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1 INTRODUCTION

1.1 Background

In Ethiopia, under the prevalent rain-fed agricultural production system, the progressive degradation of the natural resource base, especially in highly vulnerable areas of the highlands coupled with climate variability have aggravated the incidence of poverty and food insecurity. The major source of growth for Ethiopia is still conceived to be the agriculture sector. Hence, this sector has to be insulated from drought shocks through enhanced utilization of the water resource potential of the country, (through development of small-scale irrigation, water harvesting, and on-farm diversification) coupled with strengthened linkages between agriculture and industry (agro-industry), thereby creating a demand for agricultural output. In line with the above, efforts have been made by the government and NOG's to improve the situation in the country in areas of domestic water supply provision, irrigation, watershed management, etc.

The Oromia Irrigation Development Authority(OIDA) initiated Tulcha, Denka, Togona, Calle and Kurkuru Small Scale Irrigation Projects as part of its on-going programmes to develop small scale irrigation works in the region with a view to enhance food security and improve the living standard of the beneficiary farmers.

The Oromia Irrigation Development Authority (OIDA) is playing its role in the development of small scale irrigation projects in the region. Accordingly, Agricultural Growth program (AGP) office has initiated the feasibility and detail study and design of a five-small scale irrigation scheme at Bale and Arsi Zones in Oromia Region. B.B.G Engineering P.L.C has entered agreement to carry out consultancy service of Feasibility Studies and Detailed study and Designs work consultancy services.

Calle 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 area is located in ShirkaDistrict of Arsi Zone, Oromia Region. The head work site located in Solechisa Kebele. The command area also covers similar kebele.

The geographic coordinate of the position of the head work site is 553610.39m Easting, 848860.12m Northing, and 2341m Altitude in UTM Adindan Coordinate system. The command area lies between 849003.2&850596.3 North, between 554304.3&557673.4 Easting and an elevation range of from 2113 to 2309 meters above sea level (masl).

1.1.2 Accessibility

The Project District is accessible by all weathered road and dry weathered road of total 90km from Assela, Zonal Town of Arsi and 8km from Gobessa, capital city of Shirka District. From the total distance about 0.3km is accessible during dry season. Assela is 170km far from Finfinnee.

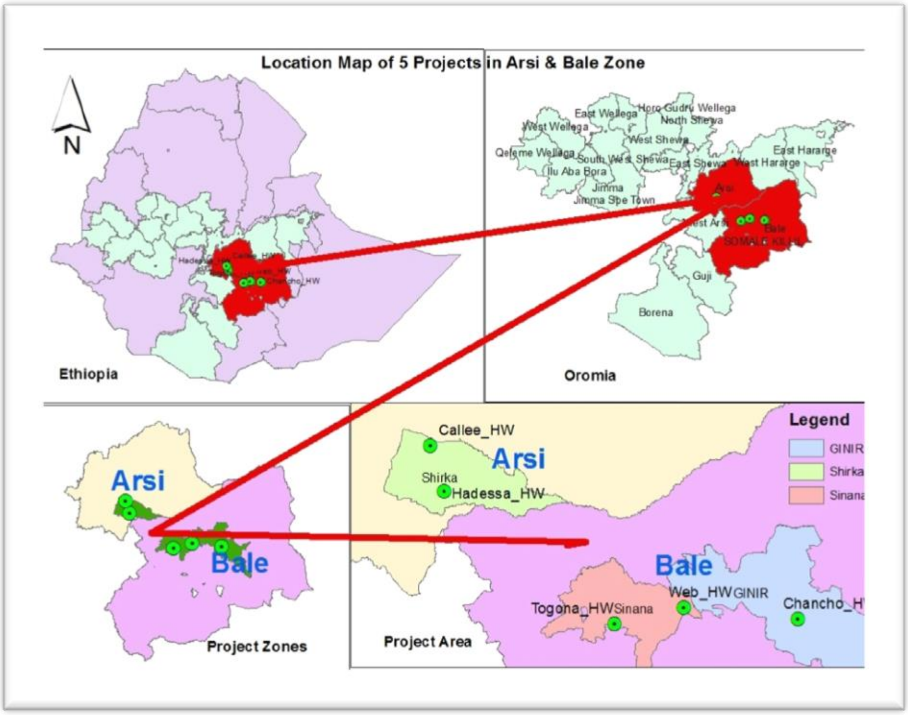


Figure 1-1:- Project Location Map

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 Calle 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 design flows for 50 years return period runoff and catchment area are 98.65m^3/s and 26.93Km^2 respectively.

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 Calle River that in Shirka. 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 **145ha** of land were surveyed and net irrigable area of **103 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 Calle 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

The proposed project has existing system. The canal system is traditional but the head work was constructed by Woreda level in 2000E.C by masonry diversion weir. Parts of the headwork are damaged like head regulator, gate, off taking canal and the weir body. The Weir has no under sluice.

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 Calle River to the entire irrigable area which identified at reconnaissancestudy including the existing weir. 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 Solechisa Kebele for construction of scheme at a reasonable cost and technically feasible structure. But the existing weir site is selected with major maintenance and modification work.

Figure 2-1:Existing Head work site

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. 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 and existing weir type, Broad crested weir is preferable for **Calle Small Scale** Irrigation Project. And from economical point of view the weir would be constructed of masonry externally covered with 300mm 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 **Calle** River is determined based on visual observation of outcrops and the detail is shown in Geotechnical Report Section-III.

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 willhave been recommended. Based on the natural river width, other parameters are determined.

The existing weir has total of 13m but the effective crest length is about 8m. Considering the existing situation and natural river characteristics, the total flow width is recommended to be 12m/ Based on the recommended parameters, other parameters are determined. The Head Work has more than enough head for command area which is bout 38m for the first outlet at 1+002m in MC.

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 98.5m^3/s and the respective tail water depth is 1.53m

Weir Hydraulic Design

Design Discharge over weir is given by

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

Approaching velocity is given by

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

The following Table 2-1 shows the detail weir design calculations.

Table 2-1: Weir height determination

Description	Value	Unit	Remark
Maximum flood in 50 years return period	98.65	m^3/s	
Weir flow Width	12	m	
River bottom elevation (lowest point)	2339.433	m	
Driving Head above canal water surface	0.3	m	
Weir Crest level (Existing)	2341.450	m	
Depth of flow in maximum discharge	**2.86**	m	
Discharge over weir	98.6549	-0.001	Q= C*Le*He^(3/2)
Maximum height flood level	2344.161	m	
Weir height from Command Elevation Respect	**2.02**	m	

Finally, the weir height is fixed to be 2.02m and the weir crest level is fixed to be 2341.45m.

(ii) ***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 CalleSmall Scale Irrigation Project, the working head will have taken as 0.3m.

(iii) ***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 Calle case the afflux is 3.2. Since the weir site is located in slopy area and both banks are stable and are above the HFL, the back water has no significant effect in the proposed project.

Table 2-2:- Weir Design Result Summary

Description	Value	Unit	Remark
Design discharge	98.650	m^3/s	
Weir crest elevation	2341.450	m	
Weir crest length	12	m	
Specific energy head (H)	2.860	m	
Approach Velocity (Va = Q/(L*(P+H)), take the water depth (P=Weir Height)	1.686	m/s	Va= Q/L*(P+H)
Velocity head (hv)	0.145	m	
Tail water depth	1.530	m	From discharge stage curve
U/S HFL	2344.310		
U/S TEL	2344.454	m	
D/S HFL	2340.963	m	
D/S TEL	2341.108	m	
Afflux	3.35	m	

(iv) ***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 double the canal 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 the maximum of the values of discharge obtained by twice the intake capacity or the 10% of the design discharge.
    - ✓ Intake capacity:0.155 $m^3/s \rightarrow$ Sluiceway capacity: 2 x 0.155 = 0.310 m^3/s
    - ✓ 50year design flood over the weir: 98.65m^3/s $\rightarrow$Sluiceway capacity: 0.1 x 98.65 = 9.86 m^3/s
- ❒ Thus minimum sluiceway capacity has to be 9.98m^3/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	1.00	m	Scour sluice to take at least 10-20% of flood flow
Number of Gates	1.00		
Scour sluice total width, Bt	1.00	m	
Gate Height, H	0.80	m	Q=CA(2gh)^0.5, h = D/ce b/n U/s & D/s Water Level, C = 0.6
Depth invert below the main weir crest	4.88	m	
Depth of flow over scour sluice, h1	4.88	m	
Discharge through scour sluice	**4.70**	m^3/s	

The size of under sluice is fixed to pass the discharge of 4.7m^3/s and the bay has dimension of 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 **0.155m³**/s. The head regulator/off taking canal has size of 0.6mx0.5m with bed elevation of 2339.43m.

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

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

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

Description	Value	Unit	Remark
Depth of Canal at Head Regulator, D	0.50	m	
Width of Canal at Head Regulator, W	0.60	m	
Discharge through Head regulator	0.309	m³/s	Q=CA(2gh)^0.5, h = Head over the opening, C = 0.58-0.8
The inverted elevation of Head Regulator	2340.65	m	Weir Crest level-Depth of Canal-Driving Head
The inverted elevation of scoring sluice	2339.43	m	

(vi) Top and bottom width of weir

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

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

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

Where, H: Height of weir (m)

H_e: Specific Energy head (sum of overflow depth & approaching velocity head(m)

σ : specific weight of weir body (=2.0-2.3)

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

Description	Calculated Value	Unit	Adopted Value
He: specific energy head	m	3.004	
P: Height of weir	m	2.02	
σ: Specific weight of weir body		2.3	
Top Width, B	m	2.00	1.1
Bottom Width, L	m	3.40	3.6

(vii) ***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 0.8m masonry divide wallis 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 weirhas 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.

(viii) Hydraulic Jump Calculation

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

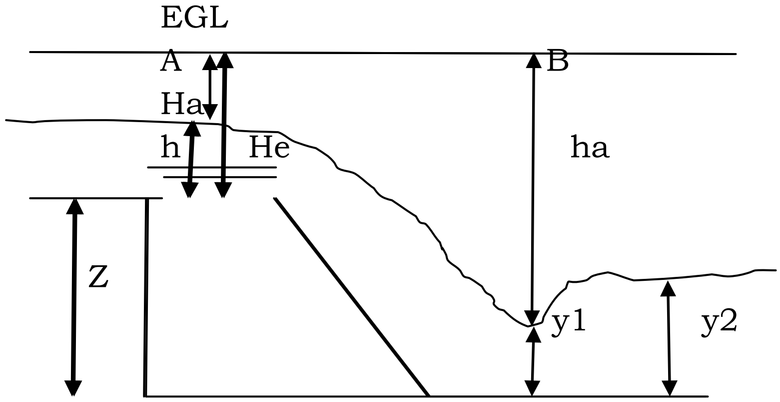


Figure 2-2: Flow over weir body

- ✓ 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

$z + H_e = y_1 + h_a$

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

$$h_a = \frac{q2}{2 * g * y^2}$$

$$V1 = \frac{q}{y1}$$

$$F_r = \frac{V_1}{\sqrt{gy_1}}$$

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

So, from calculation and graph on USBRhydraulic jump length (L) using trial and error the final result is presented in the following table 2-6.

