.3	0.1	0.25	0.35
14	SC4	72.71	TC14	90.73	8	0.3	0.1	0.25	0.35
15	SC4	72.71	TC15	135.39	8	0.3	0.1	0.25	0.35
16	SC4	154.31	TC16	164.14	8	0.3	0.1	0.25	0.35
17	SC4	154.31	TC17	118.12	8	0.3	0.1	0.25	0.35
18	SC5	3.61	TC18	243.75	8	0.3	0.1	0.25	0.35
19	SC5	133.98	TC20	224.72	8	0.3	0.1	0.25	0.35
20	SC5	133.98	TC21	641.26	18	0.3	0.1	0.25	0.35
21	SC5	222.60	TC22	663.94	14	0.3	0.13	0.25	0.38
22	MC	3590.20	TC19	707.69	14	0.3	0.1	0.25	0.35

### 3.6 Field Canals

The command area of each tertiary canal is further sub-divided into several segments by field canals, which supply water to the furrows. As shown in the layout, all field canals run across the contours. By considering the proposed crops, furrow method of irrigation has been adopted. Accordingly, irrigation water will be applied to the farm through furrows. The maximum length of furrows is considering 100 meters except some conditional canals. Irrigation water will be supplied to several furrows at a time, depending on the size of field canal that apply irrigation water. The total discharge of the tertiary canal is totally diverted to each filed canals and there will be a rotation among all field canals.

As can be seen from the layout, some of the filed canals can be used to irrigate both sides of the command area depending on the condition of the individual plots of land owned by individual farmers. All field canals are left for the beneficiaries to be arranged every irrigation season during

land preparation; meaning their bill of quantities and cost are not included. The typical off take location and size at the inlet of each of these field canals is designed.

### 3.7 Design Of Irrigation Structures

#### 3.7.1 General

In any irrigation scheme various type of structures are required for proper operation of the entire canal and drain system. Culverts are required on road crossings; division boxes are need for dividing the flow as per area coverage, drop structure in order to negotiate (balance) the canal slope with the ground slop, cross drainages are intended to provide on the canal to cross gullies/drains/rivers etc. The structures are made of concrete/masonry. Hence the analysis made for sizing of appropriate walls, are similar with that of the masonry walls of the retaining wall. A minimum 1000mm length of riprap and pitching is provided as a protection at the inlet and outlet of all structures. The type of structures proposed for the scheme is detailed below.

#### 3.7.2 Design of Drop Structures

Drop structures are flow control structures that are installed in canals when the natural land slope is too steep. The drops allow reducing the canal bed slopes to convey water without causing erosive velocities. For this, the canal is divided into different reaches over its length; each reach follows the design canal gradient. When the bottom level of the canal becomes too high compared to the natural ground level, drop structures are installed. Vertical drops are used for the dissipation of up to 1.0 m head for unlined canals and up to 1.5 m head for lined canals but exceptionally we used 1.4&1.6m drops for earths on TC 22.

An important aspect of a drop is the stilling basin, required to avoid downstream erosion. The floor of the stilling basin is set at such a level that the hydraulic jump occurs at the upstream end of the basin floor in order to avoid erosion at the unprotected downstream canal bed. A common straight drop structure is used for this scheme.

Canal Name	Chainage(m)	Qdes (m <sup>3</sup> /s)	Fb(m)	b1(m)	b2(m)	d1(m)	d2(m)	Lup (m)	Ldp (m)	H(m)
MC	2216	0.08	0.30	0.30	0.30	0.27	0.27	1.70	1.70	1.2
MC	2671	0.05	0.30	0.30	0.30	0.12	0.12	1.60	1.60	0.5
MC	3205	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.4
MC	3210	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3217	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.4

