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HEAD WORK

1.1 . DIVERSION WEIR

Laga Kolu small scale irrigation project is aimed at ensuring food self-sufficiency as well as alleviating poverty by making use of surface water (Kolu River). The project is intended to irrigate 105.5 ha of land. Major activities during the execution of the project will be the construction of head work, irrigation canals and different irrigation and drainage structures.

1.1.1   Objective of the Project

The main objective of the head work is to provide sufficient irrigation water at upstream side of the weir during the period of low flow. And the work, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head in to the canal.

1.1.2   Shape of the weir crest

Weirs differ in type and shape, but designed and constructed to serve the same purposes. The following points are considered to determine the type and shape of the weir suited to this specific site.

- ❖ A weir with a shape that cannot easily be constructed by local manpower should not be considered.

- ❖ The availability of the skilled manpower for implementing it.

- ❖ The skill of the local builders, to perform as per design and specification.

- ❖ Taking into account the cited points and other factors, broad crest stone masonry weir type with 1:1 slope of downstream face is adopted. The stone masonry embedded in cement mortar will be constructed on the mass concrete (plum concrete).

1.1.3   Length of the weir

The length of water way should be adequate to pass the design flood safely. For alluvial river it is usually determined from the Lacey wetted perimeter (p).

The wetted perimeter (p) is given by

$P = 4.75\sqrt{Q}$

Where Q = 106.83 design discharge (m3/s)

P = 4.75√106.83 = 49.1 m

But, the length of the weir as these empirical formulae is very huge, so considering the physical condition of the weir site, the peak flow discharge which pass through the channel and as the most economical section to the project 29.00 m width is taken.

1.2 Hydraulic Design of the Weir

A diversion weir raises the water level on its upstream side to create the head necessary to divert the flow through the canal intake. So the following data have been obtained from the topo map and the previous design document.

Design considerations:

Design flood = 106.83m/s

Maximum flood level = 1632.6masl

Minimum river bed level = 1629.4masl

Under sluice bed level = 1629.6masl

1.1.1 *Estimation of the water depth*

After the design peak discharge is calculated, it is obvious to find the T.W.D. which will be used for deciding the bottom elevation of the downstream floor.

Table 1 Average River Bed Slope

No	Station	Distance (m)	Elevation (m)	Cumulative height (m)	Area At (m2)
0.00	0.00	0.00	1641.13	0.00	0.00
1.00	120.00	120.00	1635.78	5.35	320.98
2.00	240.00	120.00	1629.49	11.64	1019.19
3.00	247.75	7.75	1629.40	11.73	90.52
4.00	380.00	132.25	1622.94	18.19	1978.56
5.00	600.00	220.00	1614.82	26.31	4895.56
6.00	648.98	48.98	1613.38	27.75	1323.88
Total		648.98			9628.69

Cumulative Height, $H = E_{ln} - E_{lo}$

Area, $A_n = [(H_n + H_{n-1})/2)] \times L_n$

Average Height, $H_{avg.} = 2 A / L$

Average Height, $H_{avg.} = 29.67$ m

Average Slope, $I_{avg.} = H_{avg.} / L$

Average Slope, $I_{avg.} = 0.0457$ m/m   take   0.046   m/m

Roughness Coefficient of the River bed = 0.035

Table 2 Discharge of the River

From the Manning formula $V = 1/n * R^{2/3} * I^{1/2}$, $Q = V*A$

Elevatiom	Bed	W.Depth	Water	Wett.Perm.	Hydraulic	Slope	Roughness	Velocity	Discharge
El.(m)	Width (m)	d (m)	Area A(m2)	P (m)	Rad. R (m)	I avg (m/m)	Coeff. N	V (m/Sec)	Q (m3/Sec)
1629.40	1.00	0.00	0.00	1.00	0.00	0.0460	0.035	0.00	0.00
1629.90		0.50	1.14	9.16	0.25	0.0460	0.035	1.52	1.73
1630.40		1.00	5.68	36.57	0.29	0.0460	0.035	1.77	10.04
1630.90		1.50	14.77	82.16	0.32	0.0460	0.035	1.95	28.80
1631.40		2.00	27.69	139.66	0.34	0.0460	0.035	2.08	57.66
1631.90		2.50	42.67	201.76	0.35	0.0460	0.035	2.17	92.77
1632.15		2.75	50.63	250.42	0.34	0.0460	0.04	2.11	106.83
1632.40		3.00	62.61	283.83	0.36	0.0460	0.035	2.24	139.99
1632.90		3.50	75.16	335.38	0.37	0.046	0.035	2.26	169.86