Table 2-6:- Hydraulic Jump Calculation summary

Description	Value	Unit	Remark
Design flow over weir ,Q	98.649	m3/sec	
Design flow over weir ,Q	8.22	m2/sec	
Discharge Intensity, q	2344.31	m	
Flood level U/S	2339.43	m	
River bed level	5.02	m	
Design flow over weir ,Q	2339.43		
Discharge Intensity, q	0.916		
Flood level U/S	5.02		
River bed level	8.975		
U/S total energy	2.994		Z+He= Y1+ha,
River Bed	3.447		4.7280
Pre-jump (Y1)	2337.516		
E1 = Y1+V^2/2g	98.649		
Velocity at the jump	8.22		
Froude number (Fr = q^2/(g*Y1^3)	2344.31		$Fr=(q^2/(y_1{}^3*g))$^0.5
Post jump (Y2)	2339.43		Y2=Y1/2*((1+8Fr^2)^0.5-1)
Cistern level = D/S HFL-Y2	5.02		
Trail1			
EF1	6.939		
Y1	0.746		
EF1=Y1+q^2/(2*9.81*Y1^2), iterating for Y1	6.939		
Velocity at the jump	11.023		
Froude number (Fr = q^2/(g*Y1^3)	4.075		
Post jump (Y2) (Y2=Y1/2*(-1+√(1+8*F1^2))	3.941		
Cistern level = D/S HFL-Y2	2337.022		
Trail2			
EF1	7.433		
Y1	0.716		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.433		
Velocity at the jump	11.480		
Froude number (Fr = q^2/(g*Y1^3)	4.331		
Post jump (Y2) (Y2=Y1/2*(-1+√(1+8*F1^2))	4.043		
Cistern level = D/S HFL-Y2	2336.920		
Trail3			
EF1	7.534		0.000
Y1	0.710		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.534		
Velocity at the jump	11.571		
Froude number (Fr = q^2/(g*Y1^3)	4.383		
Post jump (Y2) (Y2=Y1/2*(-1+√(1+8*F1^2))	4.063		
Cistern level = D/S HFL-Y2	2336.900		
Trail4			
EF1	7.554		
Y1	0.709		

Description	Value	Unit	Remark
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.554		
Velocity at the jump	11.589		
Froude number (Fr = q^2/(g*Y1^3)	4.393		
Post jump (Y2) (Y2=Y1/2*(-1+J(1+8*F1^2))	4.067		
Cistern level = D/S HFL-Y2	2336.896		
Trail5			
EF1	7.558		
Y1	0.709		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.558		
Velocity at the jump	11.592		
Froude number (Fr = q^2/(g*Y1^3)	4.395		
Post jump (Y2) (Y2=Y1/2*(-1+J(1+8*F1^2))	4.067		
Cistern level = D/S HFL-Y2	2336.896		
Trail6			
EF1	7.559		
Y1	0.709		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.559		
Velocity at the jump	11.593		
Froude number (Fr = q^2/(g*Y1^3)	4.395		
Post jump (Y2) (Y2=Y1/2*(-1+J(1+8*F1^2))	4.068		
Cistern level = D/S HFL-Y2	2336.895		
Trail7			
EF1	7.559		0.000
Y1	0.709		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.559		
Velocity at the jump	11.593		
Froude number (Fr = q^2/(g*Y1^3)	4.395		
Post jump (Y2) (Y2=Y1/2*(-1+J(1+8*F1^2))	4.068		
Cistern level = D/S HFL-Y2	2336.895		
Trail8			
EF1	7.559		0.000
Y1	0.709		
EF1=Y1+q^2/(2*9.81*Y1^2) iterating for Y1	7.559		
Velocity at the jump	11.593		
Froude number (Fr = q^2/(g*Y1^3)	4.395		
Post jump (Y2) (Y2=Y1/2*(-1+J(1+8*F1^2))	4.068		
Cistern level = D/S HFL-Y2	2336.895		
The length of horizontal the jump =Lj=5.8*Y2 (From Graph 9.39, USBR)	**23.59**		

Therefore, from the above graph the length of horizontal the jump =Lj=23.59m. Even though the jump length is about 23.59, the head work did not need construction of apron, the river has basaltic bed formation.

(ix) ***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:

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

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*

Since the river bed formation has rock formation, there is no need of downstream and upstream cut off and simple it is proposed by anchoring the structure with rock.

(x) ***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 theHFL 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.

(xi) ***Upper 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 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(ΣH) and vertical forces(ΣV)
iii. Determine the moments of all the forces components about the downstream edge or toe. Find the algebraic sum (ΣMr) of resisting moments and algebraic sum (ΣMh) of overturning moments. Also determine the net moment (ΣM) 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= ΣM/ΣV
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 (ΣMr/ΣMh)
vii. Determine the factor of safety against sliding (μΣV/ΣH)
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*

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

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.

$$Max.BasePressure = \frac{\mathbf{\Sigma V}}{\boldsymbol{B}} * \left(1 + \frac{6 * e}{B}\right)$$

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^3 and for water is 9.81KN/m^3. 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.

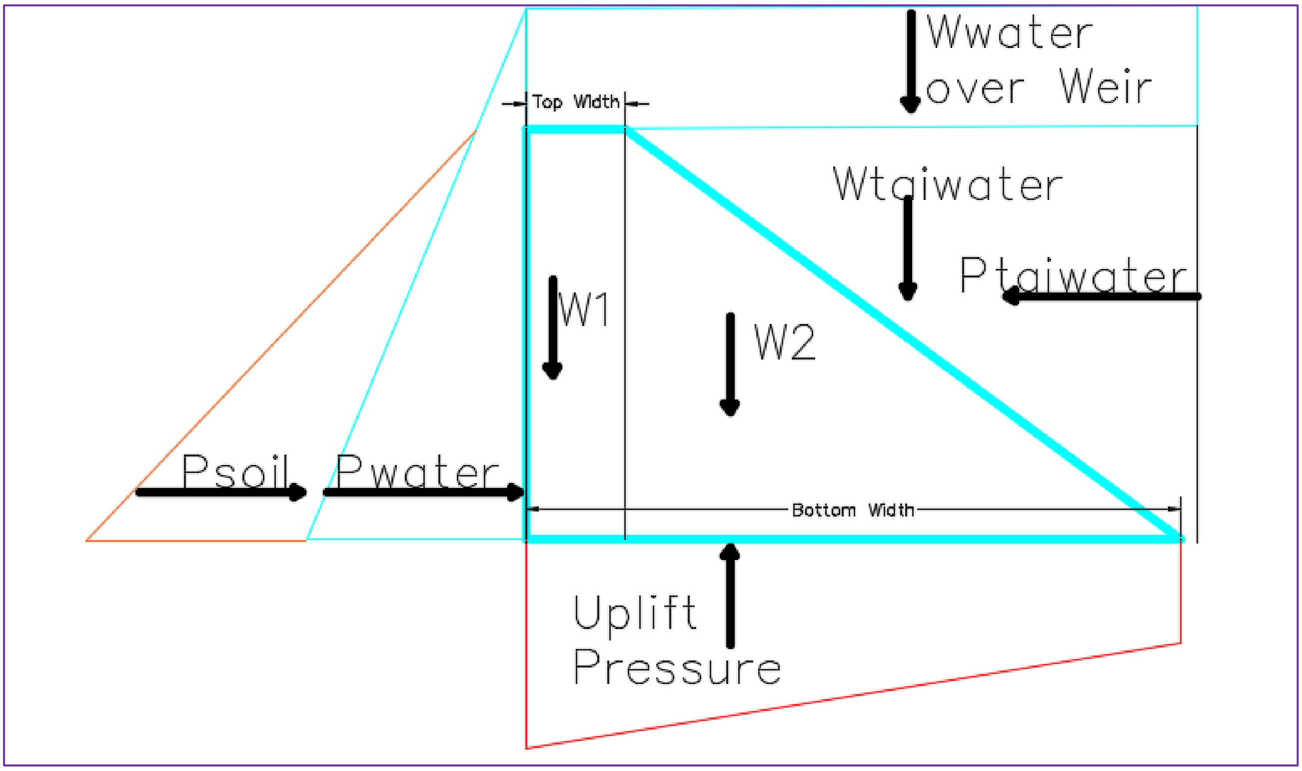


Figure 2-3:- Load Distribution during Minimum Water level

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

CONDITION 1 WHEN WATER IS AT POND LEVEL

Force designation	Magnitude of forces (KN)			Moment at "o"(KN.M)		
	Polygon Area	Vertical	Horizontal	Lever arm (m)	(+M) Resisting	(-M) Disturbing
W1	2.2187	59.90		3.05	182.71	
W2	2.5212	68.07		1.67	113.46	
P_{silt}=1.8*h^2			7.32	0.672		4.92
P_{water}			20.34	0.672		13.68
Uplift pressure		9.89		4.320		42.74
Total		146.04	27.66		320.49	61.34

1. *Factor of safety against overturning,* $F_o = \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = 4.83 > 1.5$, *the structure is safe against overturning.*
2. *Factor of safety against sliding,* $F_s = \mu * \frac{\Sigma V}{\Sigma H} = 3.24 > 1.5$, *the structure is safe against sliding.*
3. *Tension for checking*

$$X_{ave} = \frac{(\Sigma M(+) - \Sigma M(-)}{\Sigma V} = 1.7$$

The eccentricity, (e) $= e = \left(\frac{B}{2} - X_{ave}\right) = 0.10,$

But $\frac{B}{6} = 0.60$, *there for eccentricity (e) =0.1<*$\frac{B}{6}$*=0.60, 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. The allowable bearing pressure is greater than 200KN/m^2.

$$Max.BasePressure = \frac{\boldsymbol{\Sigma V}}{\boldsymbol{B}} * \left(1 + \frac{6 * e}{B}\right) = \mathbf{44.32.} \ll \mathbf{200}\boldsymbol{KN/m^2}$$

Therefore, the stability analysis shows the section under base flow condition is structurally safe.

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.