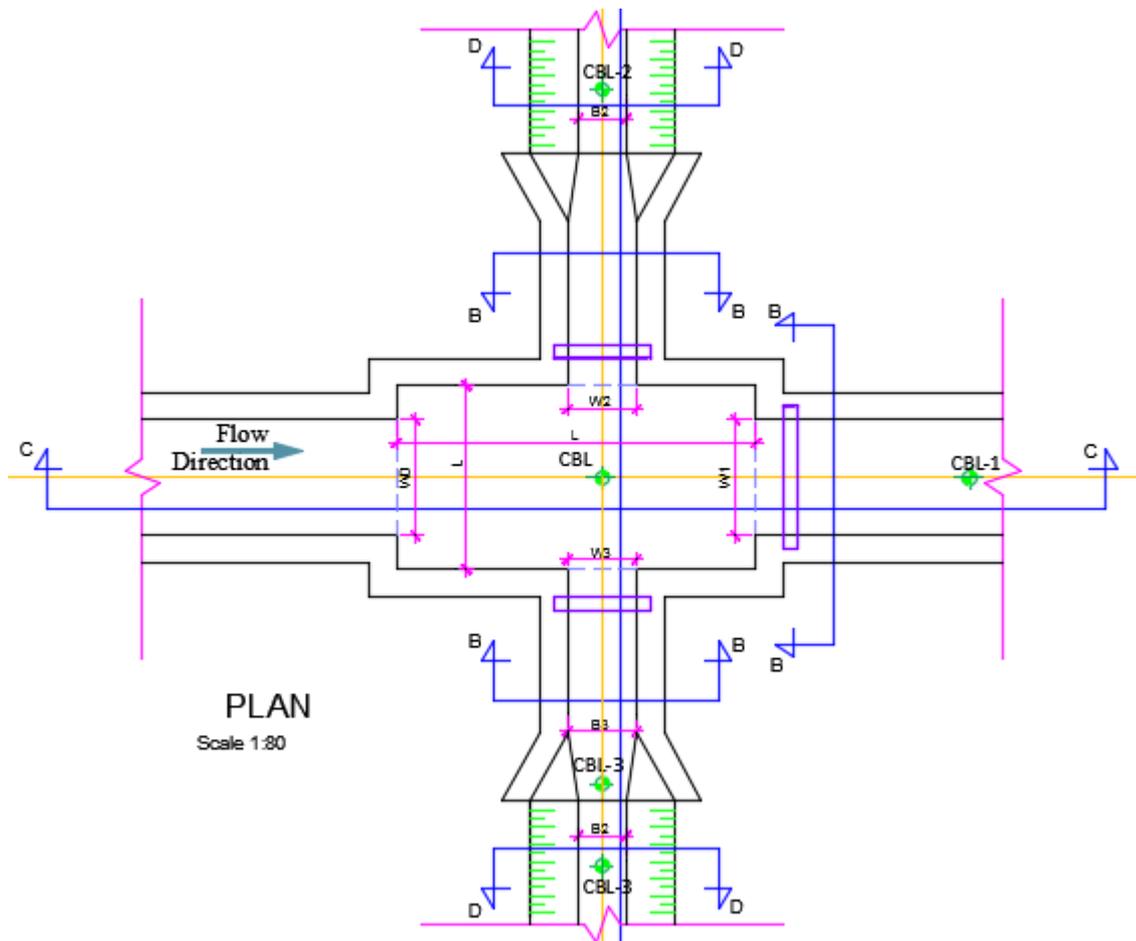
Canal Name	Chainage(m)	Qdes (m <sup>3</sup> /s)	Fb(m)	b1(m)	b2(m)	d1(m)	d2(m)	Lup (m)	Ldp (m)	H(m)
MC	3226	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3234	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.6
MC	3241	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3247	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.6
MC	3254	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.6
MC	3262	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.6
MC	3270	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.6
MC	3275	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3280	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3285	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3290	0.05	0.30	0.30	0.30	0.09	0.09	1.60	1.60	0.8
MC	3294	0.05	0.30	0.30	0.30	0.13	0.13	1.60	1.60	0.8
TC1	0	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.8
TC1	3	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.8
TC1	7	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.4
TC1	11	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.8
TC1	16	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.4
TC2	76	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	4	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.8
SC1	8	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	13	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	18	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	22	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	27	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	32	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	36	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	41	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	45	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	50	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	54	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	60	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
SC1	67	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.6
SC1	80	0.03	0.25	0.30	0.30	0.04	0.04	1.50	1.50	0.4
TC-3	135	0.00	0.25	0.30	0.30	0.07	0.07	1.30	1.30	0.4
TC-3	155	0.00	0.25	0.30	0.30	0.07	0.07	1.30	1.30	0.2
TC-3	165	0.00	0.25	0.30	0.30	0.10	0.10	1.30	1.30	0.4
SC-2	0	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.4
SC-2	7	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.4
SC-2	15	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.6
SC-2	31	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.6