Weir X-section
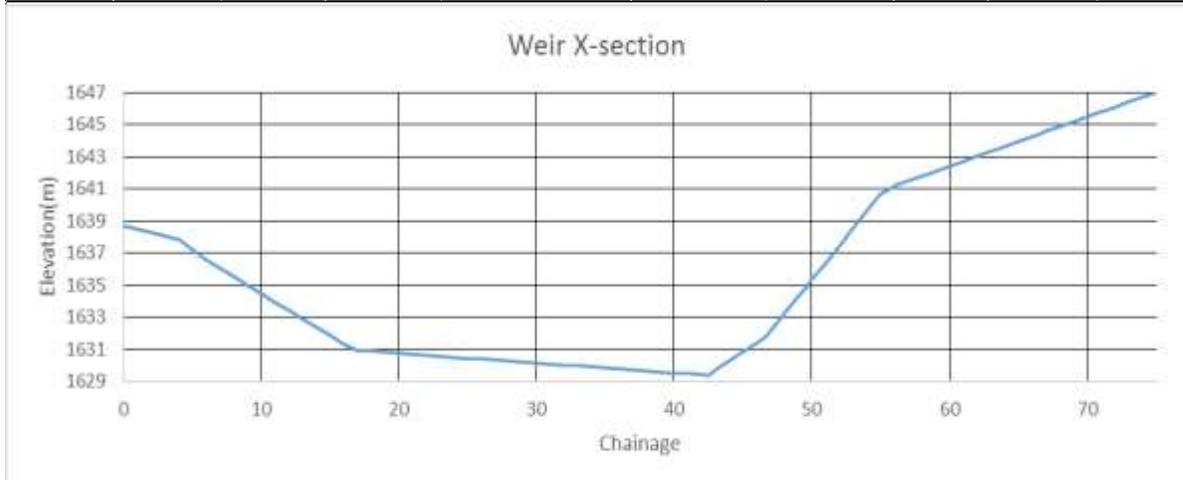
Figure 1 River Cross Section at Head work Site
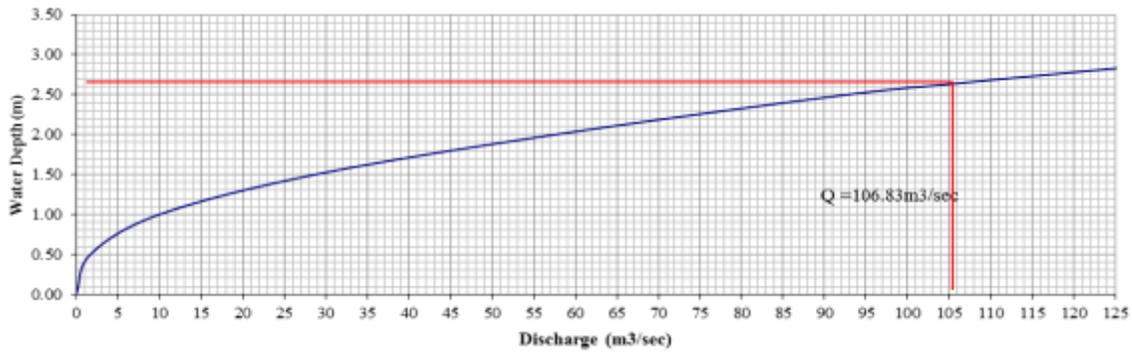
Figure 2 Stage discharge curve

From Water depth - Discharge curve, T.W.D equivalent to the flood discharge is found to be 2.75m.
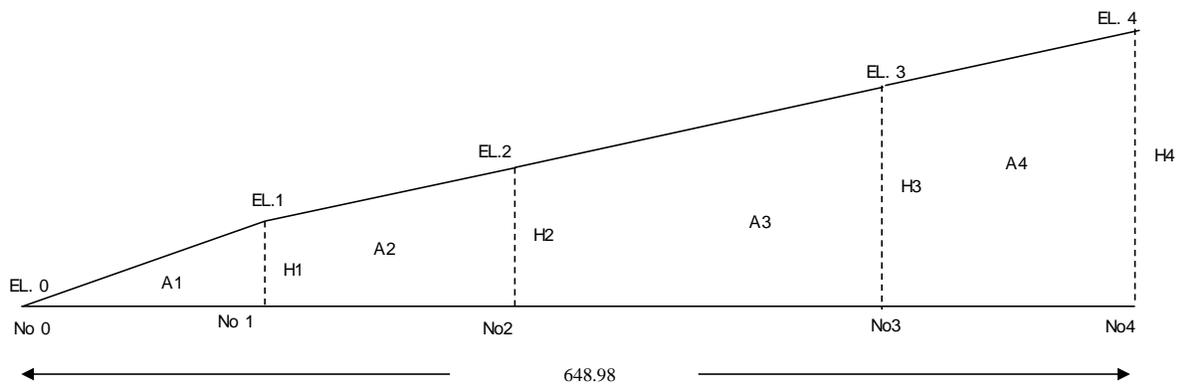
1.1.2 *Fixing weir crest level*

At the location of the selected weir site, the minimum river bed level is approximately 1629.4 above mean sea level. The weir crest elevation is fixed with reference of this river bed elevation considering the following factors.

❖ The crest level should be set at desired height or level to be able to obtain the required driving head to safely deliver the designed discharge to main canal.