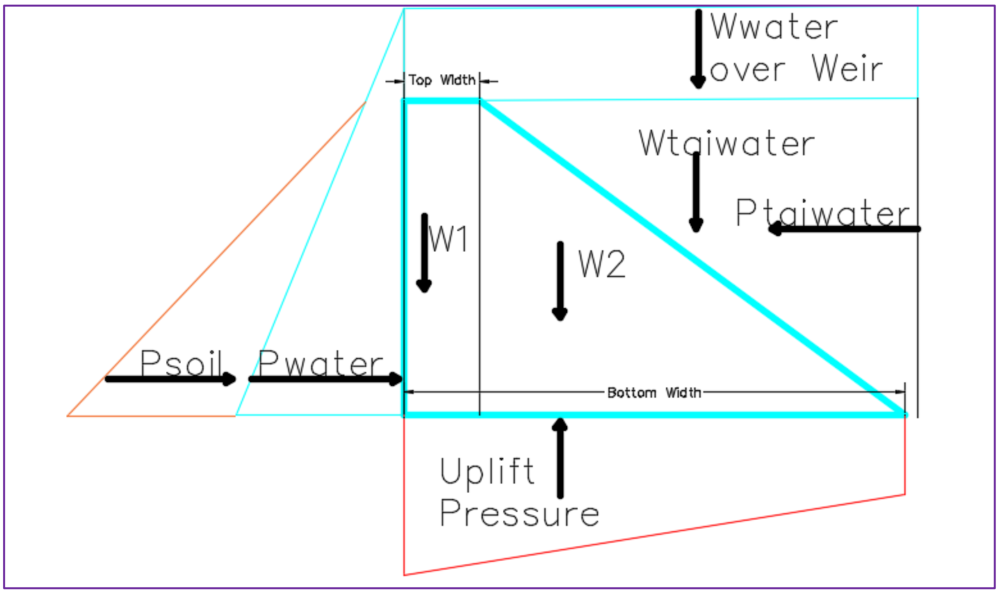


Figure 2-4:- Load Distribution during Minimum Water level

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

Force designation	Magnitude of forces (KN)			Moment at "o"(KN.M)		
	Polygon Area	Vertical	Horizontal	Lever arm, m	(+M) Resisting	(-M) Disturbing
W1	2.2187	59.90		3.05	182.71	
W2	2.5212	68.07		1.67	113.46	
$W_{\text{water over the weir}}$	10.58	103.80		1.85	192.02	
$W_{\text{tail water}}$	1.20	11.77		0.51	6.00	
$P_{\text{Tail water}}$	9.02		11.48	1.02	11.71	
Psilt=1.8*h^2			3.66	0.672		2.46
Pwater			116.65	1.626		189.62
Uplift pressure				2.467		0.00
Total		251.72	108.83		530.23	192.08

1. *Factor of safety against overturning,* $F_o = \frac{\Sigma M\ (+)}{\Sigma M\ (-)} = 2.76 > 1.5$, *the structure is safe against overturning.*
2. *Factor of safety against sliding,* $F_s = \mu * \frac{\Sigma V}{\Sigma H} = 1.50 > 1.5$, *the structure is safe against sliding.*

3. *Tension for checking*

$$X_{ave} = \frac{(\Sigma M(+) - \Sigma M(-)}{\Sigma V} = 1.34$$

The eccentricity, (e) $=e = \left(\frac{B}{2} - X_{ave}\right) = 0.51,$

But $\frac{B}{6} = 0.61$, *there for eccentricity (e) =0.0.51<*$\frac{B}{6}$*=0.61, 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. The allowable bearing pressure is greater than 200KN/m^2.

$$Max.BasePressure = \frac{\mathbf{\Sigma V}}{\boldsymbol{B}} * \left(1 + \frac{6 * e}{B}\right) = \mathbf{123.92} < 200\boldsymbol{KN/m^2}$$

Therefore, the stability analysis shows the section under Peak flow condition is structurally safe.

b) Weir Capping

To protect the weir from the wear and tear due to boulders carried by the flood, a capping of RCC C-25will 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, As = Area of reinforcement,

b = unit width,

d = thickness of slab

fyk = characteristics yield strength of reinforcement, 276 for dia. <=16mm and 400 for dia. >16mm.

Hence provide single reinforcement of Φ12 @ C/C 200mm for main bar and Φ12 @ 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. Thedivide 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.

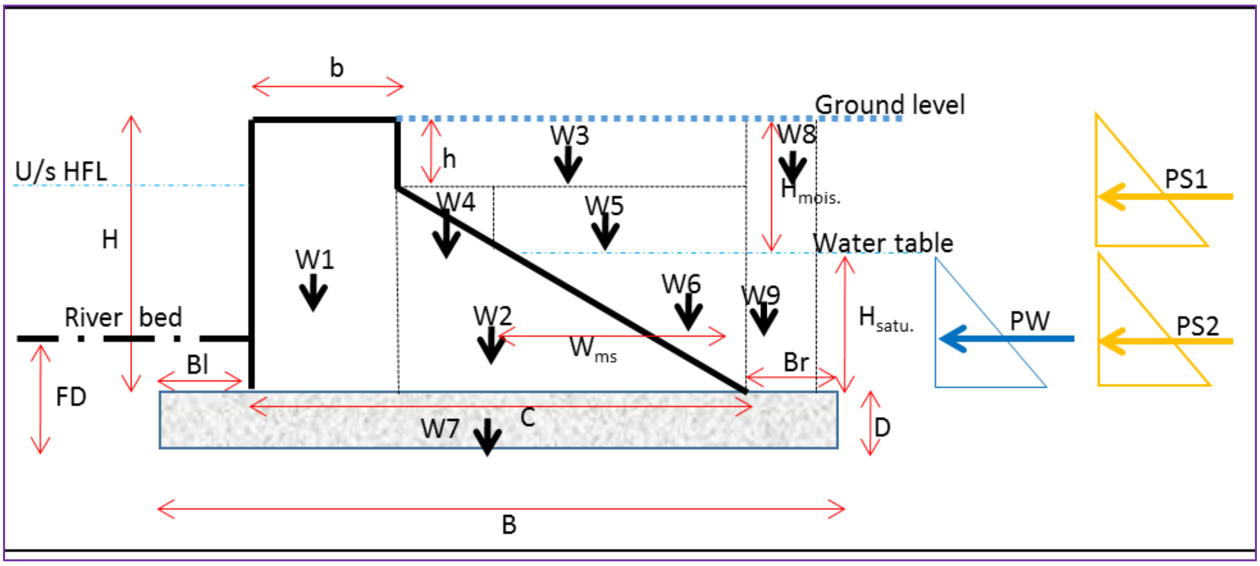
The stability analysis will be carried out taking a unit length of the structure and taking into account the geology of the area. Therefore, the most dominant forces identified are

- *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*

1.0 Data	Unit	Qty
1.1. Hydraulics data		
River bed level	m	2339.433
High flood level	m	2344.310
Free board	m	0.4
Foundation depth, FD	m	0.40
Depth of saturated bottom soil, $H_{satu.}$	m	3.42
Depth of dry Soil upper soil, H_{dry}	m	1.71
Width of saturated soil, $W_{ws.}$	m	1.49
1.2. Material data		
Unit weight of masonry ($g_{masn.}$)	KN/m^3	21.00
Unit weight of bedding concrete ($g_{conc.}$)	KN/m^3	24.00
Dry unit weight of backfill ($g_{Soil.}$)	KN/m^3	16.00
Saturated unit weight of backfill ($g_{sat.}$)	KN/m^3	20.00

Submerged unit weight of backfill (gsub.)	KN/m³	10.20
Unit weight of water (gwat.)	KN/m³	9.80
Angle of internal friction (Ø)	Degree	30.00
Active internal friction coefficient (Ka)		0.33
The friction angle b/n masonry & concrete	Degree	33.00
The friction angle b/n concrete & soil	Degree	20.00
Maximum allowable compressive strength in masonry	N/mm²	1.00
Maximum allowable tensile strength in masonry	N/mm²	0.10
Maximum allowable compressive strength in concrete	N/mm²	20.00
Maximum allowable tensile strength in concrete	N/mm²	3.00
Maximum soil bearing pressure	kN/m²	150.00
1.3.Required dimensions of the retaining wall for stability consideration		
Masonry		
Top width, b	m	0.50
Bottom width, C	m	2.60
Top section height, h	m	0.30
Total height, H=U/S HFL(CBL)- River bed level - transition thickness	m	5.13
Concrete base		
Thickness of concrete base, D=H/8 to H/6	m	0.40
Width of left side concrete base, Bl=D/2 to D	m	0.40
Width of right side concrete base, Br=10 to 15 cm	m	0.15
Total width of concrete base, B=C+Bl+Br	m	3.15

2.0. Loading and satiability analysis



CASE 1 WHEN WATER IS AT MAXIMUM LEVEL

3.0. Stability analysis

SN	Code of load	Load Formula	Load (KN)	Moment arm formula	Moment arm, m	Moment (KNm)
1.0	Self-weight					
1.1	W1	$g_{masn}.bH$	53.84	b/2	0.25	13.46
1.2	W2	$0.5g_{masn.}(C-b)(H-h)$	106.46	b+((C-b)/3)	1.20	127.75
1.3	W7	$g_{conc.}BD$	30.24	C/2+Br	1.45	43.85
			190.54			185.06
2.0	Soil (vertical)					
2.1	W3	$g_{soil.}h(C-b)$	12.60	b+((C-b)/2)	1.55	19.53
2.2	W4	$0.5g_{Soil.}(C-b-W_{ws})(H_{dry})$	8.38	b+(2/3)(C-b-Wws)	0.91	7.62
2.3	W5	$g_{Soil}W_{ws}(H_{dry})$	40.67	$(C-W_{ws})+(W_{ws}/2)$	1.86	75.50
2.4	W6	$0.5g_{satu.}W_{ws}H_{satu.}$	50.84	$(C-W_{ws})+((2/3)W_{ws})$	2.10	106.97
2.4	W8	$g_{satu.}B_rH_{satu.}$	10.26	C+Br/2	2.68	27.43
2.4	W9	$g_{satu.}B_r(H-H_{satu.})$	5.13	C+Br/3	2.68	13.72
			127.87			250.77
3.0	Soil (horizontal)					
3.1	PS	$0.5K_ag_{satu.}H_{satu.}^2$	38.96	$H_{satu.}/3$	1.14	44.39
4.0	Water (horizontal)					
4.1	PWd/s	$0.5g_{wat.}H_{satu.}^2$	57.27	$H_{satu.}/3$	1.14	65.26
	PWu/s	$0.5g_{wat.}H^2$	128.85	H/3	1.71	220.25
5.0	Uplift (masn/conc)					
5.1	PU	$g_{wat.}H_{satu.}(B-Bl)$	92.13	0.5*(B-Bl)	1.38	126.68

1. Overturning	
Sum of stabilizing moment	656.08KNm
Sum of destabilizing moment	236.34KNm
Factor of Safety	2.78 Safe
2. Sliding	
Sum of vertical force (W1+W2+W3+W4+W5+W6-PU1)	355.13KN
Horizontal sliding force (PS+PW)	96.23KN
Horizontal resisting force ($R_v\tan\emptyset$)	230.62KN
Factor of Safety	2.40 Safe
3. Tension	
Net moment = stab. Momnt+destab. momnt	419.75KNm
Net vertical force =Downward - upward	355.13Knm
X = Mr/Rv	1.18m
e = C/2 -x	0.12m
C/6 =	0.43 Safe
4. Direct compressive/tensile stress at the concrete surface	
Direct compressive stress (at the heal) base of concrete	$0.17Nmm^2$
< Pallow=$15N/mm^2$ for C15 which is concrete	
< Pallow=$1N/mm^2$ which is masonry	
Direct compressive/tensile stress (at the toe) base of concrete;	$0.10N/mm^2$
Which is +ve, therefore no tension	