Canal Name	Chainage(m)	Qdes (m <sup>3</sup> /s)	Fb(m)	b1(m)	b2(m)	d1(m)	d2(m)	Lup (m)	Ldp (m)	H(m)
SC-2	39	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.6
SC-2	46	0.00	0.25	0.30	0.30	0.01	0.01	1.30	1.30	0.4
SC-3	23	0.03	0.25	0.30	0.30	0.05	0.05	1.50	1.50	0.4
TC-11	0	0.01	0.25	0.30	0.30	0.05	0.05	1.40	1.40	0.4
TC-11	15	0.01	0.25	0.30	0.30	0.05	0.05	1.40	1.40	0.4
TC-11	45	0.01	0.25	0.30	0.30	0.05	0.05	1.40	1.40	0.6
TC-11	150	0.01	0.25	0.30	0.30	0.05	0.05	1.40	1.40	0.8
TC-11	165	0.01	0.25	0.30	0.30	0.05	0.05	1.40	1.40	0.8
SC-4	41	0.01	0.25	0.30	0.30	0.02	0.02	1.40	1.40	0.3
SC-5	4	0.03	0.25	0.30	0.30	0.06	0.06	1.50	1.50	0.6
SC-5	10	0.03	0.25	0.30	0.30	0.06	0.06	1.50	1.50	0.6
SC-5	31	0.03	0.25	0.30	0.30	0.06	0.06	1.50	1.50	0.4
SC-5	134	0.03	0.25	0.30	0.30	0.06	0.06	1.50	1.50	0.4
TC-22	15	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	0.4
TC-22	285	0.01	0.25	0.30	0.30	0.13	0.13	1.40	1.40	0.5
TC-22	318	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	1
TC-22	337	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	0.8
TC-22	356	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	0.8
TC-22	420	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	0.8
TC-22	435	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	1.4
TC-22	450	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	0.4
TC-22	465	0.01	0.25	0.30	0.30	0.09	0.09	1.40	1.40	1.6

### 3.7.3 Design of Division Boxes

Division box is provided in the system to control and quantify the volume of water supplied to the various canal networks in accordance with their respective discharge required as per the schedule. This is achieved by properly designed division box so that the width of opening provided to the off – taking canal and parent canal should be proportional to the discharge required. At different points of the main, secondary and tertiary canals division boxes are provided. Gate should be provided at the outlet of the boxes.

Since the flow in all canals are open channel, the division boxes are designed using broad crest flow formula by assuming the same equal discharge coefficient & sill height for all direction.



*Figure 3-3: Typical Division Box*

$Q_0$  = Discharge entering in to the division box from u/s canal

$Q_1$  = Discharge of the parent canal that flow to d/s side ( $Q_0 - Q_2 - Q_3$ )

$Q_2$  = Discharge of the off – taking canal to the right side

$Q_3$  = Discharge of the off – taking canal to the left side

$B_0$  = Opening width of the parent canal that flow from u/s side

$B_1$  = Opening width of the parent canal that flow to d/s side

$B_2$  = Opening width of the off- taking canal to right side

$B_3$  = Opening width of the off- taking canal to left side

Assuming the discharge passing through the opening of division box as a flow over broad crested weir ( $Q = cbH^{3/2}$ ) and coefficient of discharge,  $c$  and head over the crest,  $H$  is obtained from canal design. The design of all division boxes is carried out and the final result is tabulated and presented in the tables below.

### 3.7.4 Off-takes

Off-takes are other on-farm structures to be built on tertiary canals to divert water to field canals. Thus, they are opening to field canals but all are designed to supply one way.

There are 64 of such structures arranged on tertiary canals i.e. at head of each field canal. Each of them is to be controlled with simple shutters on which chain is to be attached to lift to the required level.

#### 3.7.4.1 Hydraulic Design Parameters of Off-takes

Flow in off-takes is governed by the orifice formula like that of turnouts. Since flow in each field canal is expected to be same as that of corresponding tertiary canal (i.e. rotation will be within tertiary units), size of turnout designed for head regulators of tertiary canal is taken same size as that of corresponding field canals. Thus same pipe diameter as designed for corresponding turnout can be used here.

Table 3-4: Design output of Division boxes and Turnouts on Main and Secondary Canals

