

THE NATIONAL REGIONAL STATE OF OROMIA
IRBA-GIRSTU SMALL SCALE IRRIGATION PROJECT

DETAIL ENGINEERING DESIGN DRAFT REPORT

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ACRONYMS

FC	Field Canal
HFL	High Flood Level
MC	Main Canal
OGL	Original Ground Level
Q	Discharge
q	Discharge per meter length
RL	Reduced level
SC	Secondary Canal
TC	Tertiary Canal
TDS	Total soluble salts
TWL	Tail Water Level
UCS	Unconfined Compressive Strength

EXECUTIVE SUMMARY

Irba-Girstu small-scale irrigation project is located in Oromia National Regional State, Bale Zone, Delo-Mena Woreda. The headwork site is located at X-Y coordinates of **595775** m-East and **713129**m -North. The project is about **540**km far from Addis Ababa, **105**km from Zonal Capita Bale-Robe and 5km from Delo-Mena. The project is accessible by four-wheel drive throughout the season. From this, about **110**km main weathered road and **2**km on foot.

Objective of this irrigation project is to bring the proposed area under irrigation based agricultural development to achieve the goal of food security by increasing agricultural production and productivity. The development of the project will transform the prevailing subsistence production system to high value and market oriented production system. The project will also improve livelihood of the project beneficiaries and at the same time create job opportunity for several people living in and around the project area.

Source of water for this irrigation project is Irba-Girstu River, which has perennial flow. Minimum flow of the river is **880**l/s in December 2019 and maximum flow of the river is about **9770**l/sec in August. Based on the given water duty of **1.52**lit/sec/ha (for 24 hours of irrigation), it will be possible to irrigate about **579** hectares of land during dry season but we use only **414** and the remaining discharge is released to the downstream users. Total catchment area of the watershed contributing surface flow to the project river (at weir site) is **52.56**km^2. The peak design flood used to design the diversion weir is **65**m^3/s, which is maximum flood expected once in 50 years of return period.

The major engineering design components of the project are diversion weir with appurtenant structures, camp (residence, office, cafeteria, store, toilet, & shower), and road crossing culvert, division box, drops, turnout, system layout, irrigation and drainage canal profiles.

The main canals are designed to serve for 24-hours, whereas secondary, tertiary and field canals will serve rotationally based on the interest of the project beneficiaries. Depending up on the suitability of the irrigation area, furrow irrigation method of water application is proposed

Total cost of the project is Birr **43,957,821.37** including 15% VAT. During construction it is expected that beneficiary of the project to contribute about **13.93**% of the total project cost which Birr **6,121,170.47**. The project cost per hectare is about **Birr 161,017.66.**

1. INTRODUCTION

Preliminary and detail engineering design part of the Irba-Girstu project address the design of headwork, infrastructures which includes the design of canals, drains, structures, roads etc. Since the command areas are available on both sides, Irba-Girstu headwork consists of intake on both sides of the river. The intake size is fixed based on the size of the land and the peak demand of the crops proposed. The net irrigable area for the right and the left sides are **214.99**ha and **57.72**ha respectively.

2. HEADWORK (DIVERSION WEIR) DESIGN

2.1 Location of the Headwork

Irba-Girstu small-scale irrigation project is located in Oromia National Regional State, Bale Zone, Delo-Mena Woreda. The headwork site is located at X-Y coordinates of 595775 m-East and 713129m -North. The project is about 540km far from Addis Ababa, 105km from Zonal Capita Bale-Robe and 5km from Delo-Mena. The project is accessible by four-wheel drive throughout the season. From this, about 110km main weathered road and 2km on foot.

2.2 Objective of the Project

The overall objective of the project is to bring the proposed area under irrigation based agricultural development to achieve the goal of food security by increasing agricultural production and productivity. The development of this project will transform the prevailing subsistence production system to high value and market oriented production system. The project improves livelihood of the beneficiaries and at the same time create job opportunity for many people living in and around the project area. The implementation of this project in Delo Mena Worda, living standard of the people residing in and around the project area will be improved. To achieve this objective, Oromia Irrigation Development Authority has studied the project for implementation.

2.3 Water Resource

2.3.1 Water Balance

Source of irrigation water for this project is Irba-Girstu River, which is a perennial River source. The total catchment area of the watershed contributing surface flow to the proposed project is 52.56km^2. According to the hydrology study result, the lean flow of Irba-Girstu

River at the study site during December is **880**l/s. On the other hand, the river flow during August is **9770**l/s.

Total project area surveyed was 420 hectares; out of this 273 hectare net irrigation area is identified. As indicated in the system layout and excel data, right main canal-1 & left main canal-1 can irrigate maximum area of **214.99** and **57.72** hectares respectively. Therefore, these canals have been designed to have carrying capacity of **326.78**l/s and **87.73**l/s of flow respectively.

Even though, there is traditional and Hay-Odo modern irrigation development practices at the downstream of the intended project area, there is sufficient flow in the river as well as Irba Kela River at just 2km downstream of the proposed headwork location of the project. Irba Kela is a perennial river that flows from the left side and joins Irba-Girstu at X-Y coordinates of 596516 m-East and 711645m -North. Due to this, there will not be any social conflict arising among the community.

2.3.2 Tail water depth

The tail water depth, d/s water profile is required to carry out the stability analysis of the weir and to design the d/s wing wall. From the weir site surveyed data, River's cross-sectional and longitudinal profiles were produced. Using these profiles, the stage-discharge curve was computed using manning formula. The tail water level in river corresponding to the selected 50 years return period designed flood discharge, 65m^3/s is 1312.31 and its depth is equal to 1.96m.

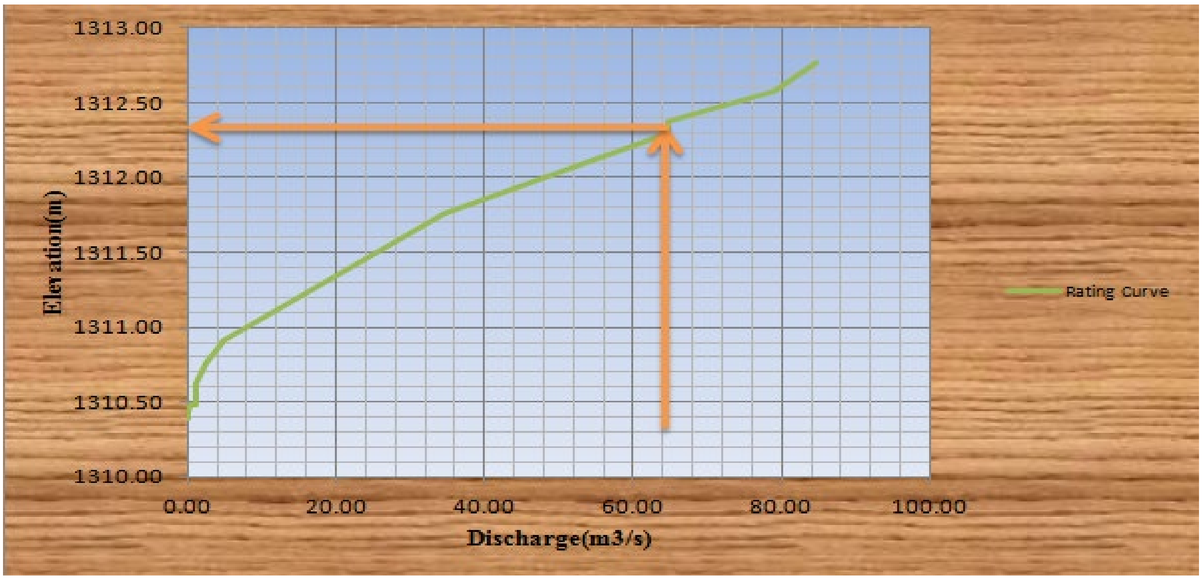

Figure 1 : Stage discharge curve

2.3.3 Design Flood

Result of the hydrology study shows that the peak design flood at Irba-Girstu diversion weir site for 5, 10, 25, 50 and 100 years of return period are 16.50, 26.60, 45.70, 65.30 and 83.40m^3/sec respectively. For the design of small-scale irrigation projects (diversion weirs), it has recommended that the peak design flood once in 50 years should be used and for the design of drainage structure, the flood once in 10-20 years should be used. Hence, to design the proposed diversion weir of Irba-Girstu irrigation project, the peak design flood once in 50 years of return period, which is 65.30m^3/s, has been taken

2.4 Foundation Condition

Foundation material type of the headwork is the main criteria to design the diversion weir. The foundation material should provide sufficient resistance to seepage to prevent excessive loss of water. If the foundation materials are silt and clay, seepage is not the main problem but bearing failure often creates the serious problem. Such kind of foundation material needs treatment i.e. removal of topsoil and filling with selected materials.

To determine foundation bearing capacity and water tightness, geotechnical investigation has been carried out at the headwork site. The finding indicated that the riverbed is taken as fresh basalt bed rock at 1.2m and Cobble to boulder size basalt rock with fragmental rock materials excavation required as cut off trench at the riverbed so as to provide safely against piping failure. The allowable safe bearing capacity of such fresh basalt rock material is presumed to be 1000KN/m^2. The scouring depth is taken geologically as 1.5m from the riverbed level. Provision of aprons shall also be considered against scouring and back erosion. Both the left and right abutments require retaining walls to constrict the river flow within its present river course. See Geological Cross sections of the weir site i.e. figure 2.

This riverbed material has no bearing capacity problem. Due to this, there is no problem of differential settlement. To provide safety against piping failure, upstream and downstream cut-off walls has been provided and checked for exit gradient.

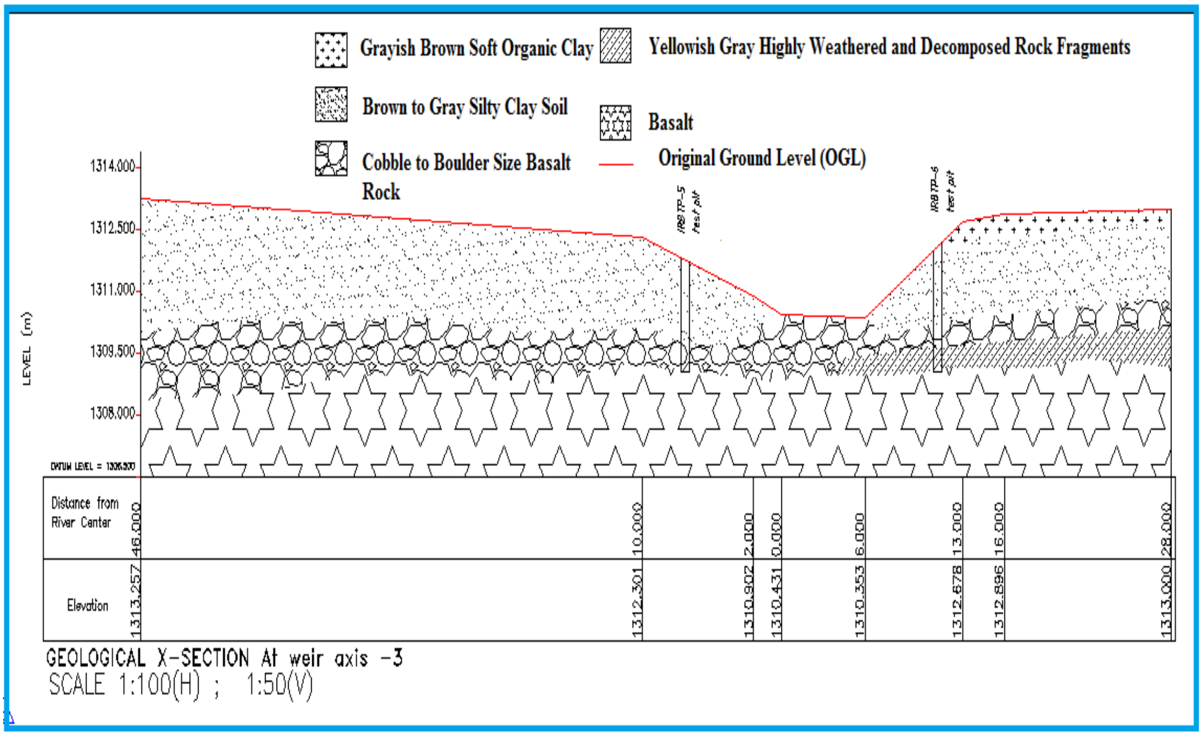

Figure 2 : Geological Cross section of the weir site

2.5 Diversion Weir

Purpose of this diversion weir is designed to raise the water level on its upstream side to create the needed head, so that the desired irrigation water will be diverted in to the canals through the intakes, which are located at the upstream end of the weir on the right and left sides of the riverbanks. The weir crest is also designed to function properly as a spillway for the flood flows.

2.6 Design Considerations

2.6.1 General

Diversion weir must be designed in the same way as all structures carrying normal dead and live loads as well as other loads under the most severe conditions that may exist. A weir structure often continuously acted on by the erosive action of flowing water, and since the structure is frequently placed under adverse condition; adequate safety factor should be considered.

2.6.2 Selection of the headwork

To select the headwork location, the following important criterions were taken in to consideration.

- The selected weir axis has narrow river cross-section, straight reach and the reach has central flow without tendency of scouring or silting;
- The canal alignment which take water from this weir will enable to command the intended project area;
- Construction materials can be easily transported to the headwork site.
- The headwork site has good foundation at moderate depth (see the geology report).

By moving up and down of the river course, we have selected the following three weir site options.

- Option-1 (595780, 712971);
- Option-2 (595782, 713028); and
- Option-3 (595775, 713129).

Therefore based on the above criterion, option-3 was selected.

2.6.3 Types & Shape of the Diversion weir

Weirs differ in types and shapes, but they are designed and constructed to serve the same purpose. Compromise must not be made on the stability of the structure. In deciding, the type and shape of the weir, the following points are considered.

- Skill of the local builders: A weir with a shape that cannot easily be constructed by local workers should not be considered.
- The availability of construction materials in close proximity;
- The financial feasibility of the project;

For this particular project, by considering the above points, and other factors, broad crest concrete weir type is adopted. The wing wall will be stone masonry embedded in cement mortar will be constructed on the lean-concert (C-10) below weir body.

2.6.4 Consideration of forces acting on the diversion weir

The main forces acting on the weir are:

(a) static water pressure of the surface water;

(b) Uplift water pressure;

(c) Self Weight (of the weir body) and water wedge;

(d) Soil reaction at the weir base;

(e) Dynamic Force - Negligible, etc.

2.6.5 Weir crest elevation

Height of the weir crest is decided based on the head requirement of the canal's intake. To fulfill the objective of the intake structures, the following points are considered in deciding the weir crest elevation.

- The weir crest level is set that the water head required to deliver water to the main canal with the design discharge;
- During dry season, the entire flow of the river (low flows) will be diverted, the weir crest elevation set at a level so that the ponded water gives the required head supply to the canal with the design flow;
- The maximum upstream water surface elevation is also considered in selecting the crest elevation. The maximum water level depends on the upstream riverbanks elevations.

The weir crest elevation affects the water profile in two ways:

(a) Height of the crest affects the discharge coefficient and consequently the water head over the weir crest and the backwater curve;

(b) Height of the weir affects the shape and location of the jump and the design of the basin.

To decide the weir crest elevation, the following parameters are taken in to consideration.

Bed level of the main canal at 283m away from the weir (first turn out) = 1310.58m

- Slope of this canal up to 283 = 0.0014
- Head loss on the canal = 0.0014* 283m =0.40m
- Required bed level of the intake canal at the weir = 1310.58+0.40m =1310.98m
- Designed water depth in the main canal = 0.40m
- Assumed head loss across head regulator = 0.15m
- Freeboard for the weir crest= 0.15m

Based on this, elevation of the weir crest is usually determined by the following equation:

Elevation of the weir Crest = intake Canal bed level + designed water depth in the intake canal + head loss across regulator + Freeboard = 1310.98+0.40+0.15+0.15m =1311.68m

Height of the weir body above the riverbed level = 1311.68- 1310.35 =1.32m

2.6.6 Length of the weir crest

Length of the weir crest depends on the physical features (width) of the river at the headwork site. The effect of the weir length on the upstream water head and sedimentation behind the weir must be understood.

- A weir with a longer crest gives a small discharge per unit length and hence the required energy dissipation per meter of the crest is smaller than what is needed for a shorter crest length;
- Constructing a weir longer than the river width causes formation of islands at the upstream side of the weir. As a result, the canal's inlet can become cut-off from the river flow. The formation of the islands upstream of the weir reduces the effective length of the crest.

Generally, the crest length should be taken as the average wetted width during the flood season. During feasibility study, the upstream and downstream of the river cross-section has been examined, and the width at the selected weir site location is measured. It is found to be around 17m. Based on this, length of the weir crest including the under sluice gates has been fixed to be 17m.

2.6.7 Maximum Design Flood

To design the diversion weirs, design flood once in 50 years is recommended for small-scale irrigation project. Therefore, the proposed diversion weir has been designed with 50 years of return period of flood discharge (65m^3/s).

2.6.8 Discharge over the weir

The weir is designed to pass flow of peak flood safely to the downstream without causing any damage on the weir structures. The weir will have two under sluice gate on left and right side of the riverbank i.e. near the main canal intake. However, if incase the gate is closed, the weir should pass the entire maximum flood safely without causing any damage to structures and as well as the land located upstream of the river reach.

Discharge over the weir is generally expressed as

$$Q = CLH_e^{3/2}$$

Where,

Q = Maximum design discharge = 65m^3/s

L = Length of the weir = 17m

He = Height of energy line above the crest = $V^2/2g + H_d$

C = Discharge coefficient. In practice a discharge coefficient, C =1.70 (for broad crested weir) is used.