CASE 2 WHEN WATER IS AT MINIMUM POND LEVEL

No.	Code of load	Load Formula	Load (KN)	Moment arm formula	Moment arm (m)	Moment (KNm)
1.0	Self-weight					
1.1	W1	$g_{masn}.bH$	53.84	b/2	0.25	13.46
1.2	W2	$0.5g_{masn.}(C-b)(H-h)$	106.46	b+((C-b)/3)	1.20	127.75
1.3	W7	$g_{conc.}BD$	30.24	C/2+Br	1.45	43.85
			190.54			185.06
2.0	Soil (vertical)					
2.1	W3	$g_{soil.}h(C-b)$	12.60	b+((C-b)/2)	1.55	19.53
2.2	W4	$0.5g_{Soil.}(C-b-W_{ws})(H_{dry})$	8.38	b+(2/3)(C-b-Wws)	0.91	7.62
2.3	W5	$g_{Soil}W_{ws}(H_{dry})$	40.67	$(C-W_{ws})+(W_{ws}/2)$	1.86	75.50
2.4	W6	$0.5g_{satu.}W_{ws}H_{satu.}$	50.84	$(C-W_{ws})+((2/3)W_{ws})$	2.10	106.97
2.4	W8	$g_{satu.}B_rH_{satu.}$	10.26	C+Br/2	2.68	27.43
2.4	W9	$g_{satu.}B_r(H-H_{satu.})$	5.13	C+Br/3	2.68	13.72
			127.87			250.77
3.0	Soil (horizontal)					
3.1	PS	$0.5K_ag_{satu.}H_{satu.}^2$	38.96	$H_{satu.}/3$	1.14	44.39
4.0	Water (horizontal)					
4.1	PWd/s	$0.5g_{wat.}H_{satu.}^2$	57.27	$H_{satu.}/3$	1.14	65.26
5.0	Uplift (masn/conc)					
5.1	PU	$g_{wat.}H_{satu.}(B-Bl)$	92.13	0.5*(B-Bl)	1.38	126.68

1. Overturning	
Sum of stabilizing moment	435.83KNm
Sum of destabilizing moment	236.34KNM
Factor of Safety	1.84 Safe
2. Sliding	
Sum of vertical force (W1+W2+W3+W4+W5+W6-PU1)	226.28KN
Horizontal sliding force (PS+PW)	96.23KN
Horizontal resisting force ($R_v$tanØ)	146.95KN
Factor of Safety	1.53 Safe
3. Tension	
Net moment = stab. Momnt+destab. momnt	199.5KNm
Net vertical force =Downward - upward	226.28Knm
X = Mr/Rv	0.88m
e = C/2 -x	0.42m
C/6 =	0.43 Safe
4. Direct compressive/tensile stress at the concrete surface	
Direct compressive stress (at the heal) base of concrete	0.17Nmm^2
< Pallow=15N/mm^2 for C15 which is concrete	
< Pallow=1N/mm^2 which is masonry	
Direct compressive/tensile stress (at the toe) base of concrete;	0.00N/mm^2
Which is +ve, therefore no tension	

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 1.88m 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

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 145ha but due to water head and infrastructure work the net area is limited to about 103ha.

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 12%, 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 run-off 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 Calle Small ScaleIrrigation Project, the dominant soil type is Clay Soil and the dominant command slope is less than 8%. Therefore, surface Irrigation method is the recommended method of irrigation in the project considering the above factor and also operation and maintenance factors.

3.3.1 Layout system

In preparing the alignment of the conveyance system of Calle 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 103 ha. The Main canal is proposed to irrigate required the left command area. The overall system is sub divided in to 1 Main Canal, 4 Secondary canals and 34 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 45 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.

$$\mathbf{Q} = \mathbf{A} * \mathbf{V} \text{..Eq1}$$

Where:

Q = Discharge (m^3/s)
A = Average of cross-section (m^2)
V = Average velocity (m/s)

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

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

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 pudding 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 Calle Small Scale Irrigation Project the soil is clay soil, the second cross-section type can be applied. The depth, d, varies from 10-30 cm.

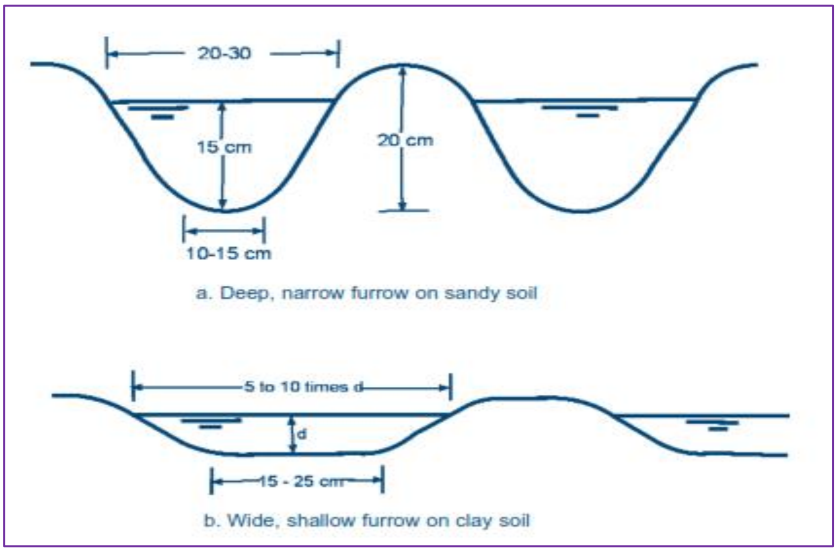


Figure 3-1 Furrow shape depending on soil type

(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 60m.

(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 across contour and hence the minimum slope is 0.5%.

(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 2.5litters per second. To obtain the most uniform irrigation, the largest stream of water that will not because 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 withoutsuffixes.

- 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 a secondary canal are named with Two suffixes. Which describes below.

- TC1-1 = Tertiary Canal One off taking from secondary Canal One
- TC2-1 = Tertiary Canal Two off taking from Secondary Canal One
- TC1-2 = Tertiary Canal One off taking from Secondary Canal Two

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

3.3.4 Main Canal Geological and Geotechnical Investigation

Geotechnicalpropertyofthemaincanal/commandoftheirrigationareacanbeclassified intotwogeotechnicalregions.TheyareRegionbetweenheadworkuptonearthebridge (mainroad)andRegionbetween steel bridge(mainroad)uptotheendofthe catchment ofcommandarea.Thedetail Geotechnical Study is shown in Section-III.

3.3.5 Crop Water Requirement

The crop water requirement/***design supply for the project is <u>1.5 lit/sec/ha</u>*** in the month of January is required with the assumption of a daily irrigation cycle of 10 hours and rotational flow in the main canals. Therefore, the total irrigation water required to satisfy net irrigation command area103 ha of land will be calculated from the formula,

Q= Duty*area

Where

Q is discharge in lit/sec

Duty= flow in lit.sec/ha=1.5 lit/sec/ha*103ha

A=area in ha=103ha

Q=1.5 lit/sec/ha*103ha=154.5 lit/sec

3.3.6 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 182 ha gross surveyed area and has net command area of 103ha. The command area has steep and gentle slopes, specially the secondary canal crosses steep slope..The command area covers minimum and maximum elevation of 2309 and 2113 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:

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

Where: Q = discharge (m3/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 (SC2) off takes. The design discharge of the main canals at 10 hrs 0.155m^3/s and has a total length of 5.11km. The longitudinal slopes of the main canal varies from steep to gentle slope. The main canal follows the existing traditional canal.

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 = 155l/s
- ✓ Longitudinal Slope, S =Variable
- ✓ 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.

- ✓ Discharge, Q = Duty *Command Area
- ✓ Flow area, A = b *d
- ✓ Wetted perimeter, P = b + 2d
- ✓ Hydraulic radius, R = A/p
- ✓ n = 0.018
- ✓ S = Variable

Assuming Bed width is 0.55m Using manning's formula, D is calculated by trial and error until the provided canal section can convey the required discharge.