Wataba Baddesa SSIP Hydraulic and Related Parameters for Division box																					
Parent Canal name	Branch Canal name	Chainage	B, m	d(m)	D (d+fb)	L, m	W0, m	W1, m	W2, m	W3, m	H0	H1	H2	H3	Hs1	Hs2	Hs3	L0, m	L1, m	L2, m	L3, m
SC1	TC3&TC4	2	0.3	0.1	0.35	1	0.6	0.4	0.3	0.3	0.10	0.09	0.04	0.05	0.01	0.06	0.05	1.40	1.20	1.10	1.10
SC1	TC5&TC6	233	0.3	0.1	0.35	1	0.4	0.3	0.3	0.3	0.10	0.08	0.04	0.04	0.02	0.06	0.06	1.20	1.10	1.10	1.10
SC2	TC7&TC8	27	0.3	0.1	0.35	1	0.3	0.3	0.3	0.3	0.10	0.02	0.02	0.01	0.08	0.08	0.09	1.10	1.10	1.10	1.10
SC2	TC9	58	0.3	0.1	0.35	1	0.3	0	0.3	0	0.10	0.00	0.02	0.00	0.00	0.08	0.00	1.10	0.00	1.10	0.00
SC3	TC10&TC11	5	0.3	0.1	0.35	1	0.5	0.3	0.3	0.3	0.10	0.10	0.04	0.05	0.00	0.06	0.05	1.30	1.10	1.10	1.10
SC3	TC12&TC13	93	0.3	0.1	0.35	1	0.3	0	0.3	0.3	0.10	0.00	0.06	0.07	0.00	0.04	0.03	1.10	0.00	1.10	1.10
SC4	TC14&TC15	73	0.3	0.1	0.35	1	0.3	0.3	0.3	0.3	0.10	0.05	0.02	0.02	0.05	0.08	0.08	1.10	1.10	1.10	1.10
SC4	TC16&TC17	154	0.3	0.1	0.35	1	0.3	0	0.3	0.3	0.10	0.00	0.04	0.02	0.00	0.06	0.08	1.10	0.00	1.10	1.10
SC5	TC18	4	0.3	0.1	0.35	1	0.7	0.6	0.3	0	0.10	0.09	0.05	0.00	0.01	0.05	0.00	1.50	1.40	1.10	0.00
SC5	TC20&TC21	130	0.3	0.1	0.35	1	0.6	0.3	0.3	0.3	0.10	0.09	0.06	0.06	0.01	0.04	0.04	1.40	1.10	1.10	1.10
SC5	TC22	223	0.3	0.1	0.35	1	0.3	0	0.3	0	0.10	0.00	0.09	0.00	0.00	0.01	0.00	1.10	0.00	1.10	0.00

### 3.7.5 Gates

These are structures used to control flow coming in to and going out of canals. Major considerations are:

Vertical lift gates will be incorporated into many of the hydraulic control structures, including:

- The Main Canal head-regulators;
- All secondary canal division boxes;
- All tertiary canal turnouts;
- All field canal off-takes

Depending on the downstream water levels, the gates will either be under free flow conditions or submerged flow conditions. Under free flow conditions the jet under gate is not submerged by the downstream water level and a hydraulic jump is formed in the stilling basin for the structure. Under submerged flow conditions the downstream water level is sufficiently high to draw out the jet. The head/discharge relationship under these two conditions is given by:

$$Q = C_d \times C_v \times a \times w \times \sqrt{2 \times g \times (h_1 - \delta \times a)} \quad \text{- free flow}$$

$$Q = C_d \times C_v \times a \times w \times \sqrt{2 \times g \times (h_1 - h_2)} \quad \text{- submerged flow}$$

Where:

$Q$	=	Discharge (m <sup>3</sup> /s).
$C_d$	=	discharge coefficient, taken as 0.6
$C_v$	=	velocity coefficient, taken as 1.0
$a$	=	gate opening
$w$	=	gate width
$g$	=	acceleration due to gravity
$h_1$	=	upstream head over the gate opening
$h_2$	=	downstream head (to the same datum as $h_1$ )
$\delta$	=	contraction coefficient, taken as 0.63

These gates are to be fabricated from mild steel. Detailed design and fabrication details for the gates are given in the drawing album. Sizes of each gate is dependent on the size of corresponding outlet which are given under head-regulators, division boxes, turnouts and off-takes

All gates will be manually operated. Gates which are smaller than 500 mm<sup>2</sup> will have no lifting spindle but chain to prevent from robbery.

### 3.7.6 Crossing Structures

In addition to the canal network, it is usually necessary to use canal structures to convey water along the canal route. Some of these structures include:-

- ❖ Drainage crossing structures like Inverted canal siphons to convey canal water under natural channels, Drainage Pipe culvert to convey drainage water under canal and Flumes to conduct canal water across deep rivers/gullies.
- ❖ Road crossings to carry canal water under roadways,

#### I. Drainage Crossing Structures

Drainage crossing structures are required wherever the canal line crosses natural drainage channels. As far as possible, the canal should be carried above or below the channel, and level crossing should be avoided since they cause silt to enter the canals and, in floods, debris and excess water may damage the canal.