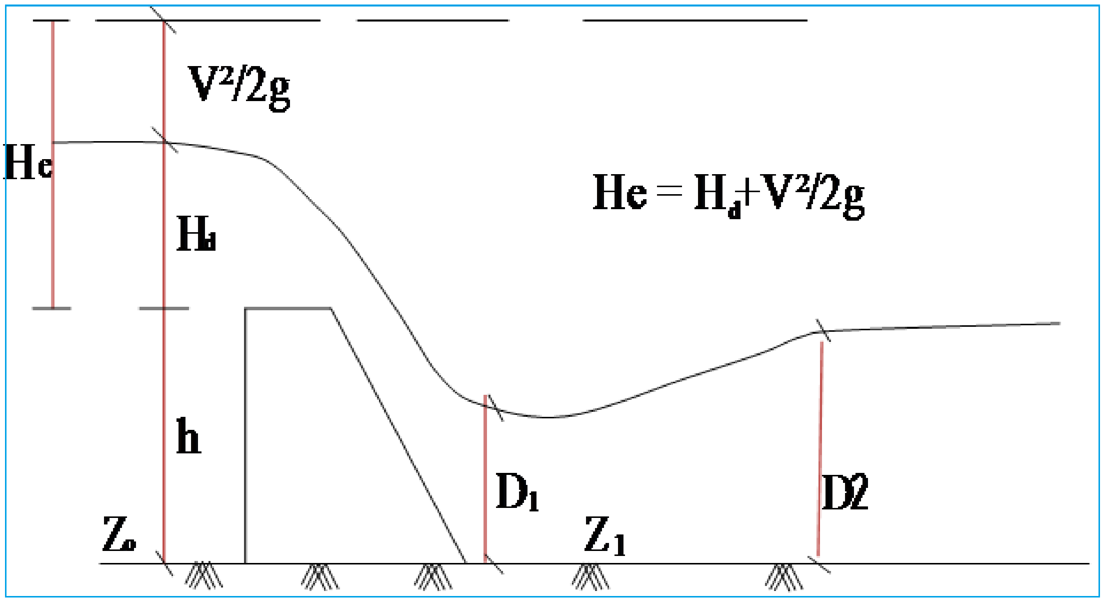

Figure 3 : Discharge curve over the weir crest

$$H_e = \left(\frac{Q}{CL}\right)^{2/3} = \left(\frac{65.00}{1.70*17}\right)^{2/3} = 1.72m$$

Approach velocity head is calculated as:

$$V_a = \frac{Q}{(h+H_d)L}$$

$$\mathrm{Va}^2/2\mathrm{g} = \mathrm{He} - \mathrm{H}_d \Rightarrow \mathrm{Va} = \sqrt{2\mathrm{g}(\mathrm{He}-\mathrm{H}_\mathrm{d})}$$

$$V_a = \sqrt{2\mathrm{g}(\mathrm{He}-\mathrm{H}_\mathrm{d})} = \frac{Q}{(h+\mathrm{H}_\mathrm{d})L}$$

Where,

Q = Maximum Design discharge =$65m^3/s$

h = Height of the weir body above the upstream river bed = 1.32m

H_d = Over flow depth without velocity head

L = Length of the weir = 17m

From $\sqrt{2g(\mathrm{He} - \mathrm{H_d})} = \frac{Q}{(h + \mathrm{H_d})L} \Rightarrow \sqrt{2 * 9.81(1.72 - \mathrm{H_d})} = \frac{65.00}{(1.32 + \mathrm{H_d}) * 17}$ by trial and error the value of H_d = 1.63m

In the same way:

Approach Velocity, $V_a = \sqrt{2g(He - \mathrm{H_d})} = \sqrt{2 * 9.81(1.72 - 1.63)} = 1.29 m/s$

Head due to velocity, $h_a = \frac{V_a^{\ 2}}{2 * g} = \frac{1.29^2}{2 \times 9.81} = 0.09m$

Upstream water level = weir crest elevation + calculated over flow depth (H_d) = 1311.68 +1.63 = 1313.31m

2.6.9 Bottom level of the weir body

Vertical cut-off or sheet piles are always provided at the upstream and downstream ends of the weir to safeguard against scouring and piping effects at the downstream end. Providing intermediate cut-off walls at the ends of upstream and/or the downstream slopes of the impervious floor are useful in holding the main structure, i.e. the weir. The depth of cut-off walls should be such that its bottom is lower than the level of possible flood scour at that section. The downstream cut-off, in addition, should also be sufficient to reduce the exit gradient within safe limits, which is decided by the sub-surface conditions.

The normal scour depth below Highest Flood Level (HFL), R is given by the equation.

By Regime scour depth method,

$$R = 1.36 \times \left(\frac{q^2}{f}\right)^{1/3} = 1.35 \times \left(\frac{3.82^2}{1}\right)^{1/3} = 3.30m$$

Where,

R = hydraulic mean depth

q = Discharge per meter length= Q/L = ($65m^3/s$)/17m = $3.82m^2/s$.

f = is Lacey's silt factor, usually taken as f=1

Reduced level of bottom of the weir body (R.L) = upstream high flood level -1.25R = 1313.31 – 1.25*3.30 = 1309.18. Take score depth of 1.50m below the riverbed level.

Since, the riverbed material at the proposed headwork site is formed of fresh basalt rock at 1.2m, cut-off walls at the upstream and downstream ends of the weir has been provided to safe guard against piping.

The downstream cut-off wall at the end of the weir apron is calculated as (R.L) = HFL – 1.5R = 1313.31-1.5*3.30 =1308.36m. To safeguard the weir against piping the d/s cut-off wall is fixed at 1308.36m, which 1.50m below the riverbed level, based on the geologist recommendation.

2.6.10 Section of the weir body

A gravity weir must be designed with sufficient factor of safety to resist the following three tendencies to destruction:

1. Overturning:

There is a tendency for a gravity weir to overturn about the downstream toe of the foundation or about the downstream edge of any horizontal section. The most critical condition for inducing overturning is, when, at the upstream face, the uplift pressure exceeds the vertical stress at the horizontal section.

2. Sliding:

The horizontal force tends to displace the weir in a horizontal direction. The frictional and shear resistance of the masonry and the foundation will resist this tendency. The shear friction factor, which is used for the stability criterion of all large weirs, should also be used for small weirs.

3. Overstressing:

The unit stress in the gravity weir and the foundation must be within allowable bearing values. The basic shape is supposed as the shape stable dynamically based on height of the fixed weir (h) and water depth on the crest (H_d). There are Bligh's formula and Etcheverry's formula to assume the shape.

The weir is to be constructed on pervious foundation, uplift force considered to act along the bottom of the weir body. Hence, specific weight of the weir becomes (ρ-1).

Top Width of weir

a. Bligh's Formula

$$B_1 = \frac{H_e}{\sqrt{\rho - 1}} = \frac{1.72}{\sqrt{2.4 - 1}} = 1.45m$$

b. Etcheverry's Formula

$$B_1 = 0.552\,(\sqrt{h} + \sqrt{(H_d + H_{va})}) = 0.552 * ((1.32)^{\wedge}0.5 + (1.63 + 0.09)^{\wedge}0.50) = 1.36m$$

Where,

B_1= Top width of the weir body

H_e= Overflow depth H_d + velocity head =1.72m

ρ = Specific weight of the weir body or concrete (ρ = 2 – 2.4), take ρ = 2.4

h = Height of the weir body above the upstream river bed = 1.32m

The bottom width of the weir

a. Bligh's Formula;

$$L = \frac{He + h}{\sqrt{\rho - 1}} = \frac{1.72 + 1.32}{\sqrt{2.4 - 1}} = 2.57m$$

Where

L: Bottom width of the weir (m)

Note: After checking the stability analysis of the weir body, the top and bottom widths of the weir body decided to be 1.50m and 3.0m respectively.

2.6.11 Bed level of the stilling basin (apron)

Building a weir elevates the total energy line upstream of the structure. The difference between the upstream and downstream energy grade line becomes very high. Therefore, the energy must be dissipated before it reaches the natural river course; otherwise, it causes damage to the banks and downstream of the apron. The flow over the weir is in a supercritical state. For this reason, the energy tends to dissipate through a hydraulic jump downstream of the weir. To control the location of the jump, the apron and stilling basin are designed to suit a range of the river discharge.

To estimate hydraulic jump on the downstream horizontal apron of the weir, the following equation is used.

$$D_2 = \frac{D_1}{2}\sqrt{1 + 8F_1^{\,2}} - 1$$

$$D_2 = \frac{D_1}{2}\left(\sqrt{\frac{2q^2}{gD_1} + \frac{1}{4}D_1^{\,2}} - \frac{1}{2}\right)$$

Where,

q = Discharge per meter width = $65m^3/17m = 3.82m^2/sec$,

Based on energy of flow, the hydraulic jump (pre and post jump) occurring at the stilling basin can be calculated as follows.

Total energy level at Upstream = Total energy level at downstream ⇒ Eo = E1

$$Z_o + h + H_d + \frac{V_a^2}{2g} = Z_1 + D_1 + \frac{V_1^2}{2g}$$

Where,

Z_o = Upstream bed level = 1310.35m

h = Weir height above the ground =1.32m

H_d = Over flow Depth over the weir crest =1.63m

V_a = Approach velocity = 1.29m/s; $\frac{V_a^2}{2g} = 0.09$

Z_1 = Downstream apron level =1309.85m

D_1 = Pre-jump depth

V_1 = Velocity at the jump = q/D_1

By substituting the value of V_1, the equation will be reduced to

1310.35+1.32+1.63+0.0.09 = 1309.85 +D_1 + $\frac{(3.82/D_1)^2}{2*9.81}$

By trial and error the value of D_1= 0.51057 ≈ **0.51m**

Pre-jump water level = 1309.85+0.51 = 1310.36**m**

$$V_1 = \frac{q}{D_1} = \frac{3.82}{0.51} = 7.49 m/s$$

Froude number, $F_1 = \frac{V_1}{\sqrt{gD_1}} = \frac{q}{\sqrt{gD_1^3}} = \frac{3.82}{\sqrt{9.81*0.51^3}} = 6.55$

$$D_2 = \frac{D_1}{2}\left(\sqrt{1+8F_1^2} - 1\right) = \frac{0.51}{2}\left(\sqrt{1+8*6.55^2} - 1\right) = 4.48m$$

$$V_2 = \frac{q}{D_2} = \frac{3.82 m^3/s}{4.48 m} = 0.85 m/s$$

Where,

D_2 = Post-jump depth

V_2 = Post-jump velocity

Post-jump water level = 1309.85+4.48 = 1314.34**m**

Head loss or dissipated energy because of the jump

$$H_L = \frac{(D_2 - D_1)}{4D_1D_2} = \frac{(4.48-0.51)}{4*0.51*4.48} = 0.43m$$

From the Stage discharge curve, the tail water depth D_3 is 1.96m. D_3 is less than the post jump level D_2. Therefore, the jump will recede downstream to a point where the flow condition allows the jump to occur.

2.6.12 Upstream Apron

The Functions of an upstream apron are to protect the upstream riverbed from being eroded by the approaching velocity and to increase the length of percolation path in order to avoid piping and to reduce uplift force acting over the entire base area.

The downward water pressure is always higher than the uplift pressure in the region of the upstream side of the weir. The thickness of the upstream apron can be based on the practice of the construction and the perfection of leakage proofing. 0.50m thick is usually sufficient.

2.6.13 Downstream Apron:

Downstream apron prevent scouring of downstream of the weir due to the overflowing water. Downstream apron has two main functions. The first is to lengthen the path of percolation and the second is to dissipate energy. Therefore, the properly designed stilling basin and energy dissipater must be provided at the downstream end of the weir.

Length of Stilling Basin (downstream apron)

The jump length is a function of Froude number of the incoming flow (F_1), and flow depth, d_1. The length of the downstream horizontal floor should be such that the entire jump is confined only to the floor. This will ensure that the stone protection provided on the downstream of the floor is not affected adversely by the jump. Hence, the length of the downstream horizontal floor is kept equal to the length of the jump, which is equal to 5 to 6 times the height of the jump i.e. 5 to 6 (d_2-d_1). Here d_1 and d_2 are pre-jump and post-jump depths of flow. Take 5.5, which is the average of 5 and 6.

Therefore, the length of the downstream apron can be calculated using hydraulic jump method.

$L = 5.5(\mathrm{d}_2 - \mathrm{d}_1)$ (Ref. G.L. Asawa "Irrig. Eng." page 405 and Bharat Singh "Fundamentals of irrigation engineering" ninth edition page 330)

Thus, $L = 5.5(\mathrm{d}_2 - \mathrm{d}_1) = 5.5(4.48 - 0.51) = 21.85 \approx 21m$

When F_1 lies between 4.5 to 9.0, as in the case of dam spillways, the jump performance is at its best. The jump is called steady jump. The length of the jump is almost constant and equal to 5.5*D_2 Hence for large spillways, where D_2 may be quite high, very long expensive stilling basins may be required. Some auxiliary device may be introduced for further stabilizing the flow and to reduce the length of the basin. But in our case even though the Froude number is between 4.5 and 9, it not requires some auxiliary devices due to basaltic river bed level at around 1m depth (Irr. Eng. & Hydraulic Structure-GARG page 1142).

Thickness of downstream apron

There are two approaches to determine the apron thickness. The first assumes that the apron consists of individual unit volumes, which are structurally not linked, and the weight of each individual unit balances the uplift pressure. This assumption leads to an increase in the structure's cost; the computations involved are very easy and result in a structure with a high safety factor.

The second approach to design the apron is by considering the whole structure as one unit and determining the bending moment and shear force at the critical section, which is at the toe of the weir.

To determine the thickness of the apron both dynamic and static case should be considered. The lower parts of the apron will generally require larger thickness when static case is selected, but the upper part of the apron (the toe section) will have larger thickness when dynamic case is considered. Therefore, the thickness at any point say A, B and C is calculated from

$$t = \left[H_{\max}\left(1 - \frac{L_A}{L_c}\right) + \left(TWL - WL_A\right)\right] * \frac{f}{(\gamma_m - 1)}$$

Where,

t = thickness of apron at any point (m)

f = factor of safety = 1.3

Hmax = U/S and D/S head differential (maximum) head, m

Lc= weighted creep length total (m), $L_c = \sum L_v + \frac{1}{3}\sum L_H \geq CH_{\max}$

$$L_c = \sum 1.50+1.00+1.00+0.50+\frac{1}{3}\sum 0.50+2.00+1.50+1.50+7.00+7.00+6.50+0.50 \quad =13.33m$$

LA = Weighted creep length at point A (m)

TWL =Tail water level = 1312.31m

WLA = water level at point A

γ_m = Specific gravity of concrete = 2.4

C = Percolation coefficient (Lane's creep ratio), the value of C for Rocks with Cobble stone and gravel =2.50 (OIDA Technical guideline for design of head work, page 58).

Case-1 Dynamic Case or high flood condition (H_{max} =1.00m)

$L_c \geq CH_{max}$ =13.33 ≥2.5*1.00 = 13.33 ≥ 2.5, safe against piping (seepage)

Case-2 Static Case or no over flow condition (H_{max} =1.32m)

$L_c \geq CH_{max}$ =13.33 ≥ 2.5*1.32 = 13.33 ≥3.31, safe against piping (seepage)

Point	Weighted creep Length			Hmax[1-La/Lc]	TWL-Wla	Safety Factor /(2.3-1)
	H	V	L			
Dynamic Case H_{max} = **1m**						
A	5.5	4.5	6.33	0.52	0.54	1.06
B	12.5	4.5	8.67	0.35	0	0.35
C	19.5	4.5	11	0.17	0	0.17
Static Case H_{max} =**1.32**						
A	5.50	4.50	6.33	0.69	0.50	1.19
B	12.50	4.50	8.67	0.46	0.50	0.96
C	19.50	4.50	11.00	0.23	0.50	0.73

Table 1: Thickness of downstream apron floor

At the toe of the weir, the term ($TWL-WL_A$) should be reduced by 50% and becomes

$$t_A = \left[H_{\max}\left(1 - \frac{L_A}{L_c}\right) + 0.50(TWL - WL_A) \right] * \frac{f}{(\gamma_m - 1)}$$

Check the thickness of each point

Point A 1.50 > 1.06m Ok (dynamic case)

Point B 1.00 > 0.96m Ok (Static case)

Point C 0.75 > 0.73m Ok (static case)

2.6.14 Khosla's Exit Gradient (GE)

If the upward thrust exceeds a certain value at the exit, piping will occur. It has been determined that for a standard form consisting of floor length **b** and vertical cutoff of **d**, the exit gradient at the downstream end is given by, (Khosla's Method).

$$G_E = \frac{H_{\max}}{d} * \frac{1}{\pi\sqrt{\lambda}} = \frac{1.00}{0.27} * \frac{1}{\pi\sqrt{48.65}} = 0.169$$

Where,

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+96.3^2}}{2} = 48.65\,, \quad \alpha = \frac{b}{d} = \frac{26.60}{0.27} = 96.30$$

H_{max} = head difference between u/s water level and tail water level (maximum) head = 1.00m

b = Total floor length = 26.60m (see Auto cad Drawing)

d = downstream Vertical cut-off wall = 0.27m (see Auto cad Drawing)

From the above result, it is safe against piping, since G_E = 0.169 is between safe exit gradient of 0.25 to 0.20 Shingle material consisted of boulders with some gravel and sand

2.7 Structural Analysis of the Weir Body

The weir have to be checked against externals forces to decide whether the dimensions adopted from design based on hydraulic criteria is sufficient from structural stability point of view or not. The stability of the weir was checked for two conditions i.e. for high flood level and pond Level conditions.

The weir wall will constructed of plain concrete. As described in the above sub section, Apron length and thickness, the impervious apron was checked for uplift pressure/piping. The external forces considered on stability of the weir body and found to be the provided weir section is safe against (sliding, overturning, over stress and against contact pressure in high flood level condition) all disturbing forces in both cases with high flood and no flow condition..

Acting forces on the Weir

All external forces acting on the weir are the result of flowing water in the river on which the structure is constructed. A typical force system of a weir consists of the following components.

1. **Static water pressure of the surface water:** Its value can be easily calculated if the effect of changing the static pressure upstream to the dynamic one downstream is neglected. Usually in structural analysis of a weir, the dynamic force is neglected.
2. **Uplift water pressure:** Uplift pressure caused by water percolating under or along the sides of hydraulic structures reduces the effective weight of a structure and is therefore particularly significant in the stability analysis. Stilling basin floor, for example, is subjected to the uplift pressure.

$$U_1 = \frac{1}{2} w_o \mu B_2 h_1 \quad and \quad U_2 = \frac{1}{2} w_o \mu B_2 h_2$$

$$U = \frac{1}{2} w_o \mu B_2 (h_1 + h_2)$$

Where

w_o: unit weight of water (9.8 KN/m3)

B_2: width of bottom of weir body (m)

h_1, h_2: water depths of upstream and downstream (m)

μ: uplift coefficient

3. **Earth Pressure due to sediment**

$$P_e = \frac{1}{2}(w_1 - w_o)C_o he$$

Where,

W_1: unit weight of deposited silt (18 KN/m^3)

w_0: unit weight of water (9.8 KN/m^3= 1tom/m^3)

C_0: coefficient of earth pressure ≈ 0.4-0.5 (Ref. OIDA Technical Guideline for Design of Headwork, Page 95)

h_e: height of deposited silt (in principle it is treated to be deposited on the crest)

4. **Friction forces at the base which develop to balance the horizontal forces**
5. **Weight of the weir and water wedges:** Weight of the weir is calculated simply by multiplying unit weight of the weir by its volume. Weight of the water wedges present the weight of water that is on the weir body and act either against or in favour of the weir stability, it depends on the slope of the weir and the water surface at downstream.
6. **Soil reaction at the weir base**

As this weir is to be constructed monolithically (as one integrated unit with the apron), the structure is more stable and it would not fail because of unbalanced moments. However, the stability analysis is done as follows.

See the schematic diagram showing the condition of stability (Figure-4)

Table 2: Stability Analysis

No.	Types of forces	Forces (KN)		Arm length	Moment about D (KN.m)	
		Horizontal	Vertical	(m)	Positive	Negative
1	Self weight, $W_1 = 23*1.32*1.5$		45.54	0.80	34.16	
2	Self weight, $W_2 = 0.5*1.5*1.32*23$		22.77	2	45.54	
3	Water pressure, $P_1 = 1.72*1.32*9.8$	22.25		0.66		14.68
4	Water pressure, $P_2 =0.50*1.32*1.32*9.8$	8.54		0.44		3.76
5	Water pressure, $P_3 = 0.5*1.63*1.50*9.8$		11.98	0.5	5.99	
5	Water pressure, P4 = 0.5*1.96*1.50*9.8		14.41	2.5	36.02	
6	Water pressure, P5 = 0.50*1.96*1.32*9.8	12.68		0.44		5.58
8	Silt Pressure, $P_e = 0.5*0.40*(1.32-1)*1.32*1.32$	3.21		0.44		1.41
9	Uplift pressure,$U_1 = 0.5*3*(1.63+1.32)*9.8*0.40$		17.35	1		17.35
10	Uplift pressure,$U_2 = 0.5*3*1.96*9.8*0.40$		11.52	2		23.05
Total		**46.68**	**123.57**		**121.7**	**65.83**

Take

Density of concrete, $\gamma c = 23KN/m^3$ (for apron and weir body)

Density of water $\gamma w = 9.8KN/m^3$

μ: The coefficient of uplift is μ = 0.40

Unit weight of deposited silt (18 KN/m3)

Co: Coefficient of silt pressure, = 0.40-0.50

Check for

1. **Sliding**

$$\frac{\eta\sum F_v}{\sum H_F} > 1.5 \Rightarrow \frac{0.75\sum 123.57}{46.68} = 1.99 > 1.5$$, safe from sliding

Where,

η is coefficient of friction between two surfaces (foundation and concrete), μ= friction coefficient = 0.70～0.75, (Ref. OIDA Technical Guideline for Design of Headwork, Page 97). Take μ= 0.75

∑Fv = Resultant of vertical forces = Sum of all vertical forces =123.57KN

∑FH = Resultant of Horizontal forces = Sum of all horizontal forces =46.68KN

2. **Over-turning**

If the resultant of all forces acting on the weir body at any of its section passes outside the toe, the weir shall rotate and over turn about the toe. The ratio of the righting moments about the toe (anti-clock wise) to the over turning moments about the toe (clock-wise) should be greater than 1.5.