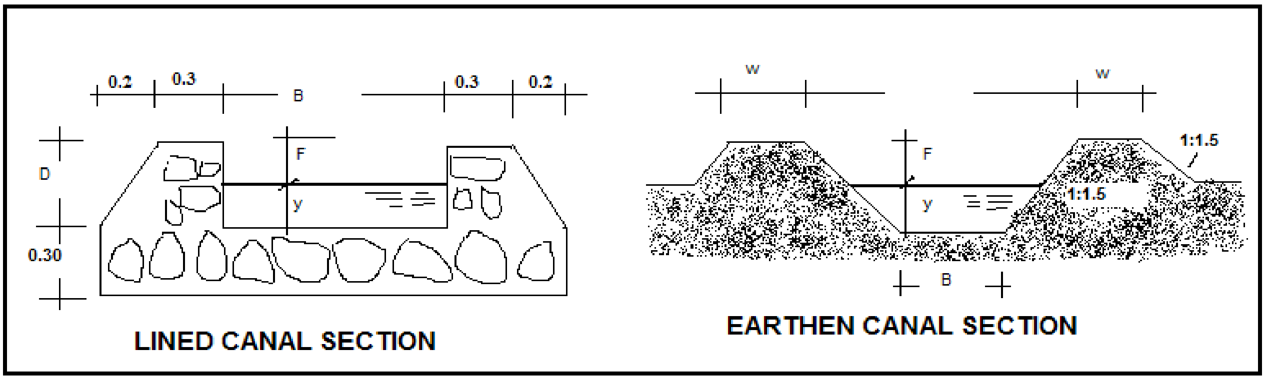


Figure 3-2: Lined and Earthen canal section

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

Chainage (m)	Structure	Irrigable Area (ha)	Q (m^3/s)	B (m)	FSD (m)	FB (m)	Total Depth (m)	Slope	V (m/s)
0	HW	103	0.155	0.55	0.30	0.25	0.55	0.005	1.08
100	Flume1	103	0.155	0.55	0.30	0.25	0.55	0.005	1.08
159	FP	103	0.155	0.55	0.30	0.25	0.55	0.005	1.08
389	Superpassage1	103	0.155	0.55	0.30	0.25	0.55	0.005	1.08
510	0	103	0.155	0.55	0.30	0.25	0.55	0.050	3.40
580	0	103	0.155	0.55	0.30	0.25	0.55	0.100	4.81
610	0	103	0.155	0.55	0.30	0.25	0.55	0.025	2.41
711	0	103	0.155	0.55	0.30	0.25	0.55	0.071	4.07
761	Superpassage2	103	0.155	0.55	0.30	0.25	0.55	0.071	4.07
841	0	103	0.155	0.55	0.30	0.25	0.55	0.020	2.15
872	Superpassage3	103	0.155	0.55	0.30	0.25	0.55	0.020	2.15
952	0	103	0.155	0.55	0.30	0.25	0.55	0.100	4.81
982	0	103	0.155	0.55	0.30	0.25	0.55	0.020	2.15
1002	FC1	100	0.149	0.55	0.30	0.25	0.55	0.020	2.15
1053	0	100	0.149	0.55	0.30	0.25	0.55	0.005	1.08
1113	0	100	0.149	0.55	0.30	0.25	0.55	0.071	4.07
1224	0	100	0.149	0.55	0.30	0.25	0.55	0.067	3.93
1325	0	100	0.149	0.55	0.30	0.25	0.55	0.017	1.97
1405	0	100	0.149	0.55	0.30	0.25	0.55	0.040	3.05
1506	0	100	0.149	0.55	0.30	0.25	0.55	0.025	2.41
1757	Flume2	100	0.149	0.55	0.30	0.25	0.55	0.025	2.41
1795	FP	100	0.149	0.55	0.30	0.25	0.55	0.050	3.40
1895	0	100	0.149	0.55	0.30	0.25	0.55	0.067	3.93
1966	SC1	96	0.145	0.50	0.20	0.25	0.45	0.067	3.32
2127	0	96	0.145	0.50	0.20	0.25	0.45	0.050	2.87
2135	Superpassage4	96	0.145	0.50	0.20	0.25	0.45	0.050	2.87
2216	0	96	0.145	0.50	0.20	0.25	0.45	0.020	1.82
2267	FC2	96	0.143	0.50	0.20	0.25	0.45	0.083	3.71
2326	Superpassage5	96	0.143	0.50	0.20	0.25	0.45	0.083	3.71
2377	0	96	0.143	0.50	0.20	0.25	0.45	0.020	1.82
2437	0	96	0.143	0.50	0.20	0.25	0.45	0.050	2.87
2498	0	96	0.143	0.50	0.20	0.25	0.45	0.025	2.03
2528	SC2	69	0.103	0.50	0.20	0.25	0.45	0.025	2.03
2568	0	69	0.103	0.50	0.20	0.25	0.45	0.025	2.03
2578	0	69	0.103	0.50	0.20	0.25	0.45	0.050	2.87
2760	0	69	0.103	0.50	0.20	0.25	0.45	0.025	2.03
2941	FP	69	0.103	0.50	0.20	0.25	0.45	0.025	2.03
3032	0	69	0.103	0.50	0.20	0.25	0.45	0.029	2.17
3233	FP	69	0.103	0.50	0.20	0.25	0.45	0.029	2.17
3334	0	69	0.103	0.50	0.20	0.25	0.45	0.033	2.34
3637	FP	69	0.103	0.50	0.20	0.25	0.45	0.033	2.34
3647	0	69	0.103	0.50	0.20	0.25	0.45	0.017	1.66
3796	SC3	36	0.054	0.40	0.15	0.25	0.40	0.017	1.39
4000	0	36	0.054	0.40	0.15	0.25	0.40	0.025	1.71
4293	0	36	0.054	0.40	0.15	0.25	0.40	0.017	1.39
4343	FP	36	0.054	0.40	0.15	0.25	0.40	0.017	1.39
4454	0	36	0.054	0.40	0.15	0.25	0.40	0.050	2.41
4656	0	36	0.054	0.40	0.15	0.25	0.40	0.020	1.53
4767	Superpassage6	36	0.054	0.40	0.10	0.25	0.35	0.020	1.29
4797	FP	36	0.054	0.40	0.10	0.25	0.35	0.020	1.29
5110	SC4	36	0.054	0.40	0.15	0.25	0.40	0.020	1.53

Note: HW- Head Work ,SC- secondary canal off taking point, FC – Field Canal, FP - Foot path.

There are 4 division boxes (DB) and 2 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 Two secondary canals 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 3-2 below.

Table 3-2: Hydraulics Parameters of Secondary Canal

Chainage (m)	SC Name	Structure	Area (ha)	Q (m^3/s)	B (m)	FSD (m)	FB (m)	D(m)	Lined Type	Slope	V (m/s)
0	SC1	OFFTAKE	3.2	0.025	0.30	0.05	0.30	0.35	RCC Lined	0.303	3.43
70	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	RCC Lined	0.385	3.86
121	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	RCC Lined	0.417	4.02
151	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	RCC Lined	0.37	3.79
211	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	RCC Lined	0.286	3.33
238	SC1	TC1-1&TC2-1	3.2	0.025	0.30	0.05	0.30	0.35	Masonry	0.143	2.35
291	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	Masonry	0.091	1.88
355	SC1	TC3-1&TC4-1	3.2	0.025	0.30	0.05	0.30	0.35	Masonry	0.091	1.88
412	SC1	0	3.2	0.025	0.30	0.05	0.30	0.35	Masonry	0.118	2.13
482	SC1	TC5-1	3.2	0.025	0.30	0.05	0.30	0.35	Masonry	0.118	2.13
0	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	RCC Lined	0.385	4.03
73	SC2	TC1-2&TC2-2	26.7	0.040	0.40	0.05	0.30	0.35	RCC Lined	0.385	4.03
117	SC2	TC3-2&TC4-2	26.7	0.040	0.40	0.05	0.30	0.35	RCC Lined	0.238	3.17
184	SC2	TC5-2&TC6-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.100	2.05
209	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.111	2.17
299	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	RCC Lined	0.167	2.65
339	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.05	1.45
403	SC2	TC7-2&TC8-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.01	0.65
408	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	RCC Lined	0.2	2.91
468	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.091	1.96
500	SC2	TC9-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.091	1.96
581	SC2	TC10-2&TC11-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.1	2.05
657	SC2	TC12-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.1	2.05
677	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.111	2.17
712	SC2	TC13-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.111	2.17
787	SC2	0	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.091	1.96
817	SC2	TC14-2	26.7	0.040	0.40	0.05	0.30	0.35	Masonry	0.091	1.96
0	SC3	0	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.476	4.93
50	SC3	0	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.4	4.52
101	SC3	TC1-3&TC2-3	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.4	4.52
121	SC3	0	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.323	4.06
161	SC3	TC3-3	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.323	4.06

Chainage (m)	SC Name	Structure	Area (ha)	Q (m^3/s)	B (m)	FSD (m)	FB (m)	D(m)	Lined Type	Slope	V (m/s)
192	SC3	0	32.5	0.049	0.40	0.06	0.30	0.36	RCC Lined	0.222	3.37
232	SC3	0	32.5	0.049	0.40	0.06	0.30	0.36	Masonry	0.091	2.16
239	SC3	TC4-3&TC5-3	32.5	0.049	0.40	0.06	0.30	0.36	Masonry	0.091	2.16
353	SC3	TC6-3&TC7-3	32.5	0.049	0.40	0.06	0.30	0.36	Masonry	0.091	2.16
0	SC4	0	36.3	0.054	0.40	0.07	0.30	0.37	RCC Lined	0.25	3.86
20	SC4	TC1-4&TC2-4	36.3	0.054	0.40	0.07	0.30	0.37	RCC Lined	0.25	3.86
91	SC4	TC3-4&TC4-4	36.3	0.054	0.40	0.07	0.30	0.37	RCC Lined	0.25	3.86
139	SC4	0	36.3	0.054	0.40	0.07	0.30	0.37	RCC Lined	0.111	2.58
207	SC4	TC5-4&TC6-4	36.3	0.054	0.40	0.07	0.30	0.37	RCC Lined	0.111	2.58
238	SC4	0	36.3	0.054	0.40	0.07	0.30	0.37	Masonry	0.063	1.93
386	SC4	TC7-4&TC8-4	36.3	0.054	0.40	0.07	0.30	0.37	Masonry	0.063	1.93

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 secondary canal and supplies irrigation water to field canals and run nearly as a contour canal. In the system layout there are 8 tertiary canals. 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 3-3 below.

Table 3-3: Hydraulics Parameters of Tertiary Canals

S.No.	SC Name	Chainage	TC Name	TC Length	TC Q (l/s)	FC Name	B (m)	FSD (m)	FB (m)	Total Depth (m)
1	SC1	0.0	TC1-1	40.39	15.0	Turn out	0.35	0.10	0.25	0.35
2	SC1	40.4	TC1-1		15.0	FC1-1-1	0.35	0.10	0.25	0.35
3	SC1	0.0	TC2-1	93.46	15.0	Turn out	0.35	0.10	0.25	0.35
4	SC1	93.5	TC2-1		15.0	FC1-2-1	0.35	0.10	0.25	0.35
5	SC1	0.0	TC3-1	30.94	15.0	Turn out	0.35	0.10	0.25	0.35
6	SC1	30.9	TC3-1		15.0	FC1-3-1	0.35	0.10	0.25	0.35
7	SC1	0.0	TC4-1	65.22	15.0	Turn out	0.35	0.10	0.25	0.35
8	SC1	65.2	TC4-1		15.0	FC1-4-1	0.35	0.10	0.25	0.35
9	SC1	0.0	TC5-1	73.06	15.0	Turn out	0.35	0.10	0.25	0.35
10	SC1	73.1	TC5-1		15.0	FC1-5-1	0.35	0.10	0.25	0.35
11	SC2	0.0	TC1-2	11.18	15.0	Turn out	0.35	0.10	0.25	0.35
12	SC2	11.2	TC1-2		15.0	FC1-1-2	0.35	0.10	0.25	0.35
13	SC2	0.0	TC2-2	25.47	15.0	Turn out	0.35	0.10	0.25	0.35
14	SC2	25.5	TC2-2		15.0	FC1-2-2	0.35	0.10	0.25	0.35
15	SC2	0.0	TC3-2	48.31	15.0	Turn out	0.35	0.10	0.25	0.35
16	SC2	48.3	TC3-2		15.0	FC1-3-2	0.35	0.10	0.25	0.35
17	SC2	0.0	TC4-2	49.97	15.0	Turn out	0.35	0.10	0.25	0.35
18	SC2	50.0	TC4-2		15.0	FC1-4-2	0.35	0.10	0.25	0.35
19	SC2	0.0	TC5-2	41.38	15.0	Turn out	0.35	0.10	0.25	0.35
20	SC2	41.4	TC5-2		15.0	FC1-5-2	0.35	0.10	0.25	0.35
21	SC2	0.0	TC6-2	72.09	15.0	Turn out	0.35	0.10	0.25	0.35
22	SC2	20.4	TC6-2		15.0	FC1-6-2	0.35	0.10	0.25	0.35
23	SC2	72.1	TC6-2		15.0	FC2-6-2	0.35	0.10	0.25	0.35
24	SC2	0.0	TC7-2	272.99		Turn out	0.35	0.10	0.25	0.35
25	SC2	58.6	TC7-2		15.0	FC1-7-2	0.35	0.10	0.25	0.35
26	SC2	175.9	TC7-2		15.0	FC2-7-2	0.35	0.10	0.25	0.35
27	SC2	273.0	TC7-2		15.0	FC3-7-2	0.35	0.10	0.25	0.35
28	SC2	0.0	TC8-2	107.63	15.0	Turn out	0.35	0.10	0.25	0.35
29	SC2	43.1	TC8-2		15.0	FC1-8-2	0.35	0.10	0.25	0.35
30	SC2	107.6	TC8-2		15.0	FC2-8-2	0.35	0.10	0.25	0.35
31	SC2	0.0	TC9-2	51.72	15.0	Turn out	0.35	0.10	0.25	0.35