❖ The weir crest should be set to allow a safely passage of maximum flood discharge within designed weir crest length.

❖ The bed level of the under sluice should be below sill level of canal head regulator.

❖ The main canal at the head reach should not be too deep in order to avoid large excavation work, to minimize construction cost and to reduce maintenance and side slopes stability problems.

Height of weir was calculated based on flowing data

. Average level of the highest field (a) =1629.54m

. Water depth required (b) = 0.47m

. Head loss across the field (c)=0.00m

. Head loss at the turnout (d) = 0.05m

. Slope of canal * Distance from the weir (e) = 0.001*870.94=0.87m

. Head loss across head regulator (f) = 0.00m

. Crest level of the weir =a+b+c+d+e+f= 1631m

Crest Height = Crest level - river bed level = 1.6m

1.1.3 *Flow depth over the weir crest*

Flow depth over the weir crest was computed by the following empirical formula.

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

$$H_e = (Q/CL)^{2/3}$$

Where,

Q = peak flood discharge in m^3/s = 106.83 m^3/s

L = length of weir crest in m = 29.00 m

H_e = over flow depth including approaching velocity head in m

C = discharge coefficient which varies = 1.70

$H_e=[109.78/(1.7*29)]^{2/3}=1.67m$

Hydraulic design of the weir
Water depth on the crest

Hva=0.07

Hd=1.6   He=1.67

h
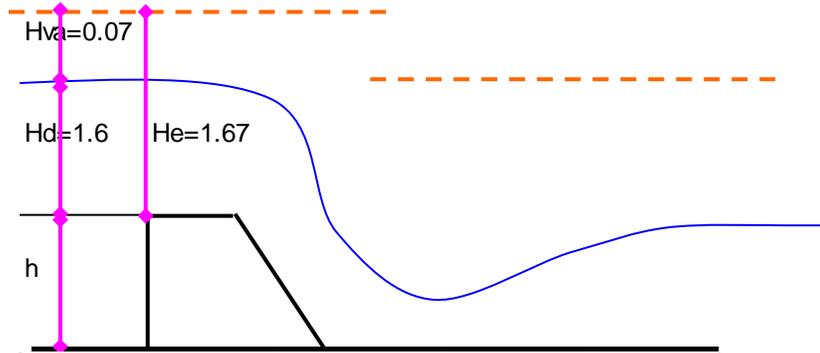
Figure 3 Flow depths over the crest

Approaching velocity

$V_a = \sqrt{2g(He-h_d)}$ and, $V_a = Q/(L*h_d)$

$Q/(L*h_d)=\sqrt{2g(He-h_d)}$

Where,

H_d = depth of water over the weir crest in m

V_a= approaching velocity in meter and, $h_d = H - V_a^2/2g$

$V_a^2/2g = H - h_d = 1.67 - h_d$

But, $V_a = ((Q/L \ (h)$

$((Q^2/ [L*h]^2 * 1/2g = 1.67-h$

The Value of h can be calculated by trial and error method as follows.

Table 3 depth of water over the weir crest

hd	Left Side	right Side	Remark
1.00	0.10	0.67	
1.20	0.09	0.47	
1.60	0.07	0.07	ok
1.50	0.07	0.17	

Therefore hd = 1.6 m

1.1.4 *Stilling Basin*

a. Energy method

i. Velocity head in the approach channel

hvo = H - h =0.07m

ii. Hydraulic Jump

1. In case of earth foundation.

River bed Elevation = 1629.4

D/S Water level =1632.15

Weir crest level =1631.00

Height of the crest, P =1.6m

1.1.5 *Determination of the bed level of the stilling basin*
By Trial and Error Method

It is necessary to finned the jump height and to adjust the bottom level of the basin so that the water surface level of the jump is a little higher than the D/S water surface level - say 20 ~ 40cm.Therefore, the floor level of the stilling basin must be found by trial and error method.

Assuming the floor level the same as the river bed.

Assumed EL.1 = EL.3

Assumed EL.1 =1629.4m

H = E0 = E1

Z = EL.0 - Assumed EL.1

Z =1.59m

E0 = H + Z=1.67+1.59

E0 = 3.27

q = Q/b

Where, q - Unit peak discharge (m3/sec/m)

Q - Peak discharge (m3/sec) =106.83 m3/sec

b - Weir Length (m) =29.00m

q =3.68m3/sec/m

From the energy rule E0 =E1 =d1 + hv1

Where, d1 -Critical depth (m)

hv1 - critical velocity head (m)

Hence, E1 = d1 + hv1 = (d1 + v12/2g)*1/2g

E1 = d1 + hv1 = (d1 +q2/2gd12)

Z = E0 +E1 Therefore, d1 +q2/2gd12

d1 +q2/2gd12 = 3.27m, using trial and error method the value of d1 are shown as below.