$\frac{\sum M(+)}{\sum M(-)} > 1.5 \Rightarrow \frac{121.70}{65.83} = 1.85 > 1.5$, safe against over turning

3. **Tension**

To avoid tension at the base, the forces must pass through the middle third of the structure base. Therefore, the following condition should be satisfied.

Eccentricity, $e < \frac{B}{6}$

Where, e = eccentricity of the resultant force from the center of the base

Therefore, $e = \left| \frac{\sum M}{\sum F_v} - \frac{B}{2} \right| < \frac{B}{6}$

Where, $\sum M = \sum M(+) - \sum M(-) = 121.70 - 65.83 = 55.87$

Total weir width, B = 3.00m

$\sum M$ (+) = 121.70KN. m and $\sum M$ (-) = 65.83KN. m

$\sum F_v$=123.57KN

$e = \left| \frac{121.70}{123.57} - \frac{3.00}{2} \right| < \frac{3.00}{6} = 0.52 < 0.58 \Rightarrow$ Safe from tension

4. **Overstressing**

$$P = \frac{\sum F_v}{B}\left(1 + \frac{6*e}{B}\right) = \frac{123.57}{3.00}\left(1 + \frac{6*0.52}{3.00}\right) = 84.03KN/m^2$$

According to the engineering geology report, the riverbed material is fresh basaltic rock. The allowable safe bearing capacity of this material is presumed to be greater than1000KN/m^2. Therefore, 84.03KN/m^2 far less than the allowable bearing capacity of the bed materials (see the geology report).

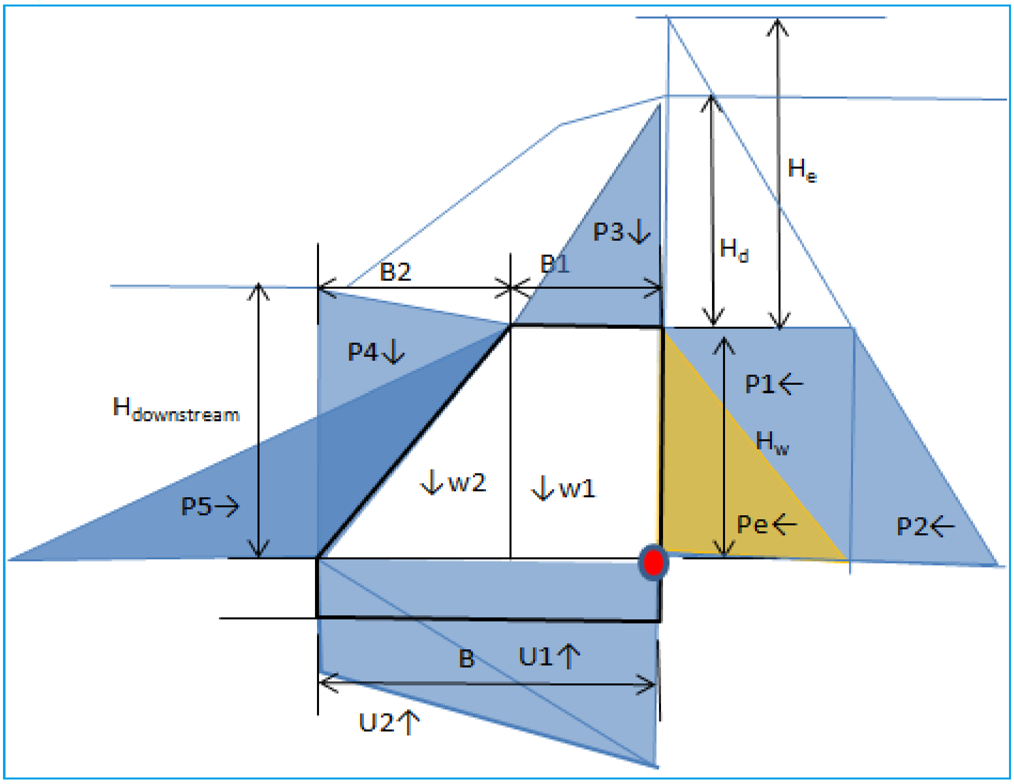

Figure 4 : Schematic diagram showing force distribution

2.8 Wing Walls

The heights of upstream and downstream wing walls were determined in the hydraulic design work. Based on availability of material in site gravity stone masonry wall are proposed to be constructed. The critical condition for u/s & d/s wing walls is no flow condition, because during high flood condition the structures are on safer side since soil pressure and water pressure balance each other. But in no flow condition wing walls are subjected to critical side soil pressure. This type of condition may occur during maintenance when under sluice gate is open & the coming flow of the river passes through under sluice. For the structures to be safe:

a. the masonry section of the retaining (wing) walls must have enough weight to resist the thrust due to earth pressure from its rear without overturning

b. The resultant force of entire weight of the wall and the external forces acting on it are required to cut the base within middle third

When these conditions are satisfied, the nature of the stress acting on the masonry wall at its base will be compression only. Thus stability is checked under these critical conditions and the result of stability analysis for both walls indicate that it was safe against sliding, overturning, over stress and against contact pressure for worst case mentioned.

The analysis and design of the wing walls will be done the same as retaining wall design.

Top levels of the upstream wing walls = Weir crest elevation + over flow depth on the weir crest + freeboard =1311.68+1.63+0.40=1313.71m (0.40m freeboard is added).

Top levels of downstream wing walls = downstream apron level + post-jump depth of flow (d_3) + freeboard =1310.35 + 1.96+0.40 =1312.71m.

To decide bottom level of the upstream wing walls, take the following design parameters.

High Flood level (HFL) =1312.31m

Calculated normal scour depth, $R = 1.35 \times \left(\frac{q^2}{f} \right)^{1/3} = 1.35 \times \left(\frac{3.82^2}{1} \right)^{1/3} = 3.30m$

Bottom level of the upstream and downstream wing walls, RL = HFL-1.5R = 1312.31-1.5*3.3 = 1307.36m. Since, the underlain material of the river is fresh basaltic rock, bottom level of both u/s and d/s wing walls fixed equal to the weir body level i.e. 1309.35m.

2.8.1 Active Earth Pressure against vertical walls

By Rankine's theory

Intensity of Active Earth Pressure is given as follows

$P_a = K_a \gamma h$

Total earth pressure on the retaining wall

$$p = \frac{1}{2} k_a \gamma h^2$$

The pressure p is acting at the height of h/3 above the apron =3.35/3 =1.12m

Where,

h= depth at which the active earth pressure is calculated =3.35m

γ = specific weight of backfill soil = 1.80, bulk density of soil =1.80mg/cm3 =1800kg/m3

ka = coefficient of active earth pressure, $k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.37$

ϕ = angle of internal friction of the soil (angle of repose), for saturated soil =270

The base width of the wing walls (b) is taken as 1/2.5 to 1/1.5 times the height of retaining wall. After check stability Analysis the bed width taken as b = 2.0m

2.8.2 Stability analysis of wing wall

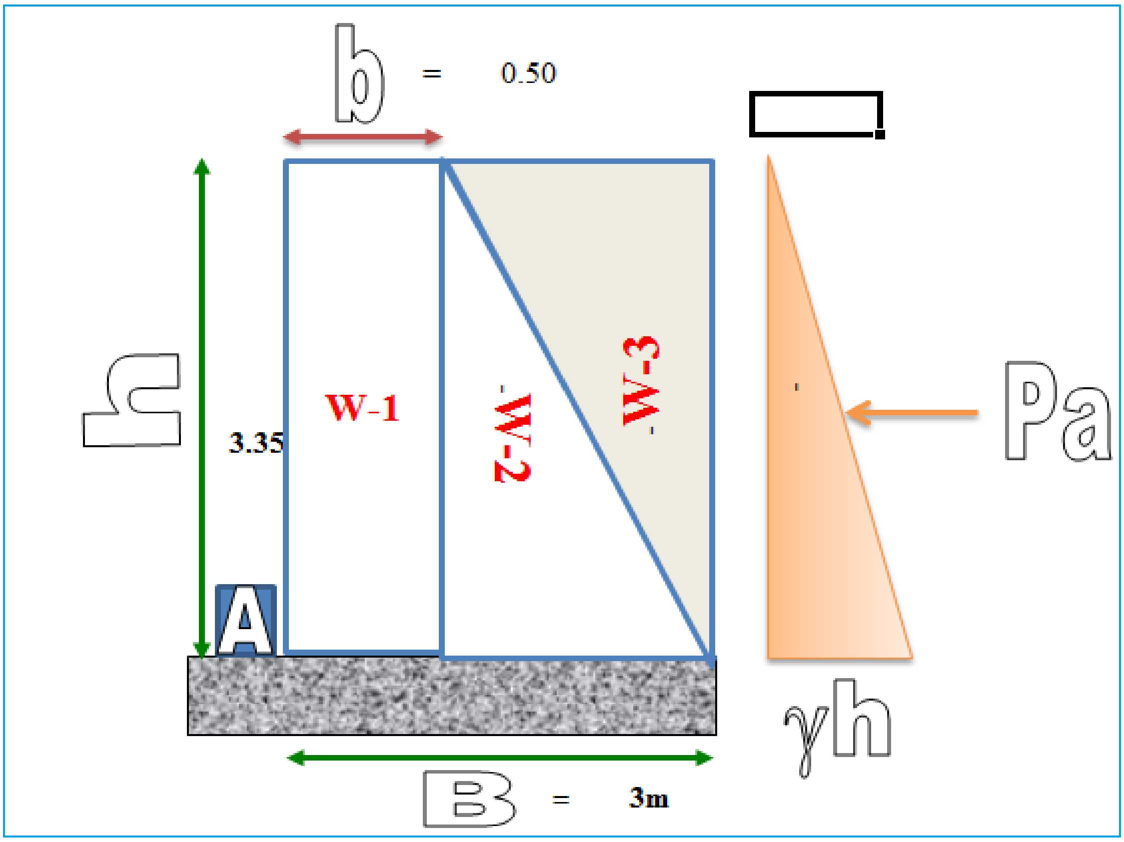

Figure 5 : Schematic diagram of wing wall

Table 3 : Force analysis of wing wall

Sn	Type Force	Forces		Moment arm	Moments	
		V	H	L.A	Mr	Mo
1	Self Weight, W_1	35.22		0.25	8.8	
2	Self Weight, W_2	52.83		1	52.83	
3	Soil weight, W_3	52.83		1.5	79.24	
4	Active earth pressure, Pa =½(kag_sh^2)		38.02	1.12		42.51
Sum		**140.9**	**38**		**140.9**	**42.5**
Mnet					**98.36**	

Note: Density of masonry, γ_m = 2.2ton/m and density of soil, γs = 1.80 ton/m

Stability condition

1. **Check against over turning**

$$\frac{\text{Moment due to load W}}{\text{Moment due to force P}} > 1.5 \Rightarrow \frac{\sum M(+)}{\sum M(-)} > 1.5 \Rightarrow \frac{140.88}{42.51} = 3.31 > 1.5$$, safe against overturning

Where, $\sum M(+) = 140.88 \, and \quad \sum M(-) = 42.51$

2. **Check against sliding**

$\frac{\sum W_v \mu}{P} > 1.5 \Rightarrow \frac{140.88*0.70}{38.02} = 2.59 > 1.5$, safe against sliding

$\sum W_v$ =Sum of Vertical loads = 140.88 ton/m

P = horizontal Force = 38.02 ton/m

μ = efficient of friction = 0.70 to 0.75, take the minimum value of μ = 0.70

3. **Check against base pressure (over stressing)**

After finding out the eccentric e, the maximum and minimum normal stress intensities at the two ends (toe & heel) can be calculated from the following formula (Irrigation Eng. & Hydraulic Structures, page 101-104).

$$P_b = \frac{W_v}{b}(1 \pm \frac{6e}{b})$$

Eccentricity, $e = \frac{b}{2} - \bar{x} = \frac{2.0}{2} - 0.698 = 0.302$

e = eccentricity of the resultant force

$$e < \frac{b}{6} = 0.63 < \frac{2.00}{6} = 0.33 < 0.42$$

Where,

$\bar{x}$ = lever arm of the resultant force

$$\bar{x} = \frac{M}{W_v} = \frac{(140.88 - 42.51)}{140.88} = 0.698$$

Normal stress at the toe of the retaining wall is given by

$$P_b = \frac{W_v}{b}(1 + \frac{6e}{b}) = \frac{140.88}{2.00}(1 + \frac{6*0.33}{2.00}) = 140.18 ton/m^2 .$$

The underlain material of both riverbanks is fresh basaltic rock material, therefore no bearing problem.

2.9 Design of Protection Works (Riprap Protection)

Riprap is provided at the downstream and upstream end of the weir apron where there is fear of scouring would be caused on the riverbed taking into account the condition of the riverbed material and flow condition of the river. Therefore, riprap is provided continuously to prevent the scouring of the riverbed, because it is apparent the scouring is caused due to removal of deposited silt or riverbed materials.

Just after the end of the concrete floor, riprap of 1.5 to 2R long is generally provided. Where, R is the depth of scour below the riverbed=2.00m, (Santosh Kumar Garg "Irrigation eng & hydraulic structures" page 510). Take the minimum, which is 1.5R

Therefore, Length of riprap = 1.75R =1.75*2.00 =3.50m.

2.10 Design of Intake Facility

Canal intake is installed near the under sluice gate in order to avoid entrance of silt into the main canal. Size of the intake and velocities may be determined as follows. Taking in to account the headwork Topographic nature, the intake selected for this project is concrete pipes controlled by gate. The main reason for this is that the intake canal will pass through the wing walls on both right and left side riverbanks. Due to this concrete pipe is selected.

Net irrigable area to be irrigated is 214.36ha on the right side and 57.74ha on the left side. Based on the given water duty of 1.52 lit/sec/ha (for 24 hours), it require 325.83 l/sec discharge on the right side and 87.76l/s on the left side. Therefore, the canal carrying capacity will be 325.83l/s on the right side and 87.76l/s on the left side.

Concrete pipe intake (the first 15m length)

To design main canal intake concrete pipe, $Q = CA\sqrt{2gh}$

Where,

A =flow area for pipes (m2)

h = head difference (head loss in intake pipe, assume h=0.15m)

g = acceleration due to gravity =9.81m/s2

C = coefficient of discharge = 0.6 to 0.7, But take C=0.73 for submerged case

Q = maximum discharge in the canal = 0.33m^3/sec

$0.33 = 0.73A\sqrt{2*9.81*0.15}, A = 0.26m^2$

From $A = \frac{\pi d^2}{4}, d = 0.58m$,

As it can be seen from the above result, intake of the canal requires about 0.58m diameter of concrete pipes, which is not practical. Therefore, the size of the internal pipe diameter is increased to 0.60m for ease of access during cleaning.

In the same step above, the size of internal diameter of the left side intake is 0.26m but for practical case, it will be 0.50m.

2.11 Intake Gate

The type of gate is decided after considering its purpose, installation, location, ease of operation, safety, and economy of water intake. The gate is selected considering the effective usage of the water resource and operation method to reduce over diversion. The gate selected for this simple structure is gate with spindle made up of 5mm thick steel sheet. The gate will have dimension of 0.65m height and 0.65m width to regulate the flow.

2.12 Under Sluice Gates

Under Sluice Gate is one of the most useful facilities to control both water utilization and flood. The gate structure must be watertight to ensure stable water intake and firm enough against several external forces such as flowing water, steady and smooth operational workability is also required for its function.

Above all, under sluice helps to allow the removal of silt deposited near the canal intake. This is designed to ensure sufficient scouring capacity in order to dispose of the peak flood.

The following conditions shall be fulfilled for the purpose of water use and flood control. From the water use point of view, the conditions required are:

1. To keep water level constant and to control intake level and discharge; and
2. Water tightness.

On the other hand, conditions required from the flood control point of view are:

1. Capability of being operated quickly and smoothly so as to release water safely;
2. To remain workable without accumulation of materials carried by water; and
3. Structural stability and durability are also required.

The size/dimension of the under sluice gates will have 1m width and 1.32m height. To avoid accumulation of silt around the intake gates, the level of the under sluice gate must be fixed 0.62m below the level of the intake gate. For small-scale irrigation project, the under sluice gate should not be lowered up to riverbed (apron) level. This is because small discharge of the river, especially during dry season, it will be lost by seepage through the gate side. To avoid this, from practical point of view, the under sluice bed level fixed just at least 0.25m below the intake gates.

2.13 River Bank Protection

The purpose of riverbank protection /training is:

- To provide a safe passage to flood discharge without overflowing of the banks for protection of cultivated or inhabited area;

- To prevent outflanking of a weir constructed across the river and to bring the river on to the work in a straight non-tortuous (meandering) approach;
- To deflect the river away from a bank which it might be attacking;
- To provide minimum depth of flow.

As it has been investigated during feasibility study and from topographic contour map, the river course near the head work needs no bank protection or river training. The floodwater flows being confined between the existing natural riverbanks. Therefore, the natural riverbanks will not need addition embankments.

3 IRRIGATION AND DRAINAGE SYSTEM DESIGN

3.1 General

A well-designed irrigation system will make irrigation application easy and more efficient. Irrigation farm structures, layout and drainage system are integral parts of the whole irrigation system. The methods/ criteria used to design the entire irrigation and drainage system are discussed below.

The objective of this project development is to make the farmers of the command area beneficiaries by irrigating 273ha of net irrigable area. This is intended to be developed by diverting water from Irba-Girstu River. For this purpose, a network of canal system, related infrastructures layout system and irrigation farm structures, comprising various components starting from main conveyance to field canals has been designed.

To achieve this aim, the following issues are very relevant.

- The layout of canal system
- Crop water requirement
- Efficiencies of different components of canal system

It is, therefore, very essential that the different components of the canal system should be able to carry adequate discharge which may be able to meet the needs of proposed crops under the command of individual components. And since design of small scale irrigation scheme intended to be handed to farmers after completion, calls for careful consideration of the operation needs and management skills of the beneficiaries. The aim has been to provide simple, strong and sustainable irrigation and drainage systems that are easy to understand and operate. Accordingly, the project has been provided with simple water control and regulating structures that fulfil these requirements.

3.2 Irrigation system layout

3.2.1 General

The Irrigation System layout for the Irba-Girstu Command is being planned and designed keeping in view of;

- It fits best into the existing topography
- It serves the requirement of proposed crop
- Irrigation water is able to reach every part of command area by gravity flow.
- The cost of construction and operation is minimum.

- It operates efficiently free from trouble.

Moreover, the irrigation system proposed comprises the canal network having two main canals that distribute water between secondary canals on both sides of the command area. The distribution system consists of a network of secondary, tertiary and field canals.

Main Canal

The layout of main canal is the most important and vital component of the entire planning work, that call for most careful consideration of all the factors governing the alignment: topography, natural drainage pattern etc. The main canal is aligned nearly along the contour lines to minimize loose of head, but due to topographic case most part of right main canal crosses contours. The total length of the canal is 3.92km for the right main canal and 0.91km for left main canal. Fortunately, on the alignment of both main canals there are no crossing structures except one main road crossing culvert.

Secondary canals

There are few number of secondary canals on both sides, 3 on left and 1 on the right side. All secondary canals aligned across contour. The secondary canals are either distributed water among tertiary canals or directly feed the field canals.

Tertiary canals

Some Block of command area are planned to be irrigated using the tertiary canals that off take directly from the secondary canals and supplies irrigation water to field canals. The average length of the tertiary canals is 500m.