S.No.	SC Name	Chainage	TC Name	TC Length	TC Q (l/s)	FC Name	B (m)	FSD (m)	FB (m)	Total Depth (m)
32	SC2	51.7	TC9-2		15.0	FC1-9-2	0.35	0.10	0.25	0.35
33	SC2	0.0	TC10-2	273.11	15.0	Turn out	0.35	0.10	0.25	0.35
34	SC2	39.0	TC10-2		15.0	FC1-10-2	0.35	0.10	0.25	0.35
35	SC2	156.0	TC10-2		15.0	FC2-10-2	0.35	0.10	0.25	0.35
36	SC2	273.1	TC10-2		15.0	FC3-10-2	0.35	0.10	0.25	0.35
37	SC2	0.0	TC11-2		15.0	Turn out	0.35	0.10	0.25	0.35
38	SC2	50.3	TC11-2		15.0	FC1-11-2	0.35	0.10	0.25	0.35
39	SC2	0.0	TC12-2	400.82	15.0	Turn out	0.35	0.10	0.25	0.35
40	SC2	60.0	TC12-2		15.0	FC1-12-2	0.35	0.10	0.25	0.35
41	SC2	221.0	TC12-2		15.0	FC2-12-2	0.35	0.10	0.25	0.35
42	SC2	361.5	TC12-2		15.0	FC3-12-2	0.35	0.10	0.25	0.35
43	SC2	400.8	TC12-2		15.0	FC4-12-2	0.35	0.10	0.25	0.35
44	SC2	0.0	TC13-2	56.18	15.0	Turn out	0.35	0.10	0.25	0.35
45	SC2	56.2	TC13-2		15.0	FC1-13-2	0.35	0.10	0.25	0.35
46	SC2	0.0	TC14-2	423.95	15.0	Turn out	0.35	0.10	0.25	0.35
47	SC2	80.9	TC14-2		15.0	FC1-14-2	0.35	0.10	0.25	0.35
48	SC2	242.3	TC14-2		15.0	FC2-14-2	0.35	0.10	0.25	0.35
49	SC2	343.0	TC14-2		15.0	FC3-14-2	0.35	0.10	0.25	0.35
50	SC2	423.9	TC14-2		15.0	FC4-14-2	0.35	0.10	0.25	0.35
51	SC3	0.0	TC1-3	457.72	15.0	Turn out	0.35	0.10	0.25	0.35
52	SC3	39.8	TC1-3		15.0	FC1-1-3	0.35	0.10	0.25	0.35
53	SC3	139.3	TC1-3		15.0	FC2-1-3	0.35	0.10	0.25	0.35
54	SC3	258.7	TC1-3		15.0	FC3-1-3	0.35	0.10	0.25	0.35
55	SC3	358.2	TC1-3		15.0	FC4-1-3	0.35	0.10	0.25	0.35
56	SC3	457.7	TC1-3		15.0	FC5-1-3	0.35	0.10	0.25	0.35
57	SC3	0.0	TC2-3	562.06	15.0	Turn out	0.35	0.10	0.25	0.35
58	SC3	40.2	TC2-3		15.0	FC1-2-3	0.35	0.10	0.25	0.35
59	SC3	160.8	TC2-3		15.0	FC2-2-3	0.35	0.10	0.25	0.35
60	SC3	261.2	TC2-3		15.0	FC3-2-3	0.35	0.10	0.25	0.35
61	SC3	361.6	TC2-3		15.0	FC4-2-3	0.35	0.10	0.25	0.35
62	SC3	461.9	TC2-3		15.0	FC5-2-3	0.35	0.10	0.25	0.35
63	SC3	562.1	TC2-3		15.0	FC6-2-3	0.35	0.10	0.25	0.35
64	SC3	0.0	TC3-3	359.73	15.0	Turn out	0.35	0.10	0.25	0.35
65	SC3	40.0	TC3-3		15.0	FC1-3-3	0.35	0.10	0.25	0.35
66	SC3	160.1	TC3-3		15.0	FC2-3-3	0.35	0.10	0.25	0.35
67	SC3	260.0	TC3-3		15.0	FC3-3-3	0.35	0.10	0.25	0.35
68	SC3	359.7	TC3-3		15.0	FC4-3-3	0.35	0.10	0.25	0.35
69	SC3	0.0	TC4-3	468.71	15.0	Turn out	0.35	0.10	0.25	0.35
70	SC3	40.8	TC4-3		15.0	FC1-4-3	0.35	0.10	0.25	0.35
71	SC3	162.9	TC4-3		15.0	FC2-4-3	0.35	0.10	0.25	0.35
72	SC3	264.9	TC4-3		15.0	FC3-4-3	0.35	0.10	0.25	0.35
73	SC3	366.9	TC4-3		15.0	FC4-4-3	0.35	0.10	0.25	0.35
74	SC3	468.7	TC4-3		15.0	FC5-4-3	0.35	0.10	0.25	0.35
75	SC3	0.0	TC5-3	157.60	15.0	Turn out	0.35	0.10	0.25	0.35
76	SC3	59.2	TC5-3		15.0	FC1-5-3	0.35	0.10	0.25	0.35
77	SC3	157.6	TC5-3		15.0	FC2-5-3	0.35	0.10	0.25	0.35

S.No.	SC Name	Chainage	TC Name	TC Length	TC Q (l/s)	FC Name	B (m)	FSD (m)	FB (m)	Total Depth (m)
78	SC3	0.0	TC6-3	464.19	15.0	Turn out	0.35	0.10	0.25	0.35
79	SC3	60.4	TC6-3		15.0	FC1-6-3	0.35	0.10	0.25	0.35
80	SC3	182.1	TC6-3		15.0	FC2-6-3	0.35	0.10	0.25	0.35
81	SC3	282.5	TC6-3		15.0	FC3-6-3	0.35	0.10	0.25	0.35
82	SC3	383.0	TC6-3		15.0	FC4-6-3	0.35	0.10	0.25	0.35
83	SC3	464.2	TC6-3		15.0	FC5-6-3	0.35	0.10	0.25	0.35
84	SC3	0.0	TC7-3	54.14	15.0	Turn out	0.35	0.10	0.25	0.35
85	SC3	54.1	TC7-3		15.0	FC1-7-3	0.35	0.10	0.25	0.35
86	SC4	0.0	TC1-4	256.29	15.0	Turn out	0.35	0.10	0.25	0.35
87	SC4	59.0	TC1-4		15.0	FC1-1-4	0.35	0.10	0.25	0.35
88	SC4	157.6	TC1-4		15.0	FC2-1-4	0.35	0.10	0.25	0.35
89	SC4	256.3	TC1-4		15.0	FC3-1-4	0.35	0.10	0.25	0.35
90	SC4	0.0	TC2-4	124.17	15.0	Turn out	0.35	0.10	0.25	0.35
91	SC4	62.1	TC2-4		15.0	FC1-2-4	0.35	0.10	0.25	0.35
92	SC4	124.2	TC2-4		15.0	FC2-2-4	0.35	0.10	0.25	0.35
93	SC4	0.0	TC3-4	232.79	15.0	Turn out	0.35	0.10	0.25	0.35
94	SC4	58.1	TC3-4		15.0	FC1-3-4	0.35	0.10	0.25	0.35
95	SC4	135.7	TC3-4		15.0	FC2-3-4	0.35	0.10	0.25	0.35
96	SC4	232.8	TC3-4		15.0	FC3-3-4	0.35	0.10	0.25	0.35
97	SC4	0.0	TC4-4	733.34	15.0	Turn out	0.35	0.10	0.25	0.35
98	SC4	61.3	TC4-4		15.0	FC1-4-4	0.35	0.10	0.25	0.35
99	SC4	163.4	TC4-4		15.0	FC2-4-4	0.35	0.10	0.25	0.35
100	SC4	264.6	TC4-4		15.0	FC3-4-4	0.35	0.10	0.25	0.35
101	SC4	365.7	TC4-4		15.0	FC4-4-4	0.35	0.10	0.25	0.35
102	SC4	467.9	TC4-4		15.0	FC5-4-4	0.35	0.10	0.25	0.35
103	SC4	570.1	TC4-4		15.0	FC6-4-4	0.35	0.10	0.25	0.35
104	SC4	672.2	TC4-4		15.0	FC7-4-4	0.35	0.10	0.25	0.35
105	SC4	733.3	TC4-4		15.0	FC8-4-4	0.35	0.10	0.25	0.35
106	SC4	0.0	TC5-4	211.89	15.0	Turn out	0.35	0.10	0.25	0.35
107	SC4	31.9	TC5-4		15.0	FC1-5-4	0.35	0.10	0.25	0.35
108	SC4	116.6	TC5-4		15.0	FC2-5-4	0.35	0.10	0.25	0.35
109	SC4	211.9	TC5-4		15.0	FC3-5-4	0.35	0.10	0.25	0.35
110	SC4	0.0	TC6-4	712.47	15.0	Turn out	0.35	0.10	0.25	0.35
111	SC4	59.3	TC6-4		15.0	FC1-6-4	0.35	0.10	0.25	0.35
112	SC4	158.3	TC6-4		15.0	FC2-6-4	0.35	0.10	0.25	0.35
113	SC4	257.3	TC6-4		15.0	FC3-6-4	0.35	0.10	0.25	0.35
114	SC4	316.7	TC6-4		15.0	FC4-6-4	0.35	0.10	0.25	0.35
115	SC4	415.7	TC6-4		15.0	FC5-6-4	0.35	0.10	0.25	0.35
116	SC4	514.5	TC6-4		15.0	FC6-6-4	0.35	0.10	0.25	0.35
117	SC4	613.5	TC6-4		15.0	FC7-6-4	0.35	0.10	0.25	0.35
118	SC4	712.5	TC6-4		15.0	FC8-6-4	0.35	0.10	0.25	0.35
119	SC4	0.0	TC7-4	180.03	15.0	Turn out	0.35	0.10	0.25	0.35
120	SC4	40.1	TC7-4		15.0	FC1-7-4	0.35	0.10	0.25	0.35
121	SC4	180.0	TC7-4		15.0	FC2-7-4	0.35	0.10	0.25	0.35
122	SC4	0.0	TC8-4	709.22	15.0	Turn out	0.35	0.10	0.25	0.35
123	SC4	60.9	TC8-4		15.0	FC1-8-4	0.35	0.10	0.25	0.35

S.No.	SC Name	Chainage	TC Name	TC Length	TC Q (l/s)	FC Name	B (m)	FSD (m)	FB (m)	Total Depth (m)
124	SC4	161.9	TC8-4		15.0	FC2-8-4	0.35	0.10	0.25	0.35
125	SC4	404.7	TC8-4		15.0	FC3-8-4	0.35	0.10	0.25	0.35
126	SC4	506.2	TC8-4		15.0	FC4-8-4	0.35	0.10	0.25	0.35
127	SC4	607.7	TC8-4		15.0	FC5-8-4	0.35	0.10	0.25	0.35
128	SC4	709.2	TC8-4		15.0	FC6-8-4	0.35	0.10	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 Night Storage

The base flow of Calle River is about **200l/s** and the required water is about **155l/s** and the downstream release should be minimum of 20% is about **31l/s,** but the project has more than 45l/s for down. Therefore,night storage is not required for this particular project.