Table 4 Pre-conjugate depth

d1	$d1 + q^2/2gd1^2 = 3.27$	Remark
0.52	3.12	
0.52	3.08	
0.50	3.27	ok

So from this result d1 = 0.50 m

Critical velocity, V1 = q/d1 =7.37m/sec

Fraud number, Fr = V1/√ (gd1) =3.33

Post conjugate depth, d2

d2 = d1/2[(√1+8Fr2) -1] =2.12m   and d3 =2.75m

Therefore, the jump will be forced to upstream and finally be drowned out at the source becoming a submerged jump.In this case the hydraulic jump on sloping surface is the most stable one.

Therefore, no need of depression of the stilling basin and the stilling basin elevation is the same as the river bed =1629.4

The length of the stilling basin is:-

L= 5 to 7 (d_2-d_1), use the mean of 5 and 7 and it becomes and L = 6(d_2-d_1)

L=6*(2.12-0.5)=9.72

L = 4*d2 =4*2.12=8.48m   take   9m   Energy Method

  ➢ **Stilling basin length, L=9m**

The result of the calculation can be shown as below:-
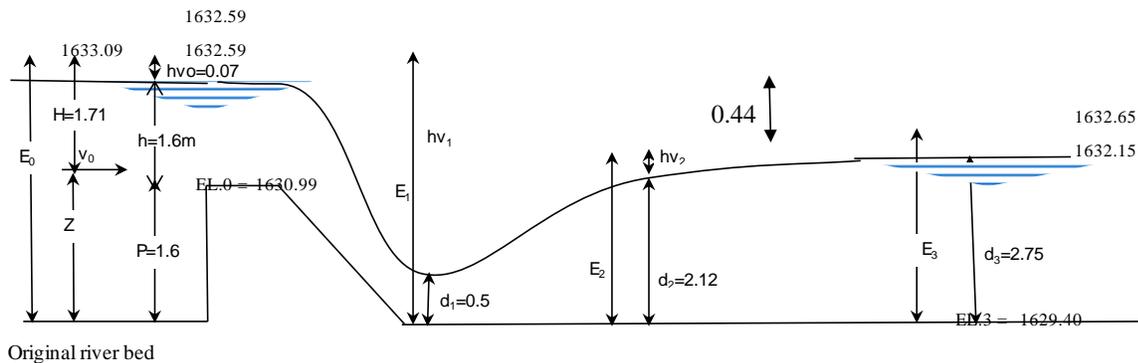
Figure 4Energy Profile of the weir

1.1.6 *Depth of Scour*

Properly designed the stilling basin dissipates the great majority of the turbulent energy in the flow. At the outflow from the basin there remains a certain proportion of energy in the flow that scours the d/s of the basin. The scour holes so formed may progress towards the structure and results in structural failure. Such failures can be prevented by providing piles or cut- off at u/s

and d/s ends of the impervious floor, much below the calculated scour level. The depth of scour can be calculated using Lacey's equation.

Hydraulic mean depth, $R = 1.35*(q^2/f)^{0.333}$ Where,

R - Hydraulic mean depth

q - Discharge per meter length

f - Lacey's silt factor, for coarse sand f = 1.5

Hydraulic mean depth, R =2.81m

Let us provide u/s cut-off at depth of 1.25R (i.e.3.51m from the top of u/s water level.

Bottom level of u/s cut-off = u/s HFL - 1.25R = 1629.09m

Let us, therefore, provide the u/s cut-off up to a bottom level of 1629.4m, i.e. 0.31m take 2.5m u/s cut-off from geological report

Let us provide d/s cut-off at depth of 1.50R below the d/s water level (which is 1632.88m with retrogression).

Hence, the R.L. of bottom of d/s cut-off = 1628.66m

Let us, therefore, provide the d/s cut-off up to a bottom level of 1629.4 m, i.e. 0.74m take 1.3m d/s cut-off -for safe exit gradient

1.1.7 *Basic Section of the weir Body*

The proposed gravity weir will be designed to resist, with sufficient factor of safety, the three tendencies to destruction; overturning, sliding and overstressing. The basic section of the weir can be determined using Bligh's method as follows.

Top width, $B = He/\sqrt{(r-1)}$

Top width, B = 1.53m take1.5m, after structural analysis

Bottom width, $L = He + p/\sqrt{(r-1)}$

Bottom width, L =2.99m take 3.20m, after structural analysis (stability)
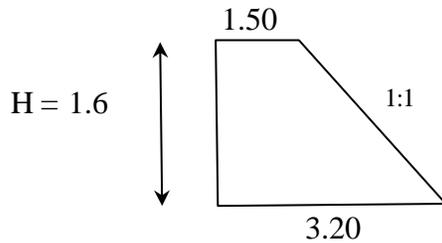
Figure 5 Section of Weir Body

Therefore, the downstream slope of the weir section is 1 horizontal to 1 vertical

1.1.8 *Length of the apron floor*

Maximum Head on the structures

. For Dynamic Case

H max = Headwater - TWL, ☞H max =0.44m

. For Static Case

H max = Weir Crest - River Bed Level

H max =1.6m

Therefore, it is necessary select the maximum head differential which usually occurs at the time of flood often called as dynamic case. In this case the maximum differential H max is 0.44 m for dynamic case and 1.6 m for static case. Therefore, Lane's method and H max = 1.6 m will be taken to check whether the length of the apron is sufficient.