Field Canals and Furrows

The command area of each tertiary canal is further sub-divided into several segments by field canals, which supply water to the furrows. These canals are aligned across the contour. By considering the proposed crops, furrow method of irrigation has been adopted. Accordingly, irrigation water will be applied to the farm through furrows. The average length of furrows is 60 meters. Irrigation water will be supplied to several furrows at a time, depending on the size of field canal that apply irrigation water.

Command Area

The topography of the area is characterized by steep slope. The available Topographical maps of 1:50,000 scale for entire catchment and command area of the project are not adequate for planning and design of irrigation system. The topographical surveys have been conducted with the help of Total Station Method and results have been fed into computer from which topographical maps have been generated with 0.5m contour interval for headwork and 1m interval for the command, 1:1,000 horizontal and 1:100 vertical scales.

The final size of the command area of the project is determined based on the result of the topographic survey, hydrological study, and soil survey and land evaluation study results. Accordingly, the survey has been conducted for entire command area measuring 400ha. After deducting non-suitable area, gross irrigable area is identified for final planning and design of the irrigation system out of which 273ha net irrigable area having slope less than 15% is obtained. The reduced area is constrain only due to slope (>15%), currently all of the surveyed area is rain fed and traditional irrigated.

3.3 Design Criteria

3.3.1 Field System Layout

Layout of the canal networks has been prepared on the Topographic map by taking the existing physical features of the area into consideration as explained above. As indicated on the topographic map and above 3.2, canals of the project are divided into main, secondary, tertiary and field canals. Individual canal irrigates different sizes of irrigation land. As result of this, water carrying capacity of the canals depends on the size of the area.

- The whole irrigation canals are designed to irrigate by gravity system. Topography of the area is suitable for this purpose. The irrigation land has sufficient slope, on both along and across the contours;
- Main canals is design to serve for 24 irrigation hours and it will take off directly from the headwork intake;
- Secondary and some field canals take directly from the main canal through turnouts.
- Tertiary canals take water from main and secondary canals and most field canals are obtain water from tertiary canals through turnouts. Turnouts are provided on the main, secondary and tertiary canals to divert water to field canals;
- To design all open canals Manning formula is used to calculate the canal dimensions (parameters);

Shape of open canals (earthen canals) preferred in this project is trapezoidal with side slope of 1:1. To prevent excess seepage in the main canal, all length of the main canals to be lined with masonry.

Secondary and field canals are aligned as much as possible on ridges, so that water can reach to the intended irrigation land without any head problem.

3.3.2 Water distribution

The size of field unit and the furrow lengths are determined based on suitability of topography of the project. Water distribution will be rotational in the case of Secondary and Tertiary canals. Even though the irrigation hour proposed by the agronomist is 12 hours, the flow in the main canals is continuous for 24-hours. As required, depending upon on the size of each irrigation area, entire flow of the canal may be allowed to the farmer(s) for his /their allocated time. The time allocation will be in proportion to the land size of each farmer. The farmers themselves will manage internal distribution of water. To do this all beneficiaries will establish Water Users Associations voluntarily.

3.3.3 Hydraulic Design consideration

The conveyance system for this irrigation project comprises of main, secondary, tertiary and field canals. Some of the factors affecting the canal discharge are Canal gradient or longitudinal slope of the canal, Canal roughness, Canal shape and Side slope.

The hydraulic design considers calculation of velocity of flow in the canals, determining canal cross-sections, determining of longitudinal slope, freeboard, roughness coefficient and side slope of canals. All these parameters are discussed as below.

The canal design should fulfil the following requirements:

- It should deliver reliable water from head to tail for each field canals;
- The canals should be designed with economical section, velocities within prescribed limits. The section should be non-silting as well as non-scouring;
- The canals must be with minimum seepage losses;
- The canals should be operated easily and require minimum maintenance cost;
- The embankment section shall be structurally stable;
- Through these canals, available water is used to get maximum irrigation efficiency and benefits.

a. Cross-section of Canal

Canals with the same cross-sectional area, longitudinal slope, and roughness, but with different shapes, will carry different discharges because of different wetted perimeters and hydraulic radii. The most efficient geometry is when the wetted perimeter is minimal for a given discharge. The semi-circle is the canal section that has the lowest wetted perimeter for a given cross sectional area, but semi-circular canals are difficult to construct. The closest canal section to a semi-circle is the trapezoid section. This is a quite common cross-section as it is relatively easy to construct.

Therefore, a typical and most desired section of the earthen canals preferred is trapezoidal section. This section should be partly in cutting and partly in filling to balance the quantity of earthwork in excavation with that of in filling. When natural surface level is above the top of the bank, the entire section will have to be in cutting. Similarly, when natural surface level is lower than the bed level of the canal, the entire section will have to be built in fill.

Trapezoidal section of earth canal has been provided where the land formation is soil, no seepage problem; there is no problem of excavation, and no stability problem. In the same way, trapezoidal or rectangular section is also selected for lined canals. However, rectangular section can be provided in the area where the land formation is unstable, unconsolidated, steep slope area to control erosion.

A change in the direction of the channel should be brought about by smooth curves and not abruptly. Similarly, when the cross-section is changed, the change should also be gradual.

b. Permissible longitudinal slope of Canals

Steeper slopes could result in such high velocities that the flow would be super-critical. It would then be difficult, for example, to siphon water out of the canal, since an obstruction in a canal where super-critical flow occurs tends to cause a lot of turbulence, which could result in the overtopping of the canal.

The deciding factor in determining the canal bed slope is usually the natural slope of the land. The steeper the channel, the more will be the velocity and more the discharge for the same cross-section. However, excessive gradients produce very high velocities, which cause erosion. Normally, a channel should slope about 0.001. Silting may take place in the channel if the slope is less than 0.0005. The recommended slope of the canal should be non-erosive and non-silting.

c. Velocity of flow in the canals

The steeper the slope, the faster the water will flow and the greater the discharge will be. Velocity increases with an increase in gradient or longitudinal slope. It therefore follows that a canal with a steeper gradient but with the same cross-section can discharge more water than a canal with a smaller gradient.

To determine bed slopes of channels, the velocities are checked and the maximum grade designed so that the velocities do not exceed the limits.

When the velocity of water flowing in the canal is such that none silting or scouring is taking place then the velocity is said to be critical. Critical velocity for canals made of earth is 0.30 to 0.60m/sec. (irrigation eng. & hydraulic structures, page 136).

Higher velocities can be safely used in lined canals. To avoid damage to the lining, maximum velocity in the masonry-lined canal should be in the range of 0.3 < V< 2.5m/s for masonry lined section and 0.3 < V < 1m/s for earthen section but can go up to 3m/s for concrete lined section. (MoANR, 2017).

d. Roughness Coefficient

The canal roughness, as depicted by the Manning roughness coefficient, influences the amount of water that passes through a canal. Unlined canals with silt deposits and weed growth and lined canals with a rough finish tend to slow down the water velocity, thus reducing the discharge compared to that of a clean canal with a smooth finish. Canals that slow down the movement of water have a high roughness coefficient. It should be understood that the lower the roughness coefficient n, the higher the ability of the canal to transport water, hence the smaller the required cross-sectional area for a given discharge.

This coefficient is mainly a function of grain size of bed and bank materials, bed shear stress, and depth of water in the channels. In addition to grain size, it depends on the size of ripples formed on the bed of the channels.

The value of n for straight earth canals is 0.025 (Irr.eng. & hydraulic structures, page 138). For different types of linings, the value of n varies, in general practice, for lined canal made up of dressed stone and plaster with mortar, the value of n can be taken as 0.014.

Type of Surface	Canal component	Value of
Masonry lining	Lined Part	0.014
Earthen Canal	Earthen Part	0.025

Table 4 : Adopted roughness coefficient

e. Free board

Freeboard is the vertical distance between the highest water level anticipated in the design and the top of the canal banks. It provides the margin of safety against overtopping of the banks due to sudden rise in the water surface of channel on account of improper operation of gates at the head regulator, accidents in operation, landslides and inflow during heavy rainfall. The excessive growth of vegetation or accumulation of sediment deposits may also result in the gradual rise of water surface levels above the design levels.

Design of channels should provide adequate freeboards to prevent overtopping of bank during sudden rises in water surface. Adequate freeboard would depend on dimensions of the flow section, flow condition, bank material, method of construction of banks and resulting damage due to failure of banks.

According to OIDA "Design of Irrigation Canal & Related Structures", page 164, a freeboard for lined canals (main canal, IRMC-1 and ILMC-1) is as follows.

$F_b = 0.05d + \beta \times hv + hw$

$F_b = 0.05*0.35+ 0.75*0.0113 +0.15 =0.18m$

Where,

F_b: Freeboard (m)

d: Water depth corresponding to design discharge (m) =0.35m

h_v: Velocity head (m) = v2/2g = (0.47)2/ (2*9.81) = 0.0113

β: Conversion factor from velocity head to static head, ranging 0.5-1.0, take 0.75

h_w: Freeboard for water surface vibration (m), taken 0.15 (page 165)

Therefore, to be safer, the freeboard is taken to be 0.25m. However, in the case of Secondary and tertiary canals 0.2 and 0.15 freeboard is taken respectively

f. Side Slope of canals

For earth canals, if the side slopes are very steep (low horizontal/vertical ratio) there is high risk of collapsing of banks, especially after heavy rainfall. Earth canals should be built with stable side slopes and with banks strong enough to carry the required flow of water safely. They should have ample capacity to carry the designed discharge at non-erosive velocities. Side slopes should be flat enough so that the banks will neither cave in nor slide when they are saturated with water. Permanent irrigation canals should not have side slopes steeper than 1 ½ horizontal to 1 vertical. The side slope of the canal depends on the properties of material through which the canal is to pass. Side slope for canals passing through sound rock is nearly vertical. The recommended values of the side slopes for canals excavated through different types of materials are given as follows.

Table 5 : Recommended canal slopes

Canal materials	Side slope (H:V)
Compacted clay soil	01:01
Clay soil	1:1/2:1
Loamy Soil	1:1/2:1
Sandy loam & black cotton soil	1:1 to 1.5:1 (in cutting)
Sandy Soil	03:01
Sound rock	1:/8:1 (in cutting)
Poor rock	½:1 (in cutting)

For concrete or masonry-lined canals, there are no strict rules for the side slopes of the canals. A major consideration is ease of construction and the fact that the concrete should stay in place during construction, thus the side slope should not be too steep. Side slopes of around 60º should be easy to construct. However, in our design, side slope for masonry-lined canal is taken to be 1:0. Because, canal size of the project is very small and for convenience of construction rectangular canal cross-section is selected.

3.3.4 Drainage system

Drainage systems are also provided to protect canals and irrigation area from damage, which would result from uncontrolled excess flow of irrigation water and surface runoff caused due to rainfall. Rainwater and excess irrigation water should be removed safely from the irrigation land by different drainage systems. Finally, the collected drainage water must enter in to the natural drainage system.

Cross-section of the drainage canals are estimated based on the maximum expected runoff from respective catchment's area. Since the amount of the flood increases towards the end, the size of the drain canals should also be increased towards the outlet.

The cross-sectional area of the drain is calculated using the following formula:

$Q = \mathrm{A} * V$ ....(1)

Where,

A = Cross-sectional area of the drain

Q = Maximum runoff rate and

V = velocity of flow

Shape of the cross-section of the drainage canal selected for this project is a trapezoidal section. The drainage canals consisted of field drainage, tertiary and collector drainage canals.

3.4 Design of Canals

The total length of both right and left main canals is 4.83kmm. Canals lining are adopted as per described in geology report for the main cannel routes. All lined main canals reach are designed as rectangular masonry canals, but all secondary canals are totally governed by the velocity. In addition, all earthen canals are designed as trapezoidal earthen canal. All tertiary canals run nearly parallel to the contour and proposed to be earthen canals.

3.4.1 Design of Lined Canals

All lined canals are designed as rectangular canals; the constructions of the canals are totally depending on the velocity of the flow. All lined contour/main canals are designed as rectangular masonry canals. For masonry & concrete surface Roughness co-efficient of n = 0.014 is adopted, but the longitudinal slope vary from canal to canal based on the slope of topographic were the alignments of canals pass. Accordingly the sample calculation using Manning formula, continuity equation and the stated design parameter was computed and also for the hydraulic parameter of the other similar canals are provided on the main report

3.4.1.1 Design of Lined Main Canal

Main canal is designed to supply irrigation water directly to field canals and secondary canals depending upon suitability and size of irrigation land/blocks.

Greater attention is given to make all the canals section design economical, with less excavation, less water loss along the line, high discharge with fewer cross-sections, easy for operation & maintenance.

The main canal starts irrigating at about **563** and **278**m at right and left main canals respectively away from the diversion headwork intake without any head problem. Beginning from the weir intake, 15m concrete pipes having diameter of 60cm 50cm at right and left side respectively is proposed. The main reason for this is that the intake canal is to pass through the wing wall and riverbank, which needs deep cut. For this reason, concrete pipe is recommended. All the canal alignment is made to maximize the command area as much as possible.

Right Main canal-1 has total length of **3.93**km. It has been designed to irrigate net irrigation area of **214.36** hectares (by gravity) with designed water duty of **1.52** lit/sec/ha (for 24 hours irrigation). Left Main canal-1 has total length of 0.91km and designed to irrigate net irrigable area of **57.74**ha. Discharge carrying capacities of the canals are **325.83** and **87.76**lit/sec respectively. Both IRMC-1 and ILMC-1 will serve for 24-hour of irrigation. Slope of both canals are varies in the range of **0.0008** to **0.006**.

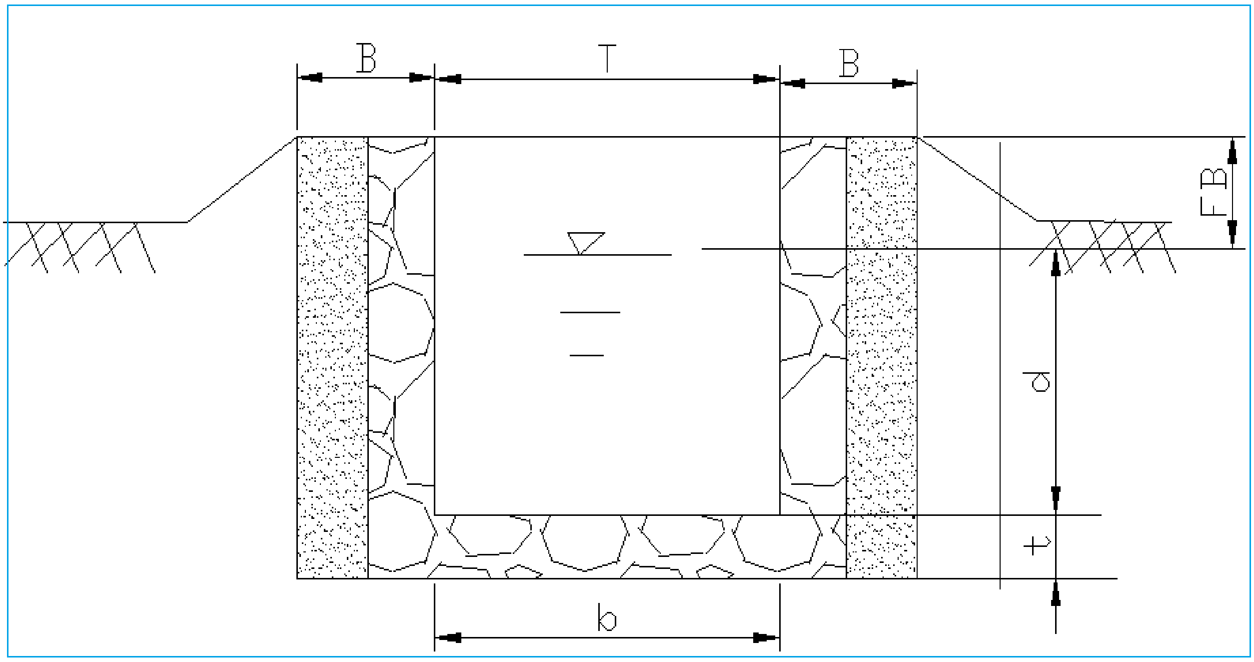

Figure 6 : Cross-Section of a canal

Where,

T = Top width of the canal

B = Bank top width

b = Bottom width

d = Water depth

F_B = Freeboard

t = thickness

Wetted area, A= b*d

Wetted perimeter, P=b+2d

Hydraulic radius, R= A/P

Velocity of flow and water depth in the canal is calculated by the equation:

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

V = Mean velocity of flow in the channel, m/s

S = Hydraulic slope

n = Roughness coefficient of the channel

Right Main canal-1 from chain age 0+020 to 0+565m has the following canal parameters:

Q =0.33m^3/s, S = 0.0025

n = 0.014 (lined canal), b= 0.80m.

By employing the above formula, water depth d in the canal d = 0.40m and V= 1.22m/s

By the same equation, the Left Main canal-1(Lined Canal) and around 26 trapezoidal tertiary canals (earthen canals) has been designed

3.4.2 Design of Earth Canals

All earthen canals are designed as trapezoidal section having side slope of m = 1:1, and free board of averagely 0.18m. For earthen/soil surface Roughness co-efficient of n = 0.025 is adopted, but the longitudinal slope vary from canal to canal based on the slope of topographic where the alignments of canals pass. Accordingly, the sample calculation using Manning formula and continuity equation and the stated design parameter was computed and the hydraulic parameter of the other earthen secondary and tertiary canals is provided on the engineering report.

3.4.2.1 Design of Earthen Secondary Canal

There are three right secondary canals with total length of 1.56km and one left secondary canal with total length of 0.93km. It has been designed to irrigate net irrigation area of 172 and 55.59 hectares respectively with designed water duty of 3.04 lit/sec/ha (for 12 hours irrigation). Discharge carrying capacity of RSC-1, RSC-2 and RSC-3 is 0.08, 0.27 and 0.18m^3/s respectively. In addition, discharge carrying capacity of LSC-1 is 0.17m^3/s. It will serve for 24-hours.

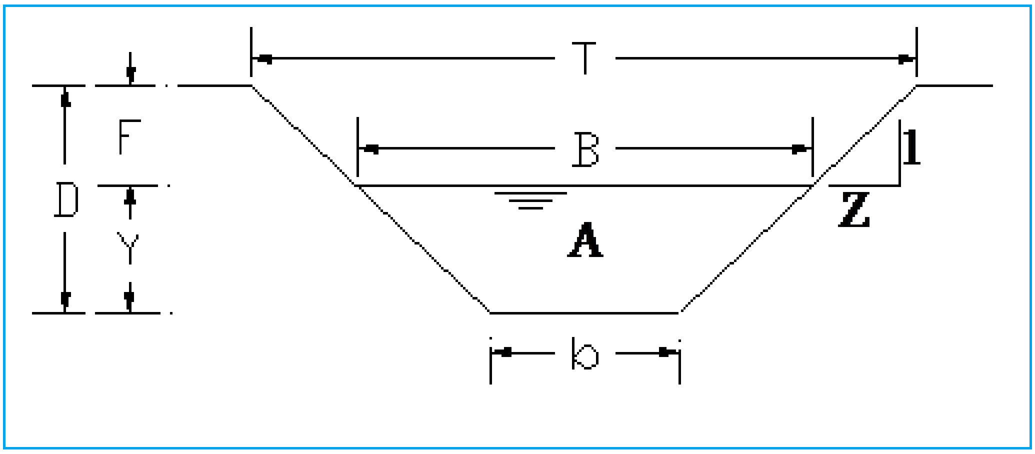

Figure 7: Cross-Section of a canal

Where,

T = Top width of the canal

B = Width of water surface when the water depth is at depth y

b = Bottom width

D = Depth of canal after free board is added

F = Freeboard

Z = Side slope of the canal

Y = Water depth in the canal

Wetted area $A = (\mathrm{b} + ZY)\mathrm{Y}$

Wetted perimeter, $P = \mathrm{b} + 2Y\sqrt{1 + \mathrm{Z}^2}$

Hydraulic radius, $R = \dfrac{A}{P} = \dfrac{(\mathrm{b} + ZY)Y}{b + 2Y\sqrt{1 + Z^2}}$

Velocity of flow and water depth in the canal is calculated by the equation:

$$V = \frac{Q}{A} = \frac{1}{\mathrm{n}} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

V = Mean velocity of flow in the channel, m/s

S = Hydraulic slope

n = Roughness coefficient of the channel

Sample example of RSC-1 has the following canal parameters:

Q =0.08m^3/s, S = 0.004

n = 0.025, b= 0.40m.