3.8 Design of Irrigation Structures

3.8.1 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.

3.8.2 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 1.5m head for lined canals.

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.

The hydraulic design of all drop structure was carried out using vertical drop structure of U.S.B.R type. The design procedures and a sample detailed design for the main canal located at chain age 0+781 is presented below:

a. Critical hydraulic

Design discharge, Q (m^3/s) = 0.155

Height of drop, h (m)

Width of drop, bc = $\frac{0.734Q}{d^{3/2}}(m) = \frac{0.734*0.155}{0.55^{3/2}} = 0.28\ (m)$

Adopted ,bc = 0.40m similar with the canal

Where d = water depth of the canal, m

Critical discharge, q = Q/bc = 0.155/0.40 = 0.39 m2/s

Critical depth, dc = $\left(\frac{q^2}{g}\right)^{1/3}$ = $\left(\frac{0.39^2}{9.81}\right)^{1/3} = 0.25m$

Lip height, a = dc/2 = 0.25 / 2 = 0.125m , adopt a = 0.15 m

b. Stilling basin

Basin width, B = $\frac{18.46\sqrt{Q}}{Q+9.91}m = \frac{18.46\sqrt{0.155}}{0.155+9.91}m = 0.72m$

Basin length, L =$\left(2.5 + \left[\frac{1.1dc}{h} + 0.7\left(\frac{dc}{h}\right)^3\right]\right)\sqrt{hdc}, m$

$$= \left(2.5 + \left[\frac{1.1 * 0.25}{1.0} + 0.7\left(\frac{0.25}{1.0}\right)^3\right]\right)\sqrt{1.0 * 0.25}, m = 1.39m$$

In similar fashion the remaining drop structure located at different canal of this particular project are carried out and the final output is presented in the table 3-4 below.

Table 3-4:- Design of Drops on Main Canal

Canal name	Chainage (m)	D1, (m)	b, m	Q (m^3/sec)	h, m	bc, m	q (m^3/s/ m)	dc, m	a, m	L, m	B, m	Lup (m)	Head Above Crest, m
MC	781	0.55	0.55	0.155	1	0.40	0.39	0.25	0.15	1.39	0.72	0.55	0.17
	811	0.55	0.55	0.155	1	0.40	0.39	0.25	0.15	1.39	0.72	0.55	0.17
	821	0.55	0.55	0.155	1	0.40	0.39	0.25	0.15	1.39	0.72	0.55	0.17
	832	0.55	0.55	0.155	1	0.40	0.39	0.25	0.15	1.39	0.72	0.55	0.17

3.8.3 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.

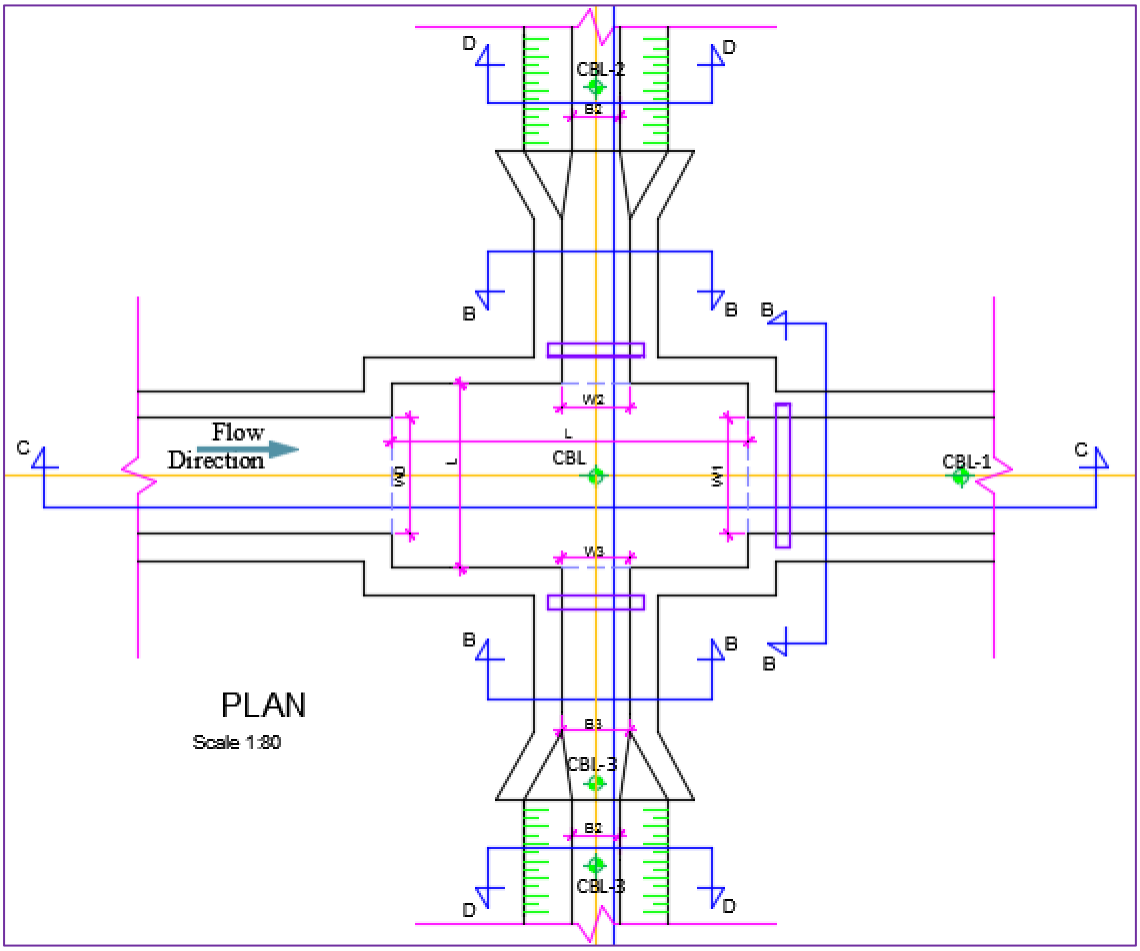


Figure 3-3: Typical Division Box

Qo= Discharge entering in to the division box from u/s canal
Q1 = Discharge of the parent canal that flow to d/s side (Qo-Q2-Q3)
Q2 = Discharge of the off – taking canal to the right side
Q3 = Discharge of the off – taking canal to the left side
B0 = Opening width of the parent canal that flow fromu/s side
B1 = Opening width of the parent canal that flow to d/s side

B2 = Opening width of the off- taking canal to right side
B3 = 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= cbH3/2) 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.

3.8.4 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 39 of such structures arranged on tertiarycanals 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.

3.8.4.1 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 3-5: Design output of Division boxes and Turnouts on Main and Secondary Canals

Canal name	Chainage	Q_0 (L/s)	Q_1 (L/s)	Q_2 (L/s)	Q_3 (L/s)	B, m	d(m)	D(d+fb)	L, m	W_0, m	W_1, m	W_2, m	W_3, m	H0	H1	H2	H3
MC_FC1	1002	155.0	149.4	5.6		0.55	0.3	0.55	1.65	0.55	0.55	0.55	0.00	0.30	0.29	0.03	0.00
MC_SC1	1966	149.4	144.6	4.8		0.5	0.2	0.45	1.40	0.50	0.50	0.50	0.00	0.20	0.31	0.03	0.00
MC_FC2	2267	144.6	143.3	1.4		0.5	0.2	0.45	1.40	0.50	0.50	0.50	0.00	0.20	0.31	0.01	0.00
MC_SC2	2528	143.3	103.2	40.1		0.5	0.2	0.45	1.40	0.50	0.50	0.50	0.00	0.20	0.25	0.13	0.00
MC_SC3	3796	103.2	54.5	48.8		0.4	0.15	0.4	1.20	0.40	0.40	0.40	0.00	0.15	0.19	0.17	0.00
MC_SC4	5110	54.5	54.5	0.0		0.4	0.15	0.4	1.20	0.40	0.40	0.00	0.00	0.15	0.19	0.00	0.00
TC1-1&TC2-1	238	25.0	22.9	0.9	1.2	0.30	0.05	0.35	1.00	0.30	0.30	0.30	0.30	0.05	0.13	0.01	0.02
TC3-1&TC4-1	355	22.9	21.0	1.5	0.5	0.30	0.05	0.35	1.00	0.30	0.30	0.30	0.30	0.05	0.12	0.02	0.01
TC5-1	482	21.0	20.2	0.8		0.30	0.05	0.35	1.00	0.30	0.30	0.30	0.00	0.05	0.12	0.01	0.00
TC1-2&TC2-2	73	40.1	39.2	0.5	0.5	0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.30	0.05	0.15	0.01	0.01
TC3-2&TC4-2	117	39.2	37.8	0.6	0.8	0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.30	0.05	0.15	0.01	0.01
TC5-2&TC6-2	184	37.8	35.4	0.3	2.1	0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.30	0.05	0.14	0.01	0.03
TC7-2&TC8-2	403	35.4	27.0	6.3	2.1	0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.30	0.05	0.12	0.04	0.03
TC9-2	500	27.0	25.5		1.5	0.40	0.05	0.35	1.10	0.40	0.40	0.00	0.30	0.05	0.11	0.00	0.02
TC10-2&TC11-2	581	25.5	17.1	6.3	2.1	0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.30	0.05	0.09	0.04	0.03
TC12-2	657	17.1	8.1		9.0	0.40	0.05	0.35	1.10	0.40	0.40	0.00	0.30	0.05	0.05	0.00	0.07
TC13-2	712	8.1	5.5	2.6		0.40	0.05	0.35	1.10	0.40	0.40	0.40	0.00	0.05	0.04	0.02	0.00
TC14-2	817	5.5	0.0	5.6		0.40	0.05	0.35	1.10	0.40	0.00	0.40	0.00	0.05	0.00	0.04	0.00
TC1-3&TC2-3	101	48.8	27.8	12.3	8.7	0.40	0.06	0.36	1.12	0.40	0.40	0.40	0.30	0.06	0.12	0.07	0.07
TC3-3	161	27.8	21.9		5.9	0.40	0.06	0.36	1.12	0.40	0.40	0.00	0.30	0.06	0.10	0.00	0.05
TC4-3&TC5-3	239	21.9	9.6	10.1	2.3	0.40	0.06	0.36	1.12	0.40	0.40	0.40	0.30	0.06	0.06	0.06	0.03
TC6-3&TC7-3	353	9.6	0.0	8.3	1.4	0.40	0.06	0.36	1.12	0.40	0.40	0.40	0.30	0.06	0.00	0.05	0.02
TC1-4&TC2-4	20	54.5	50.9	2.4	1.2	0.40	0.07	0.37	1.14	0.40	0.40	0.40	0.30	0.07	0.18	0.02	0.02
TC3-4&TC4-4	91	50.9	29.7	3.2	18.0	0.40	0.07	0.37	1.14	0.40	0.40	0.40	0.30	0.07	0.12	0.03	0.11
TC5-4&TC6-4	207	29.7	11.6	3.5	14.7	0.40	0.07	0.37	1.14	0.40	0.40	0.40	0.30	0.07	0.07	0.03	0.09
TC7-4&TC8-4	386	11.6	0.0	3.3	8.3	0.40	0.07	0.37	1.14	0.40	0.00	0.40	0.30	0.07	0.00	0.03	0.06

3.8.5 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;
- ❒ Allfield 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^3/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 h1)
δ	=	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-takesAll gates will be manually operated. Gates which are smaller than 500 mm^2 will have no lifting spindle but chain to prevent from robbery.