$L \geq CH$, Foundation material = coarse gravel Pan Lane Creep Ratio, C =3.5

CH =   5.55m

Weighted creep length

$L c = \sum L v + 1/3 * \sum L h \geq CH$

$L c = \sum L v + 1/3 * \sum L h = 8.73m$

Weighted Creep Ratio = L/H max

Weighted Creep Ratio = 5.51

  ➢ Safe exit gradient for course sand (0.17-0.20)

Total Floor length and Exit Gradient

$\square$ = b/d =   9.38

$\square$ = (1 + (1+$\square$^2)$^{1/2}$) ÷ 2 =   5.22

Ge = Hmax/d * 1/$\square$*($\square$)$^{1/2}$ =   0.17

According to Lane recommended ratios, this structure would be safe from piping on either coarse Gravel.

Thickness of the Downstream Apron Floor

Point	Weighted creep Length			Hmax[1-La/Lc]	TWL-Wla	Safety Factor /(2.3-1)	Remark
	H	V	L				
Dynamic Case Hmax =		0.44	m				
A	5.20	7.50	9.23	0.11	0.26	0.36	
B	8.20	7.50	10.23	0.07	0.00	0.07	
C	11.70	7.50	11.40	0.03	0.00	0.03	
Static Case Hmax =		1.6	m				
A	5.20	7.50	9.23	0.38	0.50	0.882	
B	8.20	7.50	10.23	0.25	0.50	0.752	
C	11.70	7.50	11.40	0.10	0.50	0.5999	

However for point A the thickness should be reduced for Dynamic case by:

$t_A = \{H_{max}*[L_A/L_C]+0.5*(TWL-WL_A)\}*Safety\ Factor/(\square_m-1)$

Check the thickness of each points

Point A =	1.60	>	1.20	ok	Dynamic Case
Point B =	0.80	>	0.75	ok	Static Case
Point C =	0.60	>	0.59	ok	Static Case
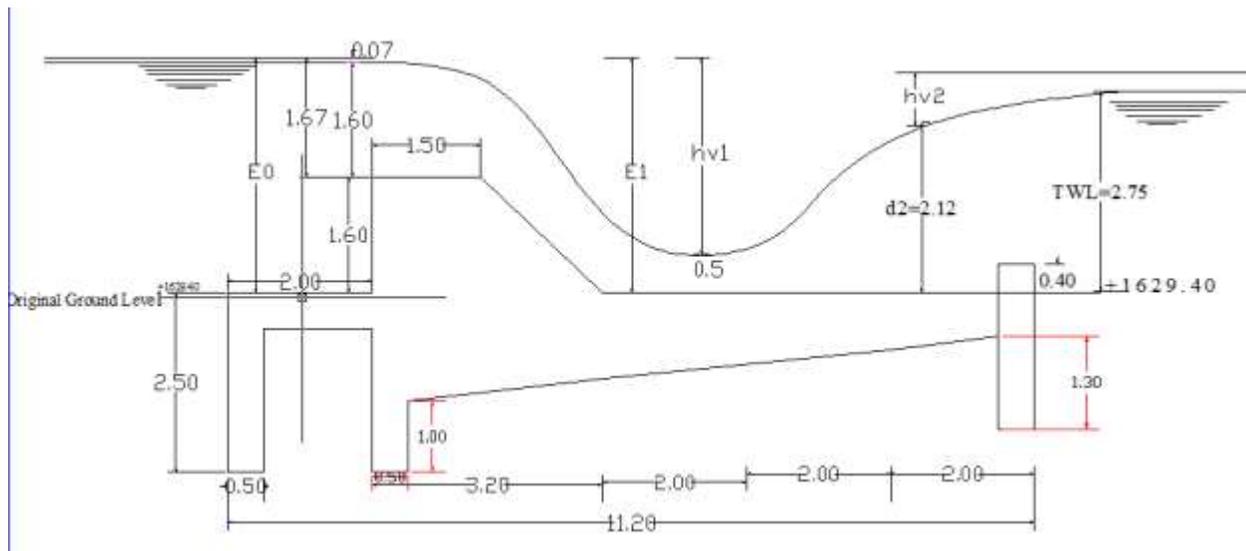
Figure 1 Thickness of the Downstream Apron Floor

1.1.9 *Free Board*

Sufficient freeboard should be provided for u/s and d/s wing walls in order to protect the walls and embankments from being overtopping by surges, splash and spray, and wave action setup by the turbulence of hydraulic jump and not to allow high flood water to bypass the diversion weir. The following empirical formulae are used to determine the freeboard.