By employing the above formula, water depth d in the canal = 0.20m

By the same equation, the remaining right and left secondary canals have been calculated

3.4.2.2 Design of Earthen Tertiary canal

There are **30** and **14** tertiary canals in the right and left side respectively that takes water from the right and left main and secondary canals. The length of the canals is about **15282.35** and **5490.80**m respectively. The design principle of these canals is the same as earthen secondary canals.

3.4.2.3 Design of Field Canal

The whole fields channels are left for the farmers to be prepared every season during land preparation, as a result their cost are not included in project cost. However, the field canal capacity is determined using the area of the farm unit and assumed stream discharge of furrow. In addition, the typical off take location and size at the inlet of each field canals is designed and included in this report.

3.4.3 Proposed Water Distribution

The main canals will supply the required water rationally to other canal namely secondary, tertiary and field canals. Therefore, field canals function rotationally. The inter farm irrigation water distribution can be decided by the water users association to be established during project implementation.

3.4.4 Method of Water Application

As shown on the topographic map, slope of the area is almost steep in nature. Accordingly, a furrow irrigation method is proposed. The furrow size of 25cm wide and 10-20cm deep can be used depending on the crop growing stage. Furrow spacing varies with the type of crops grown in the project. However, spacing of 0.5-1m is common for most row crops. Length of a furrow is approximately 100m.

3.4.5 Drainage System

Surface drainage is the diversion or orderly removal of excess water from the surface of land by means of improved natural or constructed channels. Drainage system is integral part of the irrigation system.

Drainage system protects the irrigation system such as main, secondary, tertiary and field canals, and irrigation land from damage, which would result from uncontrolled excess flow of irrigation water and surface runoff caused due to rainfall. Rain and excess irrigation water must be controlled to avoid erosion and damage of the irrigation land and irrigation system. Excess water should be removed safely from the irrigation land by different drainage systems. Finally, the collected drainage water should enter the natural drainage system.

During feasibility study, major drainage lines were identified in the command area. Drainage network of field, tertiary and collector drainage system is proposed to remove excess irrigation water and rainwater from the fields. According to the hydrology report, the drainage duty is 1.74l/s/ha (10 year of return period). This value is used to design the drainage systems.

3.4.5.1 Field Drainage canal

Surface drainage is applied primarily on flat lands where slow infiltration, low permeability, or prevent the ready absorption of high intensity rainfall. The drainage system is therefore intended to eliminate ponding and prevent prolonged saturation by accelerating flow to an outlet without causing siltation, soil erosion, and damage to the irrigation systems. Surface water can be removed safely from the irrigation land by constructing of open drains to the main outlet in order to meet the requirements of surface drainage.

To design respective drainage canal, peak runoff should be estimated based on drainage area and rainfall intensity. The length of farm boundaries determines the length of the field drains. It is often decided to place the field drains at right angles to the tertiary drain and the tertiary drain join collectors. The length of a collector is restricted either by a field boundary or by the available slope

The field drainage system is generally designed on a model basis for a sample area. This can be a single farm of less than one hectare or an area of more than one hectare. The design is only a guideline, which can be adjusted for each single farm to incorporate specific circumstances.

Peak Runoff, (Q)

The Rational Method is used to calculate peak runoff in the irrigation fields as follows. Application of the rational method is normally limited to watershed of less than 0.5km^2.

$$Q = \frac{CIA}{360} = 0.00278CIA$$

Where,

Q = Design Peak Discharge (m^3/sec)

C = runoff coefficient, which indicates the proportion of the design rainfall that actually discharges rapidly from the basin and which contributes to the peak discharge. It is assumed that for the cultivated and flat land and soil of medium infiltration, the value of C=0.50.

A = Area to be drained, ha

I = Rainfall intensity in mm/h for the design return period and for a duration equal to the "time of concentration" of the watershed.

Peak runoff from the drainage catchments areas (for field and collector drain) can be calculated with the application of the following formula.

$$Q = 0.00278CIA$$

3.5 Irrigation Structures

3.5.1 General

Type and number of different structures required for the scheme have been determined based on the scheme layout. The structures should be designed as per standard criteria and local situation, so that it is hydraulically and structurally safe. To achieve this, we focus on many parameters like:

- foundation condition,
- topographic situation,
- type & availability of construction materials,
- purpose/function of structures,
- selection of type structures,
- construction materials selection,
- fixing design criteria,
- selection of construction method being adopte.etc so that economical and structurally safe structures that will perform efficiently and competently with minimum head/amount loss, easy operation, less maintenance cost, easily accessible etc. will be implemented.

There are different hydraulic structures proposed and designed for Irba-Girstu system including:

1. Regulating Structures: Head Regulator, Division Box & Offtake
2. Conveyance structures: Drops
3. Protective Structures: Road Crossing Culvert

The design of typical structures were done using standard procedures and presented in the following sections and using drawing in album & appendixes of engineering report.

3.5.2 Design of Division box Structures

A division structure regulates the flow from one canal to the second canal system. It normally consists of a box with vertical walls in which controllable openings are provided. The minimum dimensions of the structure depend on its performance in the fully open position. The width of the outlet is usually proportional to the division of water flow to be made. These structures are constructed of stone masonry. This type of structures incorporated in Irba-Girstu small-scale project to divide the flow from secondary canals to tertiary canals. A total of 14 division box structures are designed in the system and only sample of the designs are presented. The detail design is attached in appendix and drawing album.

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.

Design of IRSC-DB-1(Division Box: 1) (Sample design)

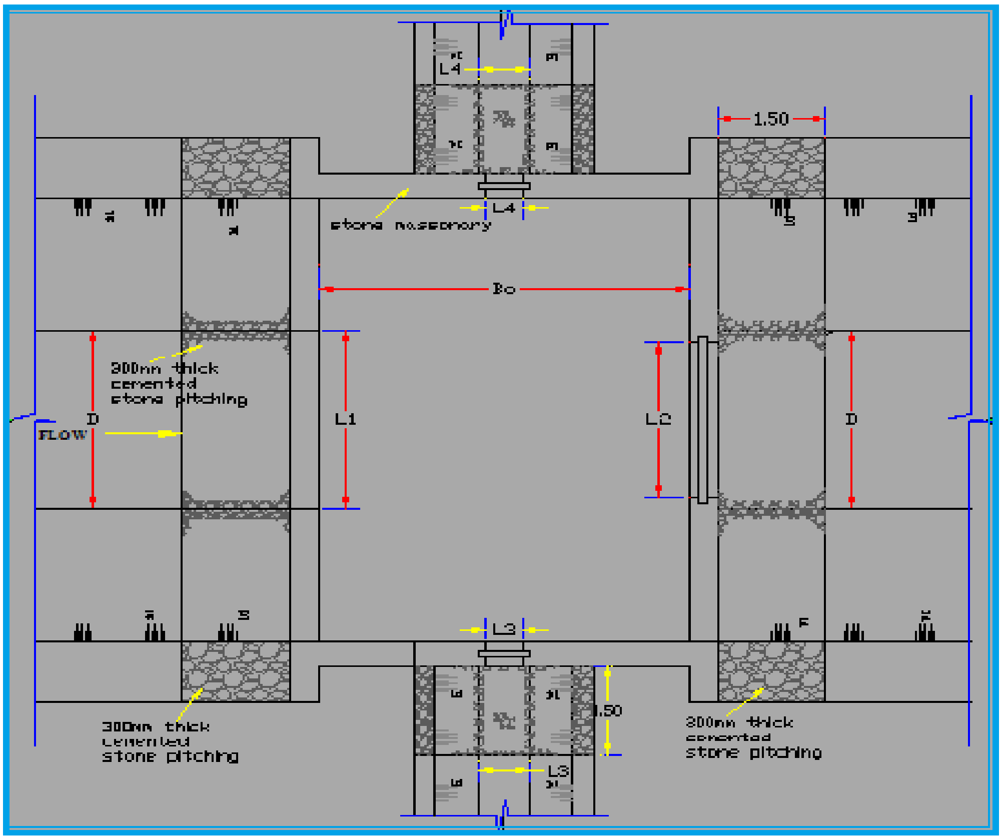

Figure 8: Plan of Sample Division Box

Station: From IRSC-1-1 to IRTC-1-1-1 and IRTC-1-1-3

Using broad crested formula, $Q= CL (h)^{3/2}$

Where

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

C= discharge coefficient, C= 1.7

L= effective length of crest in m

h= over flow depth, m

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

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

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

$Q_1/ Q_2 =L_1/ L_2$

$L_2 = L_1*Q_2/Q_1$

Where:

Q_1 = is flow in incoming parent canal, IRSC-1-1

Q_2 = is flow toward outgoing, IRSC-1-1

Q_3 = is flow to ward IRTC-1-1-1

Q_4 = is flow to ward IRTC-1-1-3

L_1= is effective crest length of weir sill across opening to IRSC-1-1

L_2= is effective crest length of weir sill across opening to IRSC-1-1

L_3= crest length of opening a cross the 3rd canal, IRTC-1-1-1

L_4= crest length of opening a cross the 4th canal, IRTC-1-1-3

The height of the division box, $D= d+f_b$

The width of the division box, $B= b+2*m*D$

Where: b= base width of the incoming canal

m= side slope of the incoming canal

D= total canal depth of the incoming canal

<u>IN CASE OF IRSC-1-DB-01</u>

$Q_1= 0.084m^3/s$, $Q_2=0.024m^3/s$

$Q_3 = 0.023m^3/s$, $Q_4 = 0.037m^3/s$

h = 0.15m

b =0.40

$L_1= Q_1 / Ch^{3/2} = 0.85$, adopt 0.3m opening size.

$L_2= L_1Q_2/ Q_1= 0.24m$, adopt 0.3m opening size.

$L_3= L_2Q_3/ Q_2 = 0.23m$, adopt 0.3m opening size

$L_4= L_3Q_4/ Q_3 = 0.37m$, adopt 0.40

Depth of Division Box, H = D of parent canal +some free board = 0.40m

Top width of Division Box = b of parent canal +2 *m*D = 1.20m. The division box is square, 1.20m*1.20m dimensions. The design of the other division boxes have done in similar method and presented in appendix-I and drawing album.

3.5.3 Turnouts/Off takes

Purpose of the turnout is to divert water from a supply canals to a smaller channel. The structure will usually consist of an inlet, a conduit or a means of conveying water through the bank of the supply channel and, where required, an outlet transition. Gates are generally used in the inlet to control the flow.

To divert irrigation water to the smallest irrigation canals, turnouts are proposed at appropriate locations. There are totally about 213 turnouts out of which 13 on right main canal-1, 4 on left main cana-1, 13 on both left and right secondary canal and 183 on both right and left tertiary canals. As indicated on the system layout, turnouts will take off directly from the main canals, secondary and tertiary canals. See the AutoCAD drawing for location and dimension of the turnouts.

Opening of the turnouts proposed to have rectangular shape and to be regulated with gates. Area of the opening is obtained from the following equation.

<u>One typical Turnout No.1 on IRMC-1 is taken</u>

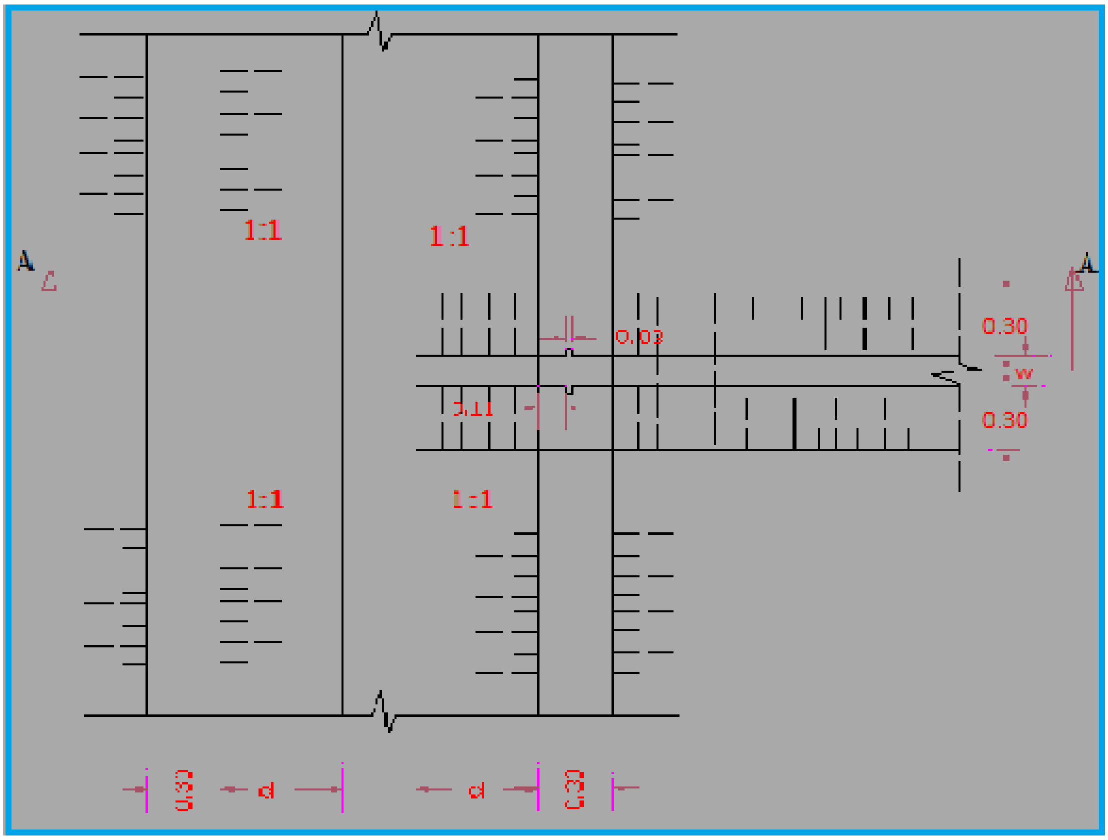

Figure 9: Plan of Sample Turnout

$$Q = CLH^{3/2}$$

Where,

H = head difference = 0.15m (water depth over the crest)

C= coefficient of discharge = 1.70

L = Crest length of opening across the off take (m)

Q = maximum discharge in the turnout (m^3/sec)

The discharge (Q) in the turnout gate can be calculated based on the water demand of the unit area and the given water duty.

Water duty = 1.52l/s/ha

Unit Area = 1.5ha

Maximum irrigation interval (for Tomato) = 12days

Assume, irrigation hours needed to irrigate 1ha = 8-10 hrs, take = 8hrs

$$\text{Peak water demand, Q} = \frac{\text{Water duty (3.04l/s/ha) X Crop Area (ha) X 12 (hrs) X Irrigation interval (days)}}{\textit{Irrigation Hours needed to irrigate the given area (hrs)}}$$

$$Q = \frac{\text{3.04(l/s/ha)X1.5ha X 12hrs/day X 12day}}{8hrs} = 3.52l/s = 0.00352\text{m}^3\text{/sec}$$

From $Q = CLH^{3/2}$,$L = \dfrac{Q}{CH^{3/2}} = 0.006m$

Since the opening is controlled with gate, take L =0.30m. The same equation is applied to design all turnouts. Summarized in Appendix-III and drawing album.

3.5.4 Design of Drop Structures

Whenever the available natural ground slope is steeper than the designed bed slope of the channel, the difference is adjusted by constructing vertical 'falls' or 'drops' in the canal bed at suitable intervals. The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from this drop, which may otherwise scour the bed and banks of the canal. The two common types of drops constructed are vertical drops and inclined drop. For Irba-Girstu Small Scale Irrigation Project vertical drops are adopted.

Sample Design: IRSC-1-1 @ 2+175m, Hr = 1.5m

Q= 0.35m^3/s, V= 1.45m/s,

b= 0.60m, d=0.40m,

CBL (Before) = 1305.02m

CBL (after fall) = 1303.52m

He= d+V^2/2g = 0.51m

Stilling basin, L= 3 (He*h) 1/2 = 2.62m

Width, W = 18.46* (Q)1/2/ (9.91+Q) = 1.06m. Adopt 1.10m

Sill height, X = 0.25 (He*h)2/3 = 0.17m, adopt 0.20m

For other drop structures on lined and earthen canals, the designs are done in similar way & summarized in Appendix-II and drawing album.

3.5.5 Design of Road Crossing Culvert Structures

Two Box culvert structures are provided both right and left main canals where the canals cross the main weathered road that runs from Delo-Mena to Berbere woreda. The hydraulic design of the sample structure is presented in Appendix-IV & drawing album.

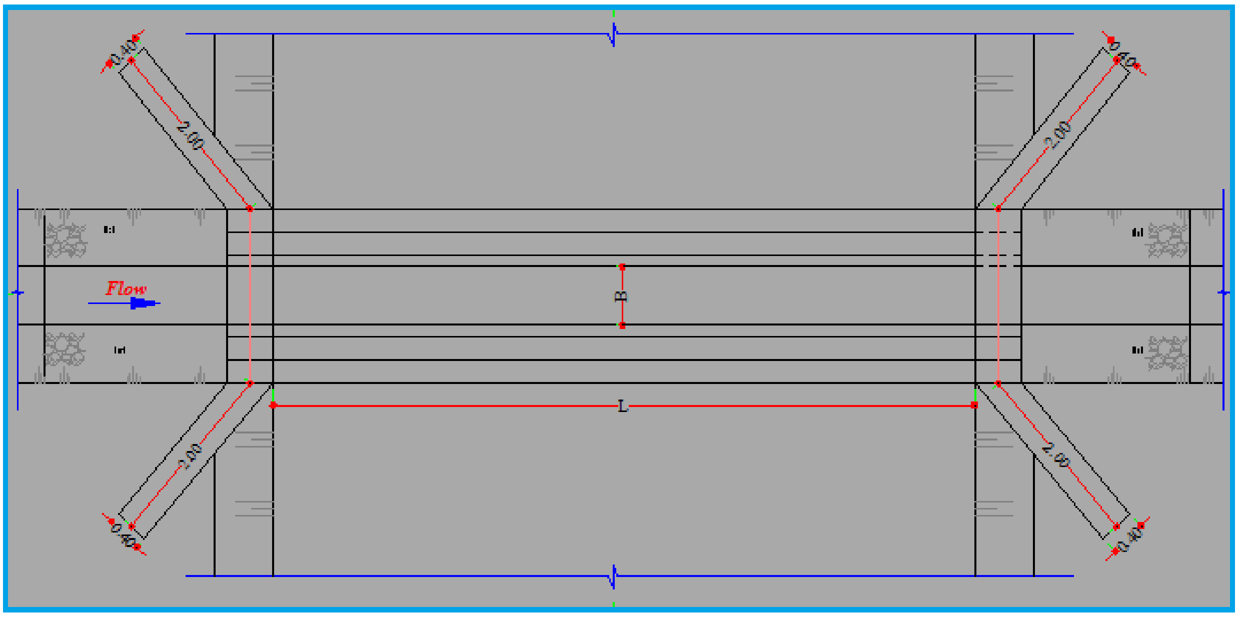

Figure 10: Plan of road crossing box culvert

4.0 BILL OF QUANTITIES & COST ESTIMATION

Quantity of each activity has been taken from the design drawings. Cost estimation of all activities has been carried out based on the unit rates obtained from the market. Total cost of the project is **43,957,821.37** Birr including 15% VAT. During construction, it is expected that beneficiary of the project to contribute about **13.93%** of the total project cost which is **6,121,170.47** Birr. The project cost per ha is about **161,017.66** Birr. Detail Bill of Quantities and cost estimation are shown in the table below.