To select the most appropriate structure, the factors to be considered are:

- Type and size of drainage channel in relation to canal size:
  - ✓ Small local drainage way
  - ✓ Seasonal stream
  - ✓ Perennial stream
- Usefulness as a supplementary sources
- Sediment and/or debris loads during flood
- Relative levels of canal water level and bed and stream bed

- Foundation conditions in and adjacent to the channel
- The strategic importance of the structure in terms of the scheme performance

As canal banks rapidly become access ways, some form of crossing should be provided either on top or parallel the cross-drainage works. In the hills, only foot traffic should be provided, but in the Flat land, light vehicular traffic (car, carts, etc.) should be allowed.

The channel should be inspected upstream and downstream of the crossing to check if erosion control structures are required and/or whether interceptor drains could be used to improve drainage of the catchment above the canal line.

Gabion checks may be used for erosion control. These structures have the advantage that they are relatively easy and cheap to construct and are structurally flexible also. The check should be adequately built into the banks to prevent any tendency for the stream to outflank the structure. The following Table shows that the return period varies depending on the project type, scale and type of structure provide. For the proposed Project, the project is small Scale and different return periods are provided in each type of structure.

*Table 3-5 Flood Return Periods for Cress-drainage Structure Design*

<b>Scheme Type</b>	<b>Structure Type</b>	<b>Location</b>	<b>Return Period</b>
Small/Medium Hills	Level Crossing, Drain Culverts, Drop and Pick up, Super Passage	Primary Canal	10
		Minor Canal	5
	Canal Siphon, Aqueduct	Primary Canal	25
		Minor Canal	10
Medium/Large Hills	Level Crossing, Drain Culverts, Drop and Pick up, Super Passage	Primary Canal	20
		Minor Canal	10
	Canal Siphon, Aqueduct	Primary Canal	50
		Minor Canal	25
Small/Medium Flat Land	Super Passage, Drain underpass	Primary Canal	10
		Minor Canal	5
	Canal Siphon, Aqueduct	Primary Canal	25
		Minor Canal	10
Medium/Large Flat Land	Super Passage, Drain underpass	Primary Canal	25
		Minor Canal	10
	Canal Siphon, Aqueduct	Primary Canal	50
		Minor Canal	25

### Design Procedure for cross drainage structures

- Establish levels and dimension of canal
- Establish levels and sections of drainage ways;
- Estimate the drain flow for the appropriate return period and estimate the corresponding flow depth at the crossing site;
- Compare levels and sizes of canal and drain. In hill areas, it is generally possible to route the canal to achieve level conditions appropriate to almost any type of crossing (by moving the alignment into or out of the slope);
- Select a structure, which is suited to the levels and dimensions of the two channels.

In the proposed project the proposed Crossing Structures are; Three Super passages are recommended that passes the catch drain

#### a. Super passages

The Super passages are provided when the drain level is above canal water level. The drain discharge is normally carried through the RCC concrete over top canal level. The canal section will have similar section with full supply condition i.e. no transition is required.

Super passages are located in Three different locations in the main canal in different sizes. The Location Size is summarized in the following table.

*Table 3-6 Summary of cross drainage structures by Supper Passage*

Name	Location	OGL	DBL	Drainage Slope	V	W	H	CBL	CTL	B	D
Supper Passage 1	2+000	1712.61	1712.61	0.01	1.01	3	0.4	1711.93	1712.607	0.5	0.38
Supper Passage 2	3+302	1685	1685.24	0.015	1.2	3	0.4	1684.81	1685.24	0.3	0.13

#### 3.7.7 Road crossings Culverts

Culverts are recommended at existing roads pathway crossing to maintain the communication. In addition, such crossings are also generally provided as required at existing cattle tracks and facilitate access into and out of the farm. They are also recommended in crossing of irrigation and drainage canals. Concrete

pipes are commonly used in the construction of culverts. In addition, footbridges will be required at intermediate locations, maximum walk way distance of 0.5km in local community living areas. At the location of each crossing the canal is converted from trapezoidal section to rectangular section to minimize span length.

Culverts are road crossing canal structures used to facilitate easy entrance to the scheme from access road and within the scheme itself. They are to be arranged along with other on-farm structures especially with drops/division boxes on main canals to secondary canals so as to minimize protection works.

For the crossings within the farm, since all canals are of small sizes, traditional crossings can be provided by beneficiaries as need be. Culverts are recommended on MC sand SCs and are considered on main road to existing villages road crossing. These selected culverts are of box type as they will be used for bridging the command to the main access road.