D/S Freeboard = 0.1(V1 + d2)

D/S Freeboard = 0.49m

However adopt 0.50   m for u/s and d/s wing walls.

Top elevation of the u/s retaining walls = TWL+FB = 1633.1m

Top elevation of the d/s retaining walls = TWL+FB = 1632.65m

1.1.10 *Design of scouring sluice*

Helps to allow the removal of silt deposited near the intake pipe. This is designed to ensure sufficient scouring capacity least to dispose off about 10% of the peak flood (106.83m3/s)) i.e. 10.68 m3/s, this value is at least greater than 2 times the intake capacity.

Using a broad crested weir formula, $Q = C_d * L * H_d^{3/2}$ Where,

Q = discharge through the under sluice = 10.68 m3/s

C_d, coefficient of discharge = 1.7

Length of under sluice section,

H_d = water depth above the crest of the under sluice during high flood = 3.0m

$(h + H_d)$ where, h is weir height and hd is design head

Hence $L = Q/C_d (H_d)^{3/2} = 1.21$ m

Use L = 1.0   m (practical length has to be used)

Under sluice gate size of B*L=1.4m*1m

The gate of a sluice can be a thick say 6mm sheet metal or other similar material.
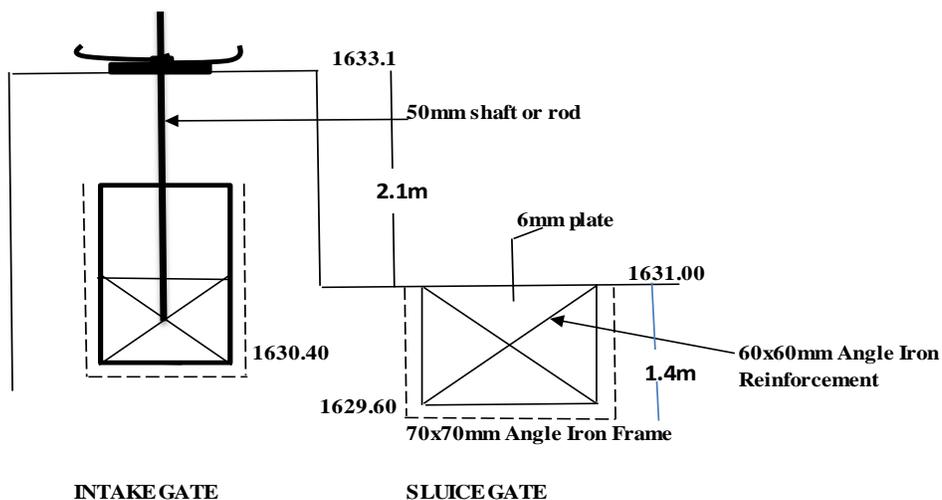
Figure 2 Intake and sluice gate

1.1.11 *Design of Intake*

Intake design was carried out using Orifice formula given by

$Q = CA \sqrt{(2gh)}$

Where

Q is the discharge through the intakes=0.19m^3/s

C –discharge coefficient =0.6 for rectangular cross-section

A- Cross-sectional area of the gate opening

h- Water head or difference between upstream and downstream water level

h=u/s HFL-FSL

    =1631-1630.47=0.53m

0.19=0.6*B*H*$\sqrt{(2*9.81*1.27)}$, Take H=0.25

0.19=0.25 B

B=0.35m

So the intake size to be provided will have dimension B*H=0.35m*0.25m

1.1.12 *Stability Analysis for the weir body (For Dynamic and Static Cases)*

Dynamic Case
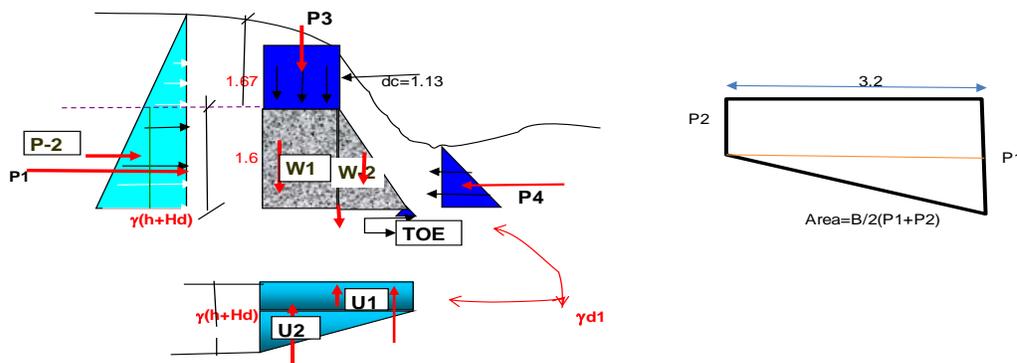
Figure 3 Stability analyses for the weir body