Table 6: Summary of Costs by Major Activities

IRBA-GIRSTU SSIP BOQ AND COST ESTIMATE SUMMARY			
Part Nr.	**Description**	**Amount (ETB)**	Comm. Share
1	GENERAL PROVISION(WORKS)	4,057,435.20	
2	HEADWORK	3,018,545	
3	ROAD CROSSING CULVERT	664,002	
4	DROP STRUCTURE	3,837,765	
5	DIVISION BOX	132,184	
6	TURNOUT	531,833	
7	MAIN CANAL	15,631,591	
8	SECONDARY CANAL	2,196,943	
9	TERTIARY CANAL	2,032,724	
10	DRAINAGE CANAL	2,131,556	2,131,556
11	FIELD CANAL	2,610,405	2,610,405
12	FIELD DRAIN	1,379,209	1,379,209
Sub Total		**38,224,192.49**	**6,121,170.47**
VAT at 15%		**5,733,628.87**	
Total Carried To Form a BID		**43,957,821.37**	
Cost per hectare		**161,017.66**	

Table 7 : Cost of General Activities

BILL-1- GENERAL WORKS					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate**	**Total Cost**
1	General works				
1.1	Mobilization (Manpower, machinery, material, work commencement to be paid on his work schedule submission)	Ls	1	50,000.00	50,000.00
1.2	Demobilization is to be paid after taking over of the project	Ls	1	60,000.00	60,000.00
1.3	Furniture and office equipment for the site supervisor (these are considered as the property of the client and formally to be handed over)				
1.3.1	Supply standard single drawer table with chair;	Set	1	35000	35,000.00
1.3.2	Supply standard 90cm width and 16cm thick single sponge mattress, a pair of pillow, a pair of bed sheet (250cm), and Blanket;	Set	1	25000	25,000.00
1.3.3	Supply 90cm wide folding iron bed	No	1	7500	7,500.00
1.3.4	Supply a Motorcycle or Equivalent; Engine 2-Stroke, Single-Cylinder, air cooled; Fuel capacity: 13 liters; 200cc: Seat Height: 835mm; Height: 1150mm with all accessories.	No	1	200000	200,000.00
1.4	Camping (3m x 13.85m office & bed room, 4m x 6m kitchen & Cafeteria, 5m x 5m store, 4m x 2m Toilet & Shower, 2m x 2m guard house				
1.4.1	Site clearing	m^2	175	8.69	1,520.75
1.4.2	Excavation	m^3	63.34	53.7	3,401.14
1.4.3	Cart away all excess excavated material for safe place with a radius of more than 500m	m^3	88.52	65.58	5,805.14
1.4.4	25cm thick hard core	m^3	89.7	309.7	27,780.09
1.4.5	Masonry work with 1:3 mortar mix	m^3	38.4	2564.81	98,498.96
1.4.6	5cm thick mass concrete (1:2:4 mix ratio)	m^3	12.71	3407.1	43,304.24
1.4.7	2cm cement screed	m^2	91	240.7	21,903.70
1.4.8	CIS walling G-32	m^2	337	350	117,950.00
1.4.9	CIS roofing G-32	m^2	194.5	355.655	69,174.90
1.4.10	Chip wood wall ceiling	m^2	256	180	46,080.00
1.4.11	Supply, assemble and fix in position eucalyptus wall post of length 3 m with span length of 1.2m	No	161	209.2	33,681.20
1.4.12	Supply and fix purlin in Eucalyptus wood size 50 x 70 mm nailed into eucalyptus truss	m	586	100	58,600.00
1.4.13	Supply, assemble and fix in position eucalyptus roof truss	No	36	241.67	8,700.12
1.4.14	Supply and fix purlin in zigba wood size 50 x 70 mm nailed into eucalyptus truss including three coats of anti - termite external treatment	m	190	94.92	18,035.00
1.4.15	Supply and fix CIS doors size 1.0x2.10m	No	14	1500	21,000.00
1.4.16	Supply and fix CIS windows size 1x1.2m	No	9	500	4,500.00
1.4.17	Fence 2.0m height & 15cm φ eucalyptus poles placed every 2m with barbed wire at 20cm vertical interval & erected in 0.6m depth embedded with concrete	LS	1	50000	50,000.00

BILL-1- GENERAL WORKS					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate**	**Total Cost**
1.5	Access Road with average width of 5m and as per required depth of cut & fill to let vehicle access to it.	Ls	5	550000	2,750,000.00
1.6	Headwork site and main canal profile set out surveying works & check any logical contradiction whether headwork hydraulics consistent with designed canal profile & approved by engineer	Ls	1	55000	55,000.00
1.7	Dewatering	Ls	1	20000	20,000.00
1.8	Dyke/Coffer dam work (temporary diversion) to be paid 50% for construction, 50% for demolition or returning to natural condition.	Ls	1	90000	90,000.00
1.9	Sign post at junction and camp office, with dimension of 1m *1.5m of 2mm thick with 2.5m height angle iron pole founded on C-10 (1:3:6) mass concrete of 0.5m minimum depth	Ls	2	40000	80,000.00
1.1	Preparation of as-built drawings and site plan including operation and maintenance manual	Ls	1	55000	55,000.00
TOTAL OF BILL NO. 1 CARRIED TO SUMMARY					**4,057,435.25**

Table 8 : Cost of Headwork Construction

BILL 2 - HEADWORK					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
2	**Head work**				
2.1	Bulk Excavation of foundation	m^3	743.15	55.68	41,379
2.2	Excavation of soft rock including cart away at 50 meter far from the site	m^3	35.78	131.29	4,698
2.3	Excavation of Hard rock to an average depth of 0.5.00m including cart away at 50 meter far from the site	m^3	33.5	462.91	15,507
2.4	Providing and laying Plain cement concrete in C-10 (1:3:6)				
2.4.1	(U/S Floor	m^3	19.5	3,477.89	67,819
2.4.2	Base Weir	m^3	56.43	3,477.89	196,268
2.4.3	Weir main body	m^3	92.97	3,477.89	323,346
2.4.4	D/S Floor I Step	m^3	126.72	3,477.89	440,715
2.4.5	D/S Floor II Step	m^3	114.6	3,477.89	398,573
2.4.6	D/S Floor III Step	m^3	80.83	3,477.89	281,128
2.4.7	Below U/S Transition walls	m^3	10	3,477.89	34,779
2.4.8	Below Abutments	m^3	3	3,477.89	10,434
2.4.9	Below D/S Transition Walls	m^3	21	3,477.89	73,036
2.5	Stone Masonry in 1:4 cement sand mortar using stones from approved Quarry				
2.5.1	U/S Transition Walls	m^3	80.5	2,664.58	214,505
2.5.2	D/S Transition Walls	m^3	118.79	2,664.58	316,533
2.6	Providing and laying Reinforced cement concrete in C20 (1:1.5:3)				
2.6.1	U/S Cut-off Wall	m^3	14.63	3,911.41	57,204
2.6.2	D/S Cut-off Wall	m^3	12.75	3,911.41	49,870
2.7	Providing, bending and binding Tor Steel Grade 400, complete in all respect.				
2.7.1	Steel reinforcement @ 75 kg/cum of RCC	kg	20.55	3,500.00	71,925
2.8	Provide and place 0.50m thick stone Rip-Rap (with 1:3 mortar)				
2.8.1	U/S Launching Apron	m^3	34.08	1,011.48	34,469
2.8.2	D/S Launching Apron	m^3	29.71	1,011.48	30,050
2.9	25mm thick Cement Plaster on stone masonry in cement sand mortar 1:4.				
2.9.1	U/S Transition Walls	m^2	220.4	303.56	66,905
2.9.2	D/S Transition Walls	m^3	384.72	303.56	116,786
2.1	Back fill at the back of wing wall with materials excavated for foundation or from surrounding area	m^3	77	163.87	12,618
2.11	5mm thick Sliding Gates (1.32 height*1m width) including supply & installation	No	2	30,000.00	60,000
2.12	5mm thick Gates with Spindle (0.60 width* 0.65m height) including supply & installation	No	2	50,000.00	100,000
TOTAL OF BILL NO. 2 CARRIED TO SUMMARY					**3,018,545.15**

Table 9 : Cost of Road Crossing Construction

BILL 3 - ROAD CROSSING CULVERT					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
3	**Road Crossing Culvert**	2			
3.1	Soil Excavation (Soft Rock)	m^3	20.01	131.29	2,627.70
3.2	Fill and compaction (with selected material)	m^3	9.77	163.87	1,600.70
3.3	Masonry with 1:4 mortar	m^3	7.36	3122.94	22,981.70
3.4	Plastering (1:3 mix ratio mortar)	m^2	24.75	303.56	7,513.10
3.5	Cement Concrete (1:2:4)	m^3	1.09	3511.53	3,824.10
3.6	Supply and lay approved quality galvanized mild Steel reinforcement according to structural drawings. Reinforcement shall be free from dirt, oil, grease, paint, retarders, etc and any other substances which may affect the reinforcement and concrete bond.	Kg	178.7	3500	625,455.10
TOTAL OF BILL NO 3 CARRIED TO SUMMARY					**664,002.40**

Table 10 : Cost of Drops Structure Construction

BILL 4 - DROP STRUCTURE					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
4	**Drop Structures**				
4.1	Excavation for foundation in all sorts of soil including depositing the excavated stuff as and where directed.	m^3	4808.55	55.68	267,739.94
4.2	Backfill and Compaction i.e supply selected material fill with in 10km borrow area, spread & compact 20 cm thick layer by layer	m^3	2653.85	163.87	434,886.30
4.3	Cemented Stone Pitching	m^3	426.22	462.91	197,299.60
4.4	Masonry in Cement Mortar (1:4) including cost and conveyance of all materials, labour charges, curing etc complete.	m^3	854	3122.94	2,666,978.37
4.5	Plastering with Cement Mortar (1:4) mix 20 mm thick complete for exposed faces of masonry walls including racking of joints and curing etc.	m^2	892.28	303.56	270,860.40
TOTAL OF BILL NO. 4 CARRIED TO SUMMARY					**3,837,764.61**

Table 11 : Cost of Division Box Construction

BILL PART 5 - DIVISION BOX					
Item Nr.	Description	Unit	Quantity	Rate (ETB)	Amount (ETB)
5	Division Box				
5.1	Excavation for foundation in all sorts of soil including depositing the excavated stuff as and where directed.	m^3	25.93	56.56	1466.69
5.2	Backfill and Compaction i.e supply selected material fill with in 10km borrow area, spread & compact 20 cm thick layer by layer	m^3	9.17	163.87	1503.34
5.3	Cemented Stone Pitching	m^3	9.41	462.91	4353.67
5.4	Masonry in Cement Mortar (1:4) including cost and conveyance of all materials, labour charges, curing etc complete.	m^3	33.47	3122.94	104531.05
5.5	Plastering with Cement Mortar (1:4) mix 20 mm thick complete for exposed faces of masonry walls including racking of joints and curing etc.	m^2	66.97	303.56	20329.41
TOTAL OF BILL NO. 5 CARRIED TO SUMMARY					132,184.16

Table 12 : Cost of Turnout Construction

BILL 6 - TURNOUT					
Item Nr.	Description	Unit	Quantity	Rate (ETB)	Amount (ETB)
6	Turn Outs	No	213		
6.1	Site clearance to the depth of 15cm	m^2	208.3	9.26	1,928.86
6.2	Soil Excavation	m^3	103.7	56.56	5,865.01
6.3	Fill and compaction (excavated or surrounding soil)	m^3	58.28	163.87	9,550.34
6.4	Dry Stone pitching (hard Core)	m^3	47.15	462.91	21,828.52
6.5	Concrete works (with 1:2:4)	m^3	22.94	3,651.01	83,749.61
6.6	Plastering with 1:3 cement mortar	m^2	235.78	303.56	71,571.86
6.7	5mm thick double framed with angle iron Gate works supply & Installation	No	213	1,500.00	319,500.00
6.8	Form Work	m^2	88.85	200.77	17,838.41
TOTAL OF BILL NO 6 CARRIED TO SUMMARY					531,832.61

Table 13 : Cost of Lind Main Canals Construction

BILL 7- MAIN CANAL					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
7	**Lined Main Canal (IRMC-1 & ILMC-1)**				
7.1	**Concrete pipe on weir intake (2*15m)**	**m**			
7.2	Site clearance to the depth of 15cm	m^2	14543.64	9.26	134674.08
7.3	Soil Excavation	m^3	8303.73	56.56	469658.99
7.4	20cm thick hard core under base	m^3	659.66	462.91	305361.03
7.5	Cement Screed (5cm) with 1:3 mortar	m^2	4363.09	335.6	1464253.37
7.6	Masonry works with 1:3 cement mortar	m^3	2866.84	3122.94	8952980.52
7.7	Plastering (with 1:3 mortar)	m	12222.48	303.56	3710255.65
7.8	Compacted Back fill with surrounding excavated soil	m^3	3169.63	163.87	519407.68
7.9	Concrete Pipe (0.60m Diameter), transport & placing	m	30	2500	75000
TOTAL OF BILL NO. 7 CARRIED TO SUMMARY					**15,631,591**

Table 14 : Cost of Secondary Canal Construction

BILL 8- SECONDARY CANAL					
Bill No.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
8	**Secondary Canal**				
8.1	Clear and grub the area from bushes trees and shrubs on the alignment of drain canal to the width of canal and embankment	m^2	9,199.57	9.26	85,188.00
8.2	**Earth Work**			56.56	0
8.2.1	Compacted Fill	m^3	2,292.18	462.91	1,061,070.75
8.2.2	Canal Excavation	m^3	2,709.41	387.79	1,050,683.86
TOTAL OF BILL NO.8 CARRIED TO SUMMARY					**2,196,943**

Table 15 : Cost of Tertiary Canal Construction

BILL 9- TERTIARY CANAL					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
9	**Tertiary Canals**				
9.1	Clear and grub the area from bushes trees and shrubs on the alignment of drain canal to the width of canal and embankment	m^2	47644.86	9.26	441191.4
9.2	**Earth Work**				
9.2.1	Compacted Fill	m^3	8413.01	163.87	1378640.07
9.2.2	Canal Excavation	m^3	3823.5	55.68	212892.44
TOTAL OF BILL NO 9 CARRIED TO SUMMARY					**2,032,724**

Table 16 : Cost of Drainage Canals Construction

BILL 10 - DRAINAGE CANAL					
Item Nr.		**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
10	**Drainage Canals**				
10.1	**Tertiary Drainage Canal (IRTD & ILTD)**	**m**			
10.1.1	Site clearance to the depth of 15cm	m^2	51686.59	9.26	478,617.84
10.1.2	Soil Excavation	m^3	12735.47	56.56	720,318.28
10.1.3	Fill and compaction (excavated or surrounding soil)	m^3	2088.38	163.87	342,222.62
	Sub total				**1,541,158.74**
10.2	**Collector Drainage Canal**	**m**			
10.2.1	Site clearance to the depth of 15cm	m^2	11845.64	9.26	109,690.64
10.2.2	Soil Excavation	m^3	5673.35	56.56	320,884.60
10.2.3	Fill and compaction (excavated or surrounding soil)	m^3	975.3	163.87	159,822.20
	Sub total				**590,397.44**
TOTAL OF BILL NO. 10 CARRIED TO SUMMARY					**2,131,556**

Table 17 : Cost of Field Canals Construction

BILL 11-FIELD CANAL					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
12	**Field Canals (No =194)**				
13.1	Site clearance to the depth of 15cm	m^2	4850	9.26	44,911.00
13.2	Soil Excavation	m^3	17039.46	56.56	963,751.75
13.3	Fill & compaction with excavated or surrounding soil	m^3	9774.47	163.87	1,601,742.56
TOTAL OF BILL NO. 11 CARRIED TO SUMMARY					**2,610,405**

Table 18 : Cost of Field Drain Canals Box Construction

BILL PART 12- Field Drainage Canals					
Item Nr.	**Description**	**Unit**	**Quantity**	**Rate (ETB)**	**Amount (ETB)**
12	**Field Drainage Canals (No =205)**				
12.1	Site clearance to the depth of 15cm	m^2	2562.5	9.26	23,728.75
12.2	Soil Excavation	m^3	9002.81	56.56	509,198.73
12.3	Fill & compaction with excavated or surrounding soil	m^3	5164.35	163.87	846,281.51
TOTAL OF BILL NO. 12 CARRIED TO SUMMARY					**1,379,209**

3 PROJECT IMPLEMENTATION

3.1 Implementation Approach

After completion of tender documents, tendering and selection of contractors can be made. Most of the preparatory activities are expected to be carried out during the rainy season.The project can be implemented with local contractors. The duration of construction can be about one year.

3.2 Construction Schedule

For the entire project activities, the total time of construction is estimated to be about one year. The schedule is depicted in the following table.

Table 19 : Indicative Implementation Schedule of the Project by Major Activities

Project Activities	**Quarter-1**			**Quarter-2**			**Quarter-3**			**Quarter-4**		
	M1	**M2**	**M3**	**M4**	**M5**	**M6**	**M7**	**M8**	**M9**	**M10**	**M11**	**M12**
Preparatory Work												
Access Road construction												
Headwork Construction												
Road Crossing Culvert												
Division Box												
Drops on Main and Secondary Canals												
Secondary and Tertiary Canals												
Turn Outs on Main canals												
Main Canal (IRMC-1) & Main Canal (ILMC-1)												
Collectors & Tertiary drainage Canals												
Sign Post for the project												

5 CONCLUSION AND RECOMMENDATION

The design of Irba-Girstu small-scale irrigation project was intended to irrigate about **273 net** hectares of land. However, the study has revealed that minimum discharge of the source river is **880lit/sec** in the months of December and the maximum flow is **9770** lit/sec in August.

From the surveyed gross area of **400 hectares, 273** net irrigation land was identified. The project water resource can irrigate **579** ha of land in dry season. However, during fieldwork, we identify around 100ha with modern irrigation and about 200ha, local irrigation is found downstream of the project. Therefore, the remaining river flow together with Irba-Kela River can cover the downstream irrigable land.

The project has two main canal (right and left main canal) designed with maximum carrying capacity of **326** and **87.76 l/s** respectively.

Generally, the project is suitable to produce all the proposed crops. Therefore, this irrigation project is crucial for the community residing in the area, as its implementation will improve crop production and productivity of the area. However, due to high infiltration rate of the soil, main canals are masonry-lined canals. Therefore, the total project cost is a bit increased.