It will have similar slope & total depth equal to the parent canal (except that some free board is allowed). Thus, the canal should converge on arriving such site and diverge while crossing it if trapezoidal otherwise crosses with same dimension in case of rectangular canal. In this project area, these culverts will serve as a bridge expected for providing bearing capacity to medium trucks that will freight products from the corresponding farm plots. Trucks shall stand aside main canals and beneficiaries shall carry and load/unload there. There will be six of such road-crossing culverts on main and secondary canals to allow transportation for human and cattle as well as bicycles in addition to Main road crossing at 0+900 in the main canal.

The pedestrian crossing shall be precast concrete with 200mm thick over the masonry wall is proposed and a sample design is shown below. the provided crossing shall be box culvert and the detail is presented in drawing.

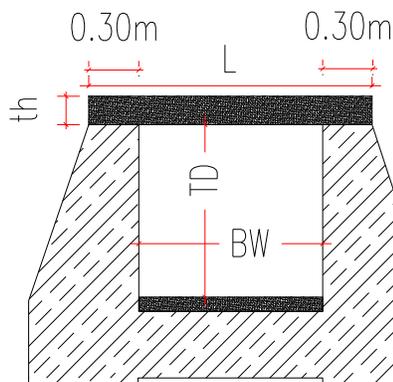


Figure 3-4: Typical Cross Section of the main canal at Foot Path Crossing

### 3.8 DESIGN OF DRAINAGE SYSTEM

#### 3.8.1 General

The preliminary aim of a drainage system in an irrigation command area is to remove excess water from the ground surface, as well as from the root zone in the sub-soil. The main source of excess water on the land surface is the rain falling over the command or catchments area and over irrigation.

#### 3.8.2 The objectives of surface drainage measures are to:

- Empty the submerged agricultural lands from surface water in certain periods so that standing crops are not damaged.
- Sufficiently lower the groundwater table to prevent water logging.
- Drain the irrigation surplus water during the dry season.

A surface drainage system serves a useful purpose at the time of heavy rainfall during storms by preventing prolonged submergence of agricultural fields. It quickly removes rain water collected on the ground. It would act the same way during the period of normal and low-intensity rainfalls whose occurrence is far more frequent than that of heavy rainfall. An efficient surface drainage system would significantly reduce the infiltration of water into the ground and increase the volume of runoff. This would be so during each and every event of rainfall, mild or heavy.

It is not practically feasible to altogether prevent temporary submergence of all lands at all times, but drainage systems can be improved to minimize the damage due to water logging at affordable costs. It is

not necessary that all submerged lands be emptied through drains. Some of them should be left as wetlands and water bodies to promote environmental protection

### **3.8.3 Existing Natural Drainage System**

There are natural gullies and streams identified that the main canal crosses in the project area. To remove the external and internal drainage.

### **3.8.4 Description of the Drainage Network Layout of the project**

Observing the terrain of the project area only catch drain is provided and the catch drain passes to the natural drainage in which the main canal crosses.

### **3.8.5 Drain Design Discharges**

In planning a surface drainage system, the prime objective is to remove the water standing on the ground surface within a period that the crops can tolerate. The volume of water to be drained depends on the intensity and duration of rainfall. The system can be designed using average frequent runoff or peak storm runoff, based on the urgency of removal of water and the soil types. The internal drainage system will be designed for the maximum quantity of water from two sources, i.e. rainfall runoff and irrigation surplus.

The design discharge is the product of the drainage modulus and the drainage area. The drainage area is the gross or total land area upstream of the point considered. For the field drains a constant section/design discharge is usually adopted. However, for tertiary, collector and main drains the design discharge increases along the drain.

Peak discharges are usually more pronounced and cause more damage on sloping land than on flat land. Therefore adopted approaches for surface drainage depend on the level of protection desirable.

Therefore, for the project area, it is preferred to design based on peak discharge. Hence peak (design) drainage modulus may be calculated using the approach detailed in the Ethiopia Road Authority Drainage Manual (ERA, 2002). The rational method, corrected for soil type and slope, uses the following equation from the ERA (2002):

$$Q = CIA$$

Where  $Q$  is the maximum rate of runoff ( $m^3/s$ ),  $C$  is the runoff coefficient representing a ratio of runoff to rainfall.