Where,

d_3, Tail Water Depth = 2.75 m

B, Weir Base width = 3.20   m

H_d, Head over the crest   = 1.6 m

h, Weir Height = 1.6 m

b, Weir Top width = 1.50 m

P_d - Water pressure at downstream side of the weir toe

$P_{u1 \text{ and } 2}$ - Water pressure at upstream side of the weir toe

P_u - Up lift pressure

$P_{1 \text{ and } 2}$ - Contact pressure of Foundation

γ_C - Specific weight of concrete =   2.30 t/m3

γ_m - Specific weight of masonry = 2.00 t/m3

γ_w - Specific weight of Water = 1.00 t/m3

Table 5 weir body Stability Analysis for dynamic case

1. Weight of the Masonary (weir body) $\gamma = W/V => W = \gamma * A$

γM	Vertical F	Area (m^2)	Force (KN/m)	Arm (m)
2.20	Wm1	2.40	5.28	2.45
	Wm2	1.36	2.99	1.13
			8.27	

2. Water pressure $\gamma = W/V => W = \gamma * A$

γW		Force	arm
1	P1	2.7	1.09
	P2	1.3	0.53
	P3	1.1	2.45
	P4	0.1	0.17

3.Uplift pressure pressure   F=P*A

	Pressure	Force/m	arm(m)
U1	1.60	1.60	1.6
U2	1.39	1.39	2.13
	2.98		

Weight (Force)	Vertical Force (KN/m)	Horizontal Force (KN/m)	Arm length (m)	Moment about toe (+)KNm/m	Moment about toe (-)KNm/m	Codition of the moment
P_{w1}		-2.68	1.09		-2.92	Over turning
P_{w2}		-1.28	0.53		-0.68	Over turning
P_{w3}	1.13		2.45	2.77		Resisting
P_{w4}		0.12	0.17	0.02		Resisting
U_1	-1.60		1.60		-2.55	Over turning
U_2	-1.39		2.13		-2.96	Over turning
W_{m1}	5.28		2.45	12.94		Resisting
W_{m2}	2.99		1.13	3.39		Resisting
ΣV	6.42					
ΣH		-3.83				
ΣEM^+				19.12		
ΣM^-					-9.12	
ΣM					9.99	Resisting

Summary

Stability against over turning

SM+/SM- >= 1.5,

2.1 >=1.5   ok

Stability against sliding

SH/SV< 0.75

0.6 <   0.75   ok

Stability against over stress

X= SM/SV

0.89< 2/3*B = 1.6m   ok

B/2 = 1.6m

B/6 = 0.53m

e = (B/2) –x

e = 0.043

Since e < a, 0.043<0.53m

Stability against compression

α C toe= (SV/B)/ (1+6e/B) = (6.98/3.2)* [(1+6(0)/3.2)] = 1.86 t/m2

Since the allowable compressive strength of masonry extends up to 100 t/m2. Therefore, the calculated value of compressive strength that is 1.86t/m2 is safe.

Stability against Buoyancy

SWm/SU > 1.0, that is 8.27/2.98=2.77 > 1.0 is ok

-The Contact pressure on foundation

P1= (SV)/ {(1+6e)/B)} =6.93 t/m

P2= (SV)/{(1-6e)/B)} = 5.9 t/m

-The foundation reaction

R1= (B/2)*(P1-P2) = 1.65t/m

R2= (B)*(P2) =18.89t/m

Static Case

Table 6 Weir body Stability Analysis for static case

Weight (Force)	Vertical Force (KN/m)	Horizontal Force (KN/m)	Arm length (m)	Moment about toe (+)KNm/m	Moment about toe (-)KNm/m	Codition of the moment
P_{w1}		-2.68	0.53		-1.43	Over turning
P_{w2}		0.00	0.00		0.00	Over turning
U_1	-1.60		4.80		-7.66	Over turning
W_{m1}	5.28		2.45	12.94		Resisting
W_{m2}	2.99		1.13	3.39		Resisting
ΣV	6.68					
ΣH		-2.68				
ΣEM^+				16.33		
ΣM^-					-9.09	
ΣM					7.23	Resisting

Summary

Stability against over turning

SM+/SM- >= 1.5,

1.8 >= 1.5 ok

Stability against sliding

SH/SV< 0.75

0.4 < 0.75 ok

Stability against over stress

X= SM/SV = 1.1 < 2/3*B = 2.13m ok

B/2= 1.6m

B/6(a) = 0.53m

e= (B/2) –x = 0.516

Since e < a 0.516<0.53m

Stability against compression

α C toe= (SV/B)/ (1+6e/B) =1.06t/m2

Since the allowable compressive strength of masonry extends up to 100 t/m2. Therefore, the calculated value of compressive strength that is 1.06t/m2 is safe.

-The Contact pressure on foundation

P1= (SV)/ {1+6e/B} =13.14t/m

P2= (SV)/ {1-6e/B} =0.21t/m

-The foundation reaction

R1= (B/2)*(P1-P2) =20.68t/m

R2= (B)*(P2) =0.68t/m

1.1.13 *Design of Retaining Walls*

Upstream Wing Wall (both sides)

The height of the top level of the wing wall is fixed considering the weir height, head over the weir and some free board.