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APPENDICES

APPENDIX I: Division Box

Division Box	Parent Canal	Originating Canals	Chain age	Q_1	Q_2	Q_3	Q_4	h	$h^{3/2}$	L_1	L_2	L_3	L4	d	fb	D	b	B
			(m)	m^3/s	m^3/s	m^3/s	m^3/s	(m)		(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
ORSC-DB -01	IRSC-1-1 to	IRTC-1-1-1 and IRTC-1-1-3	6.31	**0.084**	0.024	0.0230	0.037	0.15	0.06	0.85	0.24	0.23	0.37	0.20	0.20	0.40	0.40	1.20
ORSC-DB -02		IRTC-1-1-2 and IRTC-1-1-4	155.72	0.024	0.000	0.0106	0.013	0.15	0.06	0.24	0.24	0.23	0.14	0.20	0.20	0.40	0.40	1.20
ORSC-DB -03	IRSC-1-2 to	IRTC-1-2-1 and IRFC-1-2-1	83.46	**0.25**	0.220	0.029	0.004	0.15	0.06	2.56	2.23	0.30	0.04	0.20	0.20	0.40	0.80	1.60
ORSC-DB -04		IRTC-1-2-2 and IRTC-1-2-8	315.99	0.22	0.164	0.023	0.033	0.15	0.06	2.23	1.66	0.24	0.33	0.20	0.20	0.40	0.80	1.60
ORSC-DB -05		IRTC-1-2-3 and IRTC-1-2-9	427.36	0.164	0.125	0.021	0.02	0.15	0.06	1.66	1.27	0.21	0.18	0.20	0.20	0.40	0.80	1.60
ORSC-DB -06		IRTC-1-2-4 and IRTC-1-2-10	524.07	0.125	0.077	0.022	0.026	0.15	0.06	1.27	0.78	0.22	0.27	0.20	0.20	0.40	0.80	1.60
ORSC-DB -07		IRTC-1-2-5 and IRTC-1-2-11	725.14	0.08	0.047	0.010	0.020	0.15	0.06	0.78	0.48	0.10	0.20	0.20	0.20	0.40	0.80	1.60
ORSC-DB -08	IRSC-1-3 to	IRTC-1-3-3 and IRTC-1-3-8	411.05	**0.09**	0.065	0.013	0.009	0.15	0.06	0.89	0.66	0.14	0.09	0.25	0.20	0.45	0.60	1.50
ORSC-DB -09		IRTC-1-3-4 and IRTC-1-3-9	525.37	0.07	0.007	0.027	0.031	0.15	0.06	0.66	0.07	0.27	0.31	0.25	0.20	0.45	0.60	1.50
ORSC-DB -10	ILSC-1-1 to	ILTC-1-1-5 and ILTC-1-1-10	491.93	**0.12**	0.098	0.006	0.011	0.15	0.06	1.17	0.99	0.06	0.12	0.25	0.20	0.45	0.60	1.50
ORSC-DB -11		ILTC-1-1-6 and ILTC-1-1-11	673.34	0.10	0.065	0.015	0.018	0.15	0.06	0.99	0.66	0.15	0.18	0.25	0.20	0.45	0.60	1.50
ORSC-DB -12		ILTC-1-1-7 and ILTC-1-1-12	766.63	0.06	0.040	0.011	0.014	0.15	0.06	0.66	0.40	0.11	0.14	0.25	0.20	0.45	0.60	1.50
ORSC-DB -13		ILTC-1-1-8 and ILTC-1-1-13	843.85	0.04	0.015	0.012	0.013	0.15	0.06	0.40	0.15	0.12	0.13	0.25	0.20	0.45	0.60	1.50
ORSC-DB -14		ILTC-1-1-9 and ILTC-1-1-14	928.21	0.01	0.000	0.006	0.009	0.15	0.06	0.15	0.00	0.06	0.09	0.25	0.20	0.45	0.60	1.50

APPENDIX II: Design of Drop Structures

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
IRMC-1-1	2175.00	595561.66	711280.12	1.50	0.35	0.59	1.45	2.11	0.60	0.40	0.25	0.51	0.87	2.62	1.06	0.21	1.50	1.50
	2275.00	595655.70	711246.13	1.00	0.35	0.59	1.45	2.11	0.60	0.40	0.25	0.51	0.71	2.14	1.06	0.16	1.50	1.50
	2350.00	595726.24	711220.64	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	2425.00	595796.77	711195.15	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	2500.00	595866.04	711167.46	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	2625.00	595931.16	711067.88	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	2700.00	595989.85	711021.18	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	2750.00	596027.10	710987.99	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	2825.00	596076.75	710931.78	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	2875.00	596109.84	710894.29	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	2950.00	596159.49	710838.08	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	3000.00	596192.58	710800.60	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	3050.00	596227.65	710765.03	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	3100.00	596265.38	710732.22	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	3175.00	596321.98	710683.01	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	3225.00	596359.71	710650.20	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
	3275.00	596397.39	710617.34	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
	3375.00	596472.72	710551.58	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	3650.00	596712.88	710426.90	0.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.52	1.57	1.15	0.11	1.50	1.50
	3750.00	596791.63	710365.63	1.50	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.91	2.72	1.15	0.22	1.50	1.50
	3875.00	596872.51	710271.63	1.00	0.41	0.64	1.71	2.92	0.60	0.40	0.25	0.55	0.74	2.22	1.15	0.17	1.50	1.50
IRSC-1-1	6.31	595440.07	711983.79	1.50	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.57	1.72	0.52	0.12	1.50	1.50
	25.00	595456.80	711992.13	1.00	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.47	1.41	0.52	0.09	1.50	1.50
	50.00	595479.17	712003.29	1.50	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.57	1.72	0.52	0.12	1.50	1.50
	75.00	595501.54	712014.44	2.00	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.66	1.99	0.52	0.14	1.50	1.50
	100.00	595523.92	712025.60	1.50	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.57	1.72	0.52	0.12	1.50	1.50
	125.00	595546.29	712036.75	1.50	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.57	1.72	0.52	0.12	1.50	1.50
	150.00	595568.66	712047.91	2.00	0.079	0.28	0.63	0.40	0.40	0.20	0.09	0.22	0.66	1.99	0.52	0.14	1.50	1.50
IRSC-1-2	25.00	596151.02	710885.40	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	83.46	596193.17	710925.91	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	150.00	596240.63	710972.55	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	225.00	596294.12	711025.12	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	250.00	596311.85	711042.74	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	300.00	596345.73	711079.51	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	325.00	596362.67	711097.90	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
	350.00	596379.61	711116.29	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	375.00	596396.55	711134.67	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	400.00	596413.49	711153.06	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	425.00	596430.43	711171.44	0.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.41	1.22	0.94	0.08	1.50	1.50
	450.00	596447.37	711189.83	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	475.00	596464.31	711208.22	2.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.82	2.45	0.94	0.19	1.50	1.50
	500.00	596481.24	711226.60	2.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.82	2.45	0.94	0.19	1.50	1.50
	525.00	596498.18	711244.99	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	575.00	596532.06	711281.76	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	625.00	596565.94	711318.54	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
	650.00	596582.88	711336.92	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	700.00	596616.76	711373.70	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	750.00	596650.64	711410.47	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	775.00	596667.58	711428.86	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	800.00	596684.52	711447.24	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	825.00	596701.46	711465.63	1.00	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.58	1.73	0.94	0.12	1.50	1.50
	850.00	596718.39	711484.01	1.50	0.27	0.52	0.81	0.66	0.80	0.30	0.16	0.33	0.71	2.12	0.94	0.16	1.50	1.50
IRSC-1-3	150.00	597047.56	710190.74	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
	225.00	597122.03	710185.09	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
	250.00	597147.03	710185.09	0.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.38	1.13	0.77	0.07	1.50	1.50
	275.00	597172.03	710185.09	0.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.38	1.13	0.77	0.07	1.50	1.50
	300.00	597197.03	710185.09	2.00	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.77	0.17	1.50	1.50
	311.68	597208.71	710185.09	2.00	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.77	0.17	1.50	1.50
	325.00	597222.03	710185.09	2.00	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.77	0.17	1.50	1.50
	350.00	597247.03	710185.09	2.00	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.77	0.17	1.50	1.50
	373.15	597270.18	710185.09	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
	375.00	597271.97	710185.55	0.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.38	1.13	0.77	0.07	1.50	1.50
	425.00	597320.33	710198.25	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
	450.00	597344.51	710204.60	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
	475.00	597368.69	710210.95	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
	500.00	597392.87	710217.29	2.00	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.77	0.17	1.50	1.50
	525.00	597417.05	710223.64	1.50	0.18	0.42	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.77	0.14	1.50	1.50
ILSC-1-1	8.00	596165.40	712507.46	0.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.38	1.13	0.75	0.07	1.50	1.50
	25.00	596164.38	712490.50	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	50.00	596162.44	712465.58	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	100.00	596158.56	712415.73	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
	125.00	596156.39	712390.85	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	200.00	596132.23	712319.85	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	225.00	596124.17	712296.18	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	250.00	596116.12	712272.51	1.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.53	1.60	0.75	0.11	1.50	1.50
	275.00	596107.11	712249.21	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	283.60	596103.77	712241.28	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	300.00	596099.49	712225.45	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	325.00	596092.96	712201.32	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	500.00	596214.98	712150.96	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	525.00	596225.67	712128.36	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	600.00	596257.73	712060.55	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
	675.00	596289.79	711992.75	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	700.00	596300.40	711970.11	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	725.00	596311.01	711947.48	2.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.85	2.54	0.75	0.20	1.50	1.50
	750.00	596321.62	711924.84	2.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.85	2.54	0.75	0.20	1.50	1.50
	775.00	596332.75	711902.47	2.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.85	2.54	0.75	0.20	1.50	1.50
	800.00	596344.90	711880.62	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	825.00	596357.05	711858.77	2.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.85	2.54	0.75	0.20	1.50	1.50

Canal	Chain age	Coordinates		Drop height	Q	$Q^{1/2}$	V(canal)	V^2	b(canal)	d(flow)	$d^{3/2}$	He	$(h*He)^{1/2}$	L(Stilling basin)	B(Stilling Basin)	a(Cistern height)	Protection	
		X	Y	m	m^3/s	m^3/s	m/s	m/s	m	m	m	m	m	m	m	m	L_1(U/s)	L_2(D/s)
	850.00	596370.16	711837.58	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	875.00	596386.24	711818.44	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	900.00	596402.33	711799.30	2.00	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.76	2.27	0.75	0.17	1.50	1.50
	925.00	596418.41	711780.16	1.50	0.17	0.41	0.84	0.71	0.60	0.25	0.13	0.29	0.66	1.97	0.75	0.14	1.50	1.50
ILTC-1-2	126.95	595745.14	711348.06	1.00	0.03	0.16	0.39	0.15	0.40	0.15	0.06	0.16	0.40	1.19	0.30	0.07	1.50	1.50
	200.00	595743.55	711420.85	0.75	0.03	0.16	0.39	0.15	0.40	0.15	0.06	0.16	0.34	1.03	0.30	0.06	1.50	1.50
ILTC-1-1-11	425.00	596443.45	712346.68	1.00	0.018	0.13	0.32	0.10	0.30	0.15	0.06	0.16	0.39	1.18	0.25	0.07	1.50	1.50
	502.88	596488.26	712408.15	1.00	0.018	0.13	0.32	0.10	0.30	0.15	0.06	0.16	0.39	1.18	0.25	0.07	1.50	1.50

APPENDIX III: Turnout

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
IRMC-1-1	IRFC-1-1	565.16	1309.566	1310.22	1309.831	0.65	0.40	0.20	1.40	0.56
	IRSC-1-1	1256.45	1309.013	1309.76	1309.392	0.75	0.40	0.20	1.60	0.68
	IRTC-1-1	2143.53	1305.154	1305.80	1305.223	0.65	0.40	0.20	1.20	0.37
	IRTC-1-2	2311.34	1301.924	1302.57	1302.336	0.65	0.40	0.20	1.20	0.71
	IRTC-1-3	2664.04	1293.808	1294.46	1294.323	0.65	0.40	0.20	1.20	0.81
	IRFC-1-2	2795.66	1291.018	1291.67	1291.21	0.65	0.40	0.20	1.20	0.49
	IRSC-1-2	2909.98	1288.332	1288.98	1288.685	0.65	0.40	0.20	1.20	0.65
	IRFC-1-3	2963.63	1287.011	1287.66	1287.366	0.65	0.40	0.20	1.20	0.66
	IRFC-1-4	3344.52	1277.725	1278.38	1278.161	0.65	0.40	0.20	1.20	0.74
	IRFC-1-5	3518.89	1275.179	1275.83	1275.483	0.65	0.40	0.20	1.20	0.60
	IRTC-1-4	3700.57	1273.589	1274.24	1273.719	0.65	0.40	0.20	1.20	0.43
	IRTC-1-5	3845.21	1271.221	1271.87	1271.522	0.65	0.40	0.20	1.20	0.60
	IRSC-1-3	3934.72	1269.684	1270.33	1270.015	0.65	0.40	0.20	1.20	0.63
ILMC-1-1	ILFC-1-1	282.84	1310.583	1311.133	1310.747	0.55	0.40	0.20	1.20	0.46
	ILFC-1-2	451.22	1310.347	1310.90	1310.6	0.55	0.40	0.20	1.20	0.55
	ILFC-1-3	566.74	1310.186	1310.74	1310.434	0.55	0.40	0.20	1.20	0.55

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	ILSC-1-1	913.16	1308.973	1309.52	1309	0.55	0.40	0.20	1.20	0.33
IRSC-1-2	IRTC-1-2-7	247.28	1282.917	1283.417	1282.921	0.50	0.40	0.20	1.40	0.30
	IRTC-1-2-6	842.50	1255.334	1255.83	1255.521	0.50	0.40	0.20	1.40	0.49
	IRTC-1-2-12	885.28	1253.684	1254.18	1253.679	0.50	0.40	0.20	1.40	0.00
IRSC-1-3	IRFC-1-3-1	76.51	1269.301	1269.751	1269.232	0.45	0.40	0.20	1.20	0.00
	IRTC-1-3-1	134.99	1269.009	1269.459	1269.003	0.45	0.40	0.20	1.20	0.00
	IRTC-1-3-5	206.58	1267.151	1267.601	1267.186	0.45	0.40	0.20	1.20	0.33
	IRTC-1-3-6	260.34	1264.882	1265.332	1265.157	0.45	0.40	0.20	1.20	0.57
	IRTC-1-3-2	277.39	1264.297	1264.747	1264.389	0.45	0.40	0.20	1.20	0.39
	IRTC-1-3-7	311.68	1262.126	1262.576	1261.641	0.45	0.40	0.20	1.20	0.00
ILSC-1-1	ILTC-1-1-1	13.04	1308.31	1308.76	1308.42	0.45	0.40	0.20	1.20	0.42
	ILTC-1-1-2	124.06	1302.75	1303.20	1302.70	0.45	0.40	0.20	1.20	0.00
	ILTC-1-1-3	260.30	1296.57	1297.02	1296.60	0.45	0.40	0.20	1.20	0.33
	ILTC-1-1-4	358.81	1289.08	1289.53	1289.11	0.45	0.40	0.20	1.20	0.33
IRTC-1-1	IRFC-1-1-1	80.87	1305.133	1305.43	1305.185	0.30	0.40	0.20	0.90	0.35
	IRFC-1-1-2	227.13	1304.914	1305.21	1305.187	0.30	0.40	0.20	0.90	0.57

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-1-3	352.78	1304.725	1305.03	1304.135	0.30	0.40	0.20	0.90	0.00
IRTC-1-2	IRFC-1-2-1	71.97	1301.902	1302.20	1301.64	0.30	0.40	0.20	1.00	0.00
	IRFC-1-2-2	222.25	1299.896	1300.20	1300.126	0.30	0.40	0.20	1.00	0.53
	IRFC-1-2-3	375.44	1299.636	1299.94	1299.671	0.30	0.40	0.20	1.00	0.33
	IRFC-1-2-4	469.31	1299.476	1299.78	1299.525	0.30	0.40	0.20	1.00	0.35
IRTC-1-3	IRFC-1-3-1	44.49	1293.914	1294.21	1294.226	0.30	0.40	0.20	1.00	0.61
	IRFC-1-3-2	164.51	1293.794	1294.09	1293.94	0.30	0.40	0.20	1.00	0.45
	IRFC-1-3-3	285.48	1293.673	1293.97	1293.847	0.30	0.40	0.20	1.00	0.47
	IRFC-1-3-4	405.77	1293.552	1293.85	1293.731	0.30	0.40	0.20	1.00	0.48
	IRFC-1-3-5	527.35	1293.431	1293.73	1293.24	0.30	0.40	0.20	1.00	0.00
	IRFC-1-3-6	650.85	1293.307	1293.61	1292.958	0.30	0.40	0.20	1.00	0.00
	IRFC-1-3-7	771.78	1293.186	1293.49	1292.47	0.30	0.40	0.20	1.00	0.00
IRTC-1-4	IRFC-1-4-1	46.83	1273.619	1273.92	1273.655	0.30	0.40	0.20	0.90	0.34
	IRFC-1-4-2	156.12	1273.455	1273.75	1273.674	0.30	0.40	0.20	0.90	0.52
	IRFC-1-4-3	258.07	1273.302	1273.60	1273.475	0.30	0.40	0.20	0.90	0.47
IRTC-1-5	IRFC-1-5-1	42.26	1271.308	1271.61	1271.551	0.30	0.40	0.20	0.90	0.54

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-5-2	139.92	1271.161	1271.46	1271.282	0.30	0.40	0.20	0.90	0.42
	IRFC-1-5-3	262.06	1270.978	1271.28	1271.211	0.30	0.40	0.20	0.90	0.53
	IRFC-1-5-4	385.60	1270.793	1271.09	1270.625	0.30	0.40	0.20	0.90	0.00
IRTC-1-1-1	IRFC-1-1-1-1	65.75	1308.806	1309.11	1308.856	0.30	0.40	0.20	0.90	0.35
	IRFC-1-1-1-2	198.69	1308.54	1308.84	1308.643	0.30	0.40	0.20	0.90	0.40
	IRFC-1-1-1-3	337.53	1308.263	1308.56	1308.395	0.30	0.40	0.20	0.90	0.43
	IRFC-1-1-1-4	541.84	1307.854	1308.15	1307.719	0.30	0.40	0.20	0.90	0.00
IRTC-1-1-2	IRFC-1-1-2-1	63.82	1297.244	1297.544	1297.384	0.30	0.40	0.20	0.90	0.44
	IRFC-1-1-2-2	200.69	1297.039	1297.339	1296.582	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-2-3	309.51	1296.876	1297.176	1296.869	0.30	0.40	0.20	0.90	0.00
IRTC-1-1-3	IRFC-1-1-3-1	63.66	1308.874	1309.224	1309.069	0.35	0.40	0.20	1.10	0.49
	IRFC-1-1-3-2	192.08	1308.746	1309.096	1308.864	0.35	0.40	0.20	1.10	0.42
	IRFC-1-1-3-3	313.44	1308.624	1308.974	1308.751	0.35	0.40	0.20	1.10	0.43
	IRFC-1-1-3-4	374.70	1308.563	1308.913	1308.496	0.35	0.40	0.20	1.10	0.00
	IRFC-1-1-3-5	485.19	1308.453	1308.803	1308.671	0.35	0.40	0.20	1.10	0.52
	IRFC-1-1-3-6	632.77	1308.305	1308.655	1308.493	0.35	0.40	0.20	1.10	0.49

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
IRTC-1-1-4	IRFC-1-1-4-1	61.07	1297.249	1297.549	1297.377	0.30	0.40	0.20	0.90	0.43
	IRFC-1-1-4-2	183.12	1297.066	1297.366	1297.022	0.30	0.40	0.20	0.90	0.00
IRTC-1-2-1	IRFC-1-2-1-1	131.67	1286.756	1287.106	1286.811	0.35	0.40	0.20	1.00	0.35
	IRFC-1-2-1-2	305.91	1286.512	1286.862	1286.633	0.35	0.40	0.20	1.00	0.42
	IRFC-1-2-1-3	478.67	1286.27	1286.62	1286.391	0.35	0.40	0.20	1.00	0.42
	IRFC-1-2-1-4	649.85	1286.031	1286.38	1286.248	0.35	0.40	0.20	1.00	0.52
	IRFC-1-2-1-5	823.17	1285.788	1286.14	1286.103	0.35	0.40	0.20	1.00	0.62
	IRFC-1-2-1-6	993.85	1285.549	1285.90	1285.689	0.35	0.40	0.20	1.00	0.44
IRTC-1-2-2	IRFC-1-2-2-1	69.37	1279.607	1280.01	1279.994	0.35	0.40	0.20	1.00	0.69
	IRFC-1-2-2-2	191.58	1279.485	1279.88	1279.66	0.35	0.40	0.20	1.00	0.48
	IRFC-1-2-2-3	313.06	1279.363	1279.76	1279.715	0.30	0.40	0.20	1.00	0.65
	IRFC-1-2-2-4	445.85	1279.204	1279.50	1279.183	0.30	0.40	0.20	1.00	0.00
	IRFC-1-2-2-5	571.81	1279.053	1279.35	1278.961	0.30	0.40	0.20	1.00	0.00
	IRFC-1-2-2-6	697.11	1278.903	1279.20	1278.936	0.30	0.40	0.20	1.00	0.33
	IRFC-1-2-2-7	825.35	1278.749	1279.05	1278.76	0.30	0.40	0.20	1.00	0.31
	IRFC-1-2-2-8	955.53	1278.592	1278.89	1278.763	0.30	0.40	0.20	1.00	0.47