3.8.6 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 3-6 Flood Return Periods for Cress-drainage Structure Design

Scheme Type	Structure Type	Location	Return Period
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; Two Flumes/Aqueducts and Six Super passages are recommended.

a. Flume/Aqueduct Structures

Flumes/Aqueducts are used where canals cross over deeply incised streams or rivers where a short crossing will be cheaper than long detour with a super passage. The structure usually has masonry abutments while the flume may be of various materials. For larger spans central piers will be provided to be economical and structurally safe.The hydraulic gradient of the flume should be provided between the canal and the river, dependent on the ground conditions. The canal section upstream and downstream of the aqueduct should be lined.

There are different options to select types of Flumes. From these Plastic, Concrete Pipes and Masonry/mass concrete arch flumes is recommend for small canal and small spans. Reinforced concrete flume with central pier or without central pier is recommended for medium and large canals.Reinforced concrete flume is recommended for the main canal crossing structures in Calle Small Scale Irrigation Project.

Scour depth in the drainage channel should be checked and appropriate foundation provided to accommodate the design flood. The water velocity in the flume should be from 1.0 to 1.5m/s.The clearance will depend upon the relative level of the canal bed and highest flood level of the drainage. The recommended clearance is presented in the Ministry of Water Resource Manual.

Table 3-7:- Minimum Vertical clearances for Rectangular opening

S.N	Discharge of Drainage (m^3/s)	Minimum Vertical Clearance, mm
	Below 30	600
	30 and above but below 300	900

If the minimum clearances specified in the above table are not available, the safety of superstructure shall be ensured against consequence.

In the project, the main canal has two flumes located in different reaches and different spans. The drainage hydrology analysis of is discussed in chapter two of this document. The hydraulic analysis is conducted using Manning's equation for canal size determination and water level in the natural channel.
The summary of the result is presented in the following.

Drainage Name	Chain age	OGL	Q	DBL	Slope	V (m/s)	W (m)	H (m)	CBL (m)	CTL
Flume1	100	2339.568	3.013	2340.898	0.008	1.664	5.000	0.500	2340.148	2340.69
Flume2	1757	2339.568	16.57	2267.639	0.008	2.538	8.000	1.000	2266.989	2267.43

b. 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 Six different locations in the main canal in different sizes. The Location Size is summarized in the following table 3-8.

Table 3-8 Summary of cross drainage structures by Supper Passage

Drainage Name	Chain age	OGL	Q	DBL	Drainage Slope	V	W	H	CBL	CTL
Superpassage1	389	2338.818	0.620	2339.456	0.001	0.346	3.0	0.4	2338.706	2339.3
Superpassage2	761	2325.639	2.500	2325.541	0.001	0.346	3.0	0.4	2324.791	2325.3
Superpassage3	872	2314.851	0.307	2315.700	0.001	0.346	3.0	0.4	2314.950	2315.5
Superpassage4	2135	2267.550	0.813	2267.839	0.001	0.346	3.0	0.4	2267.189	2267.6
Superpassage5	2326	2256.427	0.410	2257.478	0.001	0.346	3.0	0.4	2256.828	2257.3
Superpassage6	4767	2183.852	0.525	2184.533	0.001	0.346	3.0	0.4	2183.883	2184.3

3.8.7 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 MCsand 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, theseculverts 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 beEight 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. Since, the canal crosses the main robe-Goro main road, the provided crossing shall be box culvert and the detail is presented in drawing.

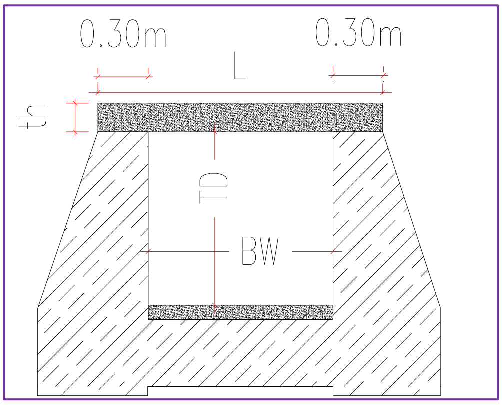


Figure 3-4: Typical Cross Section of the main canal at Foot Path Crossing

3.9 DESIGN OF DRAINAGE SYSTEM

3.9.1 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.

3.9.2 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.

3.9.3 Existing Natural Drainage System

There is no natural gullies and streams identified that the main canal crosses in the project area. To remove the internal drainage three collector, tertiary and filed drains are proposed in the irrigation system.

3.9.4 Description of the Drainage Network Layout of the project

The proposed drainage network for the project of layout system is shown in the general layout of the project. The excess water arising either from irrigation or from excessive rainfall over the irrigated land will be collected by a network of field drains, located at the lower edge of the irrigation plots, perpendicular to the direction of irrigation.

The field drains will be connected to the tertiary and then finally out falling in to collector drains, then to Calle River. The proposed drainage network for the project of water delivery system is shown in the general layout of the scheme and presented in the drawing album.

The interceptor and tertiary drains run nearly parallel to the contours, but the field drains are designed to run across the contours. The interceptor drains are fully external drains while the field and tertiary drains are totally internal drains.

3.9.5 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.

3.9.6 Main Drain Outlet And Collector Drains

In this project area, the main drain outlet is the source river itself. It is located at the end of command area, thus excess water can be discharged through an open surface drain system to different outlets. Similarly Since there is no natural drain across the main conveyance, collector drains have been designed, in order to accommodate 10 years of return period design floods and the result is summarized in table 3-9 below.

Table 3-9: Summary of Hydrology of Identified Collector Drains at MC

Name	Chainage	Flow Length (m)	Area (Ha)	Q (m^3/s)	B (m)	D (m)	S	R	SS	Velocity
CD1	100.4	294.776	5.9	0.127	0.30	0.30	0.005	0.18	1.50	0.75
CD2	388.8	330.31	6.6	0.142	0.30	0.30	0.005	0.18	1.50	0.75
CD3	761	241.37	4.8	0.104	0.30	0.30	0.005	0.18	1.50	0.75
CD4	871.5	498.151	10.0	0.214	0.35	0.35	0.005	0.21	1.50	0.83
CD5	1757	631.95	12.6	0.272	0.40	0.40	0.005	0.24	1.50	0.91
CD6	2135	284.51	5.7	0.122	0.30	0.30	0.005	0.18	1.50	0.75
CD7	2326	1315.65	26.3	0.566	0.50	0.50	0.005	0.30	1.50	1.06
CD8	4767	1391.84	27.8	0.598	0.50	0.50	0.005	0.30	1.50	1.06

3.9.7 Design Of Tertiary Drains

In order to facilitate the drains in the command are a total of 4 tertiary drains running nearly parallel to the contour have been proposed for the entire project area and shown in the layout. All of these tertiary drains are out falling into the nearby stream.

Cross-section of the drainage canals are calculated based on the maximum expected runoff from respective catchments area. Since the amount of the flood increases towards the end, the size of the drain canals should also be increase towards the outlet. Shape of the cross-section of the drainage canal preferred to be trapezoidal.

The design of tertiary drainage (TD) is similar to that of tertiary irrigation canals, except their slope is made to coincide to OGL i.e. the natural water way. There are 4 tertiary drains identified in the designed layout. Design of tertiary drains is done by manning method and tried to make the drain slope more or less to the ground and mostly in cut so as to use excess material as embankment. (Refer table below for details of hydraulic design of each tertiary drain).

Table 3-10: Summary of Tertiary Drain Design

NAME	Flow Length	Area (ha)	Required Discharge	Designed Discharge	B	D	S	n	R	SS	Velocity
CD1-2	272.5	7.4	0.074	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD1-2	88.3	0.4	0.004	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD2-2	108.1	1.4	0.014	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD3-2	86.9	1.4	0.014	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD4-2	297.0	4.2	0.042	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD5-2	92.8	1.0	0.010	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD6-2	434.0	4.2	0.042	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD7-2	134.0	1.4	0.014	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD8-2	459.7	6.0	0.060	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
CD1-3	568.7	5.7	0.057	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
CD2-3	666.2	6.7	0.067	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD1-3	494.3	8.2	0.082	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD2-3	498.5	5.8	0.058	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD3-3	208.1	3.9	0.039	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD4-3	509.6	6.7	0.067	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD5-3	110.4	1.5	0.015	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
CD1-4	254.9	2.5	0.025	0.035	0.30	0.20	0.002	0.03	0.12	1.00	0.35
TD1-4	274.8	1.6	0.016	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD2-4	767.7	0.8	0.008	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD3-4	253.0	2.1	0.021	0.021	0.30	0.15	0.002	0.03	0.09	1.00	0.31
TD4-4	778.3	12.0	0.120	0.120	0.50	0.31	0.002	0.03	0.18	1.00	0.48
TD5-4	265.3	2.3	0.023	0.035	0.30	0.20	0.002	0.03	0.12	1.00	0.35
TD6-4	735.1	9.8	0.098	0.113	0.50	0.30	0.002	0.03	0.18	1.00	0.47

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 2.9 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 **Thirteen**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
0+159	1.0	2	Pedestrian
1+795	1.0	2	Pedestrian
1+966	1.0	2	With SC1 Operation Slab
2+135	1.0	2	Super passage
2+326	1.0	2	Super passage
2+528	1.0	2	With SC2 Operation Slab
2+941	1.0	2	Pedestrian
3+233	1.0	2	Pedestrian
3+367	1.0	2	Pedestrian
3+796	1.0	2	With SC3 Operation Slab
4+343	1.0	2	Pedestrian
4+797	1.0	2	Pedestrian

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 Seven 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 different location in the main canal.

4.5 Cattle Trough / Animal Water Point

The settlement area is far from the river, so 7 cattle troughs are proposed in the main canal route at the same location of washing basin.