### 3.8.6 Hydraulic Design of Drainage Channels

In the case of Wataba Baddesa there are two types of drainage canals catch drains (CD) this are drainage canals which drains excess water from the surrounding watershed especially this drainage canals protect the main canal. The other type is field drains (FD) this types of drainage canals drains excess water from the commands or the fields and joins to the natural drains. The total hydraulic design data tabulated as follow: -

*Table 3-7 drainage canals hydraulic design out put*

Drainage Name	Reach (m)		Flow ( $m^3/s$ )	Bed slope, $S$ (m/m)	Side slope, m	Bed width, $B$ (m)	FSD, $D$ (m)	FB(m)
	From	To						
CD-1	0.00	392.17	0.335	0.0125	1.5	0.40	0.32	0.25
CD-2	0.00	193.37	0.381	0.0100	1.5	0.40	0.36	0.25
	193.37	235.96	0.381	0.0067	1.5	0.40	0.39	0.25
CD-3	0.00	291.66	0.481	0.0020	1.5	0.50	0.55	0.25
CD-4	0.00	121.56	0.817	0.0040	1.5	0.60	0.57	0.25
CD-5	0.00	794.67	1.650	0.0040	1.5	1.00	0.70	0.25
CD-6	0.00	665.62	1.152	0.0040	1.5	0.90	0.60	0.25
CD-7	0.00	68.42	1.933	0.0040	1.5	1.10	0.73	0.25
	68.42	360.01	1.933	0.0029	1.5	1.10	0.79	0.25
	360.01	569.38	1.933	0.0013	1.5	1.10	0.97	0.25
CD-8	0.00	365.23	0.469	0.0067	1.5	0.50	0.41	0.25
CD-9	0.00	298.98	0.429	0.0067	1.5	0.50	0.39	0.25
CD-10	0.00	219.98	0.597	0.0020	1.5	0.70	0.56	0.25
	219.98	516.13	0.597	0.0010	1.5	0.70	0.66	0.25
CD-11	0.00	200.81	0.229	0.0100	1.5	0.30	0.30	0.25
CD-12	0.00	189.11	0.330	0.0100	1.5	0.40	0.33	0.25
CD-13	0.00	40.01	0.389	0.0100	1.5	0.40	0.36	0.25
	40.01	137.43	0.389	0.0029	1.5	0.40	0.48	0.25
CD-14	0.00	225.02	0.696	0.0050	1.5	0.60	0.50	0.25
CD-15	0.00	152.58	0.376	0.0020	1.5	0.50	0.49	0.25
CD-16	0.00	225.36	0.588	0.0050	1.5	0.60	0.47	0.25

Drainage Name	Reach (m)		Flow (m <sup>3</sup> /s)	Bed slope, S (m/m)	Side slope, m	Bed width, B(m)	FSD, D(m)	FB(m)
	From	To						
CD-17	0.00	70.30	0.374	0.0100	1.5	0.40	0.36	0.25
CD-18	1.00	373.60	0.700	0.0050	1.5	0.60	0.51	0.25
CD-19	0.00	140.00	0.512	0.0067	1.5	0.50	0.43	0.25
	140.00	416.22	0.512	0.0017	1.5	0.50	0.59	0.25
CD-20	0.00	125.20	0.349	0.0020	1.5	0.60	0.45	0.25
CD-21	0.00	51.08	0.187	0.0050	1.5	0.40	0.30	0.25
CD-22	0.00	47.20	0.257	0.0010	1.5	0.50	0.48	0.25
CD-23	0.00	168.81	0.601	0.0020	1.5	0.70	0.56	0.25
CD-24	0.00	51.62	0.348	0.0050	1.5	0.60	0.36	0.25
CD-25	0.00	99.67	0.842	0.0010	1.5	0.80	0.75	0.25
	99.67	379.76	0.842	0.0025	1.5	0.80	0.60	0.25
	379.76	459.77	0.842	0.0002	1.5	0.80	1.08	0.25
CD-26	0.00	200.02	0.352	0.0010	1.5	0.60	0.54	0.25
CD-27	0.00	39.88	0.352	0.0250	1.5	0.30	0.12	0.25
	39.88	145.62	0.352	0.1000	1.5	0.30	0.09	0.25
CD-28	0.00	60.00	0.472	0.0010	1.5	0.70	0.59	0.25
	60.00	214.89	0.472	0.0033	1.5	0.60	0.46	0.25
CD-29	0.00	199.99	0.413	0.0050	1.5	0.60	0.39	0.25
	199.99	339.77	0.413	0.0020	1.5	0.60	0.49	0.25
	339.77	619.94	0.413	0.0050	1.5	0.60	0.39	0.25
	619.94	640.11	0.413	0.1000	1.5	0.60	0.18	0.25
FD-1	0.00	80.03	0.060	0.0050	1.5	0.30	0.19	0.25
FD-2	0.00	54.10	0.040	0.0050	1.5	0.30	0.15	0.25
FD-3	0.00	80.00	0.176	0.0020	1.5	0.40	0.36	0.25
	80.00	218.90	0.176	0.0050	1.5	0.40	0.29	0.25
FD-4	0.00	187.48	0.150	0.0025	1.5	0.40	0.32	0.25
FD-5	0.00	80.02	0.070	0.0100	1.5	0.30	0.17	0.25
FD-6	0.00	50.09	0.030	0.0050	1.5	0.30	0.13	0.25
FD-7	0.00	129.71	0.025	0.0100	1.5	0.30	0.10	0.25
FD-8	0.00	109.20	0.025	0.0020	1.5	0.30	0.15	0.25
FD-9	0.00	129.73	0.040	0.0010	1.5	0.30	0.23	0.25
FD-10	0.00	289.29	0.190	0.0010	1.5	0.30	0.47	0.25
FD-11	0.00	349.54	0.200	0.0010	1.5	0.50	0.43	0.25
FD-12	0.00	40.00	0.060	0.0010	1.5	0.30	0.28	0.25