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
IRTC-1-2-3	IRFC-1-2-3-1	54.61	1272.705	1273.00	1272.859	0.30	0.40	0.20	0.90	0.45
	IRFC-1-2-3-2	176.18	1272.522	1272.82	1272.667	0.30	0.40	0.20	0.90	0.44
	IRFC-1-2-3-3	303.59	1272.331	1272.63	1272.311	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-3-4	424.36	1272.15	1272.45	1272.167	0.30	0.40	0.20	0.90	0.32
	IRFC-1-2-3-5	547.76	1271.965	1272.27	1272.078	0.30	0.40	0.20	0.90	0.41
	IRFC-1-2-3-6	674.28	1271.775	1272.08	1271.918	0.30	0.40	0.20	0.90	0.44
	IRFC-1-2-3-7	809.06	1271.573	1271.87	1271.713	0.30	0.40	0.20	0.90	0.44
IRTC-1-2-4	IRFC-1-2-4-1	65.04	1266.933	1267.23	1266.712	0.30	0.40	0.20	1.00	0.00
	IRFC-1-2-4-2	188.02	1266.81	1267.11	1266.728	0.30	0.40	0.20	1.00	0.00
	IRFC-1-2-4-3	317.17	1266.681	1266.98	1266.82	0.30	0.40	0.20	1.00	0.44
	IRFC-1-2-4-4	440.04	1266.558	1266.86	1266.57	0.30	0.40	0.20	1.00	0.31
	IRFC-1-2-4-5	563.46	1266.435	1266.73	1266.617	0.30	0.40	0.20	1.00	0.48
IRTC-1-2-5	IRFC-1-2-5-1	67.45	1259.643	1259.94	1259.388	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-5-2	216.26	1259.42	1259.72	1259.263	0.30	0.40	0.20	0.90	0.00
IRTC-1-2-6	IRFC-1-2-6-1	68.34	1255.261	1255.56	1255.484	0.30	0.40	0.20	0.90	0.52
	IRFC-1-2-6-2	235.71	1255.01	1255.31	1255.144	0.30	0.40	0.20	0.90	0.43

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
IRTC-1-2-7	IRFC-1-2-7-1	72.70	1282.808	1283.11	1282.822	0.30	0.40	0.20	0.90	0.31
	IRFC-1-2-7-2	187.20	1282.636	1282.94	1282.383	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-7-3	324.23	1282.431	1282.73	1282.579	0.30	0.40	0.20	0.90	0.45
IRTC-1-2-8	IRFC-1-2-8-1	60.85	1279.535	1279.89	1279.768	0.35	0.40	0.20	1.00	0.53
	IRFC-1-2-8-2	181.25	1279.355	1279.70	1279.547	0.35	0.40	0.20	1.00	0.49
	IRFC-1-2-8-3	307.14	1279.166	1279.52	1279.348	0.35	0.40	0.20	1.00	0.48
	IRFC-1-2-8-4	439.43	1278.967	1279.32	1279.159	0.35	0.40	0.20	1.00	0.49
IRTC-1-2-9	IRFC-1-2-9-1	61.81	1272.775	1273.07	1272.838	0.30	0.40	0.20	0.95	0.36
	IRFC-1-2-9-2	183.96	1272.653	1272.95	1272.862	0.30	0.40	0.20	0.95	0.51
	IRFC-1-2-9-3	303.91	1272.533	1272.83	1272.753	0.30	0.40	0.20	0.95	0.52
	IRFC-1-2-9-4	424.00	1272.413	1272.71	1272.348	0.30	0.40	0.20	0.95	0.00
	IRFC-1-2-9-5	546.95	1272.29	1272.59	1272.435	0.30	0.40	0.20	0.95	0.45
	IRFC-1-2-9-6	667.71	1272.169	1272.47	1272.19	0.30	0.40	0.20	0.95	0.32
IRTC-1-2-10	IRFC-1-2-10-1	61.58	1266.856	1267.16	1266.897	0.30	0.40	0.20	0.90	0.34
	IRFC-1-2-10-2	192.43	1266.66	1266.96	1266.943	0.30	0.40	0.20	0.90	0.58
	IRFC-1-2-10-3	322.95	1266.464	1266.76	1267.061	0.30	0.40	0.20	0.90	0.90

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-2-10-4	453.43	1266.268	1266.57	1266.371	0.30	0.40	0.20	0.90	0.40
	IRFC-1-2-10-5	584.48	1266.072	1266.372	1265.998	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-10-6	714.65	1265.876	1266.176	1266.26	0.30	0.40	0.20	0.90	0.68
IRTC-1-2-11	IRFC-1-2-11-1	64.95	1259.647	1259.947	1259.031	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-11-2	206.76	1259.434	1259.734	1259.516	0.30	0.40	0.20	0.90	0.38
	IRFC-1-2-11-3	347.53	1259.223	1259.523	1259.18	0.30	0.40	0.20	0.90	0.00
	IRFC-1-2-11-4	487.55	1259.013	1259.313	1259.17	0.30	0.40	0.20	0.90	0.45
	IRFC-1-2-11-5	629.34	1258.8	1259.1	1258.979	0.30	0.40	0.20	0.90	0.48
	IRFC-1-2-11-6	771.93	1258.587	1258.887	1258.653	0.30	0.40	0.20	0.90	0.37
IRTC-1-2-12	IRFC-1-2-12-1	114.08	1253.463	1253.813	1253.625	0.35	0.40	0.20	1.00	0.46
	IRFC-1-2-12-2	270.07	1253.229	1253.579	1253.396	0.35	0.40	0.20	1.00	0.47
	IRFC-1-2-12-3	416.92	1253.009	1253.36	1253.237	0.35	0.40	0.20	1.00	0.53
	IRFC-1-2-12-4	557.80	1252.797	1253.15	1252.99	0.35	0.40	0.20	1.00	0.49
	IRFC-1-2-12-5	698.71	1252.586	1252.94	1252.724	0.35	0.40	0.20	1.00	0.44
	IRFC-1-2-12-6	864.36	1252.337	1252.69	1252.54	0.35	0.40	0.20	1.00	0.50
IRTC-1-3-1	IRFC-1-3-1-1	53.47	1268.891	1269.19	1269.105	0.30	0.40	0.20	0.90	0.51

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-3-1-2	174.96	1268.624	1268.92	1268.497	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-1-3	300.34	1268.348	1268.648	1268.384	0.30	0.40	0.20	0.90	0.34
	IRFC-1-3-1-4	423.39	1268.078	1268.378	1267.868	0.30	0.40	0.20	0.90	0.00
IRTC-1-3-2	IRFC-1-3-2-1	63.79	1264.151	1264.451	1263.847	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-2-2	191.25	1263.96	1264.26	1263.909	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-2-3	317.86	1263.77	1264.07	1263.894	0.30	0.40	0.20	0.90	0.42
	IRFC-1-3-2-4	442.73	1263.647	1263.947	1263.691	0.30	0.40	0.20	0.90	0.34
IRTC-1-3-3	IRFC-1-3-3-1	88.04	1253.491	1253.791	1253.533	0.30	0.40	0.20	0.90	0.34
	IRFC-1-3-3-2	239.91	1253.339	1253.639	1253.307	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-3-3	374.57	1253.204	1253.504	1253.496	0.30	0.40	0.20	0.90	0.59
	IRFC-1-3-3-4	511.54	1253.067	1253.367	1253.102	0.30	0.40	0.20	0.90	0.33
IRTC-1-3-4	IRFC-1-3-4-1	89.34	1244.873	1245.173	1244.881	0.30	0.40	0.20	1.00	0.31
	IRFC-1-3-4-2	283.04	1244.583	1244.883	1244.393	0.30	0.40	0.20	1.00	0.00
	IRFC-1-3-4-3	434.92	1244.355	1244.655	1244.323	0.30	0.40	0.20	1.00	0.00
	IRFC-1-3-4-4	563.12	1244.163	1244.463	1244.062	0.30	0.40	0.20	1.00	0.00
IRTC-1-3-5	IRFC-1-3-5-1	69.82	1267.031	1267.331	1267.121	0.30	0.40	0.20	0.90	0.39

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-3-5-2	164.70	1266.936	1267.236	1267.087	0.30	0.40	0.20	0.90	0.45
IRTC-1-3-6	IRFC-1-3-6-1	83.08	1264.708	1265.008	1264.325	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-6-2	204.84	1264.525	1264.825	1264.771	0.30	0.40	0.20	0.90	0.55
	IRFC-1-3-6-3	294.56	1264.391	1264.691	1264.384	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-6-4	377.09	1264.267	1264.567	1264.232	0.30	0.40	0.20	0.90	0.00
IRTC-1-3-7	IRFC-1-3-7-1	82.31	1261.952	1262.252	1261.968	0.30	0.40	0.20	0.90	0.32
	IRFC-1-3-7-2	203.99	1261.77	1262.07	1261.729	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-7-3	324.06	1261.59	1261.89	1261.594	0.30	0.40	0.20	0.90	0.30
	IRFC-1-3-7-4	456.32	1261.391	1261.691	1261.334	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-7-5	520.94	1261.294	1261.594	1261.225	0.30	0.40	0.20	0.90	0.00
IRTC-1-3-8	IRFC-1-3-8-1	96.55	1253.434	1253.734	1253.393	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-8-2	219.92	1253.249	1253.549	1253.623	0.30	0.40	0.20	0.90	0.67
	IRFC-1-3-8-3	345.85	1253.06	1253.36	1252.97	0.30	0.40	0.20	0.90	0.00
IRTC-1-3-9	IRFC-1-3-9-1	62.95	1244.913	1245.213	1244.725	0.30	0.40	0.20	0.90	0.00
	IRFC-1-3-9-2	185.46	1244.729	1245.029	1245.042	0.30	0.40	0.20	0.90	0.61
	IRFC-1-3-9-3	308.18	1244.545	1244.845	1244.361	0.30	0.40	0.20	0.90	0.00

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-3-9-4	472.03	1244.299	1244.599	1244.086	0.30	0.40	0.20	0.90	0.00
ILTC-1-1-1	IRFC-1-1-1-1	66.25	1308.192	1308.492	1308.22	0.30	0.40	0.20	1.00	0.33
	IRFC-1-1-1-2	185.55	1308.072	1308.372	1308.193	0.30	0.40	0.20	1.00	0.42
	IRFC-1-1-1-3	305.57	1307.952	1308.252	1308.042	0.30	0.40	0.20	1.00	0.39
	IRFC-1-1-1-4	404.72	1307.853	1308.153	1307.969	0.30	0.40	0.20	1.00	0.42
	IRFC-1-1-1-5	537.22	1307.721	1308.021	1307.958	0.30	0.40	0.20	1.00	0.54
ILTC-1-1-2	IRFC-1-1-2-1	63.69	1302.607	1302.907	1302.917	0.30	0.40	0.20	0.90	0.61
	IRFC-1-1-2-2	187.81	1302.421	1302.721	1302.869	0.30	0.40	0.20	0.90	0.75
	IRFC-1-1-2-3	318.07	1302.226	1302.526	1302.374	0.30	0.40	0.20	0.90	0.45
	IRFC-1-1-2-4	445.44	1302.035	1302.335	1301.816	0.30	0.40	0.20	0.90	0.00
ILTC-1-1-3	IRFC-1-1-3-1	54.43	1296.44	1296.74	1296.515	0.30	0.40	0.20	0.90	0.38
	IRFC-1-1-3-2	176.05	1296.258	1296.558	1296.272	0.30	0.40	0.20	0.90	0.31
	IRFC-1-1-3-3	340.04	1296.012	1296.312	1295.345	0.30	0.40	0.20	0.90	0.00
ILTC-1-1-4	IRFC-1-1-4-1	74.72	1288.917	1289.217	1288.947	0.30	0.40	0.20	0.90	0.33
	IRFC-1-1-4-2	171.63	1288.772	1289.072	1288.913	0.30	0.40	0.20	0.90	0.44
ILTC-1-1-5	IRFC-1-1-5-1	37.08	1288.326	1288.626	1288.542	0.30	0.40	0.20	0.90	0.52

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-1-5-2	120.83	1288.243	1288.543	1288.459	0.30	0.40	0.20	0.90	0.52
ILTC-1-1-6	IRFC-1-1-6-1	98.93	1282.308	1282.608	1282.348	0.30	0.40	0.20	0.90	0.34
	IRFC-1-1-6-2	220.48	1282.126	1282.426	1282.25	0.30	0.40	0.20	0.90	0.42
	IRFC-1-1-6-3	339.37	1281.947	1282.247	1282.397	0.30	0.40	0.20	0.90	0.75
ILTC-1-1-7	IRFC-1-1-7-1	37.74	1272.933	1273.233	1272.506	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-7-2	135.89	1272.786	1273.086	1272.908	0.30	0.40	0.20	0.90	0.42
	IRFC-1-1-7-3	286.21	1272.561	1272.861	1272.647	0.30	0.40	0.20	0.90	0.39
ILTC-1-1-8	IRFC-1-1-8-1	62.27	1265.592	1265.892	1265.822	0.30	0.40	0.20	0.90	0.53
	IRFC-1-1-8-2	179.27	1265.475	1265.775	1265.636	0.30	0.40	0.20	0.90	0.46
ILTC-1-1-9	IRFC-1-1-9-1	83.57	1257.632	1257.932	1257.834	0.30	0.40	0.20	0.90	0.50
ILTC-1-1-10	IRFC-1-1-10-1	68.58	1288.261	1288.561	1288.268	0.30	0.40	0.20	0.90	0.31
	IRFC-1-1-10-2	189.65	1288.079	1288.379	1288.281	0.30	0.40	0.20	0.90	0.50
	IRFC-1-1-10-3	314.05	1287.892	1288.192	1287.936	0.30	0.40	0.20	0.90	0.34
	IRFC-1-1-10-4	382.27	1287.79	1288.09	1287.808	0.30	0.40	0.20	0.90	0.32
	IRFC-1-1-10-5	532.59	1287.565	1287.865	1287.512	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-10-6	634.77	1287.411	1287.711	1287.722	0.30	0.40	0.20	0.90	0.61

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
ILTC-1-1-11	IRFC-1-1-11-1	93.94	1282.315	1282.615	1282.256	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-11-2	201.16	1282.155	1282.455	1281.788	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-11-3	324.20	1281.97	1282.27	1281.853	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-11-4	448.51	1280.784	1281.084	1281.037	0.30	0.40	0.20	0.90	0.55
	IRFC-1-1-11-5	571.13	1279.6	1279.9	1279.534	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-11-6	692.31	1279.418	1279.718	1279.161	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-11-7	753.55	1279.326	1279.626	1279.03	0.30	0.40	0.20	0.90	0.00
ILTC-1-1-12	IRFC-1-1-12-1	49.45	1272.916	1273.216	1272.956	0.30	0.40	0.20	0.90	0.34
	IRFC-1-1-12-2	171.66	1272.732	1273.032	1272.856	0.30	0.40	0.20	0.90	0.42
	IRFC-1-1-12-3	293.68	1272.549	1272.849	1272.584	0.30	0.40	0.20	0.90	0.33
	IRFC-1-1-12-4	412.95	1272.37	1272.67	1272.117	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-12-5	567.32	1272.139	1272.439	1272.32	0.30	0.40	0.20	0.90	0.48
	IRFC-1-1-12-6	688.97	1271.956	1272.256	1272.176	0.30	0.40	0.20	0.90	0.52
ILTC-1-1-13	IRFC-1-1-13-1	53.94	1265.55	1265.85	1265.68	0.30	0.40	0.20	0.90	0.43
	IRFC-1-1-13-2	176.47	1265.427	1265.727	1265.638	0.30	0.40	0.20	0.90	0.51
	IRFC-1-1-13-3	301.35	1265.302	1265.602	1265.577	0.30	0.40	0.20	0.90	0.57

Parent canal	Off take canal	Chain age	CBL	BTL	OGL	Canal depth	Top Bank width, b	Depth below CBL,t	Length, L	$(Cut)_d$
		m	m	m	m	m	m	m	m	
	IRFC-1-1-13-4	424.65	1265.179	1265.479	1265.273	0.30	0.40	0.20	0.90	0.39
	IRFC-1-1-13-5	568.07	1265.036	1265.336	1265.165	0.30	0.40	0.20	0.90	0.43
ILTC-1-1-14	IRFC-1-1-14-1	68.07	1257.58	1257.88	1257.369	0.30	0.40	0.20	0.90	0.00
	IRFC-1-1-14-2	197.72	1257.385	1257.685	1257.433	0.30	0.40	0.20	0.90	0.35
	IRFC-1-1-14-3	341.87	1257.169	1257.469	1257.529	0.30	0.40	0.20	0.90	0.66

APPENDIX IV: Road Crossing Box Culvert

Ser No.	Parameters	Canal	IRMC-1	ILSC-1
	Canal Data	**Symbol/Chainage(m)**	**2+103.96**	**0+368.35**
1	General ground level	GGL	1305.83	1289.28
2	U/S Full supply level	U/S_FSL	1305.83	1289.28
3	U/S Water depth	US_WD	0.50	0.25
4	U/S canal bed level	US_CBL	1305.33	1289.03
5	D/S full supply level	D/S_FSL	1305.83	1289.28
6	D/S canal bed level	DS_CBL	1305.33	1289.03
7	D/S Water Depth	DS_WD	0.50	0.25
8	Free Board	FB	0.25	0.25
9	Side Slope	SS	0.00	1.00
10	U/S Side Width (FB+FSD)*SS	U/S_SW	0.00	0.50
11	D/S Side Width	D/S_SW	0.00	0.50
12	U/S Bed Width	U/S_BW	1.00	0.60
13	D/S Bed Width	D/S_BW	1.00	0.60
	Box Data			
1	Number of pipe rows	NPR	1.00	1.00
2	Box Internal Depth	PID	0.75	0.50
3	U/S Box invert level	U/S_PIL	1305.47	1289.06
4	D/s Box invert level	D/S_PIL	1305.44	1289.05
5	Box Length (RW+2*WWW)	BL	6.80	6.80
6	U/S transition (1.2 for ID<0.50 & 1.5 for ID>=0.50)	U/S_Tran	1.50	1.20
7	D/S transition	D/S_Trans	1.50	1.20
8	U/S and D/S Guide wall length = 2*(La+Lb+Lc)	L_GW	8.38	6.68
9	Top width of Guide wall	TW_GW	0.30	0.30
10	Width of Guide wall Base Concrete	BW_GW	0.80	0.80
11	Length of Guide wall base concrete =	LBC_GW	7.78	6.08

Ser No.	Parameters	Canal	IRMC-1	ILSC-1
	2*(La'+Lb'+Lc')			
12	Thickness of Guide wall Base Concrete	TBC_GW	0.15	0.15
13	Width of Box bedding Concrete Base	WBC_PB	1.19	0.94
14	Thickness of Box bedding Base Concrete	TBC_PB	0.20	0.20
15	Length of Box bedding Base concrete	LBC_PB	6.50	6.50
	Road Data			
1	Road/embankment top level	RTL	1306.55	1289.75
2	Top width of Road/Embankment	REW	6.00	6.00
3	Embankment height	H	0.50	0.50
4	Embankment Side Slope	EM_SS	1.00	1.00