Drainage Name	Reach (m)		Flow (m <sup>3</sup> /s)	Bed slope, S (m/m)	Side slope, m	Bed width, B(m)	FSD, D(m)	FB(m)
	From	To						
	40.00	117.31	0.060	0.0040	1.5	0.30	0.20	0.25
FD-13	0.00	169.40	0.045	0.0050	1.5	0.30	0.16	0.25
FD-14	0.00	179.98	0.080	0.0040	1.5	0.30	0.23	0.25
	179.98	189.98	0.080	0.0010	1.5	0.30	0.32	0.25
FD-15	0.00	39.80	0.060	0.0010	1.5	0.30	0.28	0.25
	39.80	148.76	0.060	0.0050	1.5	0.30	0.19	0.25
FD-16	0.00	249.63	0.210	0.0010	1.5	0.50	0.44	0.25
FD-17	0.00	389.10	0.400	0.0010	1.5	0.60	0.57	0.25
	389.10	657.50	0.400	0.0003	1.5	0.70	0.71	0.25
FD-18	0.00	639.62	0.370	0.0010	1.5	0.60	0.55	0.25

## 4 PHYSICAL AND SOCIAL INFRASTRUCTURES

### 4.1 ACCESS ROAD

To carryout operation and maintenance activities of irrigation system effectively and efficiently, and to carry out any development activities within the project area, basic infrastructures especially access road in to the scheme and within the scheme are critically required. For this purpose the size and type of access and service/farm roads which are supposed appropriate for the project are selected and designed. Access road of 4m width and 5 Km length are considered along the main canal. All secondary canals will have 2.5m width access road.

### 4.2 Camping

A camping station for the construction crew such as the contractor and supervisor on the project site is indispensable for efficient implementation of the project. Consequently, consultant's and contractor's residence and/or office which is made from G-32 corrugated iron sheet /CIS/ has been designed. It is internally partitioned with chip wood wall & ceiling and founded on cemented floor. The rooms are designed such that they are well ventilated as they are equipped with window and door of same material as shown on the drawing.

The station has also comprised of 5m\*5m store which is constructed from G-32 CIS wall and roof as well as, shower and toilet rooms, Cafeteria and kitchen facility, guard house and Fence works all around the camp of area. Layout of these facilities and their cross section have been presented in the drawing album.

### 4.3 Foot Bridge

These are structures proposed on main canals at foot path crossing sites to allow easy movement of inhabitants in the project area. There are **Six** Foot Bridge structures are provided and design shall be precast reinforced concrete.

*Table 4-1:- Summary of Foot Bridge in the Main Canal*

Location	span Length, (m)	Width (m)	Crossing Method	Remark
0+108	1	2	Pedestrian	Existing Foot Path
0+460	1	2	Pedestrian	Existing Foot Path
1+836	1	2	Pedestrian	Existing Foot Path

Location	span Length, (m)	Width (m)	Crossing Method	Remark
2+751	1	2	Pedestrian	Existing Foot Path
3+026	1	2	Pedestrian	Existing Foot Path
3+329	1	2	Pedestrian	Existing Foot Path

#### 4.4 Washing Basin

In order to protect quality of irrigation water from being contaminated by polluted water from washed clothes, provision of facilities for this purpose beside canals is essential. Accordingly, we have proposed One washing basins of size as indicated on the drawing album to minimize interference with flow of the canals. The washing basin is located near settlement area at 3+316 from head work.

#### 4.5 Cattle Trough / Animal Water Point

Since the River is located near the settlement area, there is no recommended cattle trough for Wataba Baddesa SSIP.