The base of the abutment shall be kept 1.00 m below the bed level of the river

The thickness of the foundation concrete = 0.50 m

The EL of junction of concrete and masonry 1629.40m

The EL of base of concrete = 1628.90m

Top of the abutment = Top of the wearing coat = 1633.10m

Height of the abutment up to the junction = 3.7m (H)

Assuming the base width = 0.75*H =2.775 say 2.8m

Assuming f = 30

C = 0

Density of stone masonry = 2000kg/m3

Density of saturated earth = 1800kg/m3

Testing At Junction
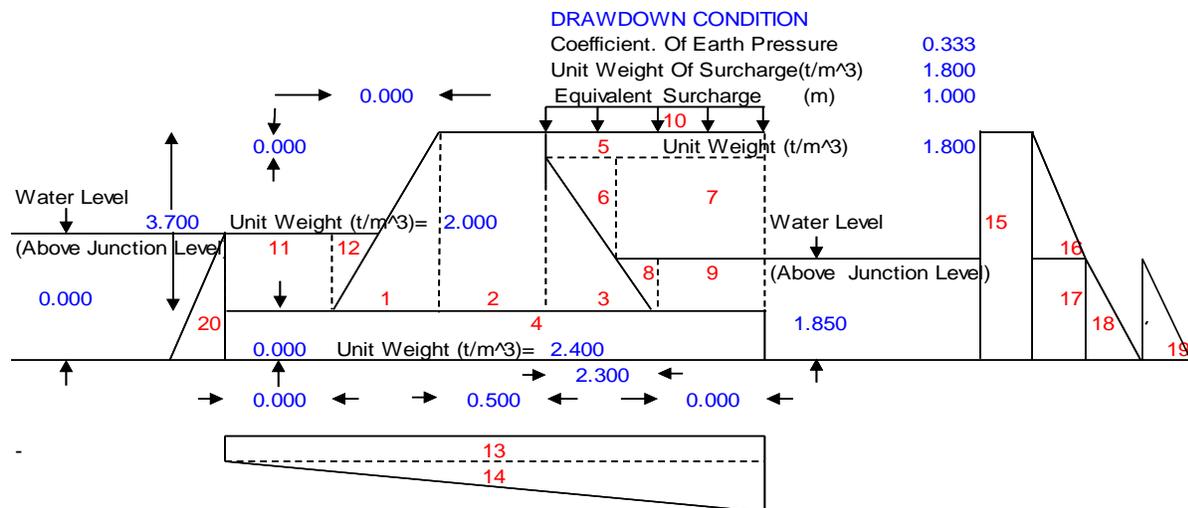
Figure 4 Stability analyses for U/S Retaining walls at junction

Table 7 U/S Retaining Wall Stability Analysis testing at junction

Item No.	Vertical (t)	Horizontal (t)	Lever Arm (m)	Mv (t-m)	Mh (t-m)	Remarks
1	2	3	4	5	6	7
1	0.000		0.000	0.000		Front face of wall is vertical
2	3.700		0.250	0.925		
3	8.510		1.267	10.779		Back face is inclined
4	0.000		1.400	0.000		Testing is done at Junction Level
5	0.000		1.650	0.000		Back inclination starts just at top
6	1.915		1.267	2.425		Back face is inclined
7	3.830		2.225	8.521		
8	1.915		2.417	4.627		Back face is inclined
9	0.000		2.800	0.000		
10	4.140		1.650	6.831		Equivalent surcharge effect
11	0.000		0.000	0.000		
12	0.000		0.000	0.000		
13	0.000		1.400	0.000		15% Uplift is Considered
14	-0.389		1.867	-0.725		15% Uplift is Considered
15		2.220	1.850		4.107	Equivalent surcharge effect
16		1.027	2.467		2.533	
17		2.054	0.925		1.899	
18		0.456	0.617		0.281	
19		1.711	0.617		1.055	
20		0.000	0.000		0.000	
SUM	23.621	7.468		33.383	9.876	
X-bar(m)=		0.995		Data For Testing Interface		Results
e(m)=		0.405		(MASONARY-FOUNDATION)		(Based on Option-1 or Option-2)
f1(t/m^2)=		15.753		Option-1	C(t/m^2)= 10.500	F.O.S.(sliding)= 7.329
f2(t/m^2)=		1.119			Phi(deg)= 47.000	
F.O.S.(overturning)=		3.380		Option-2	"f"= 0.700	F.O.S.(sliding)= 2.214

f = Foundation Pressure on both side is Positive

Max Pressure = 15.75 safe

Min Pressure = 1.12   safe

F.O.S against overturning = 3.38 safe

F.O.S against sliding = 2.21   safe

Testing at Foundation level

DRAWDOWN CONDITION
Coefficient. Of Earth Pressure   0.333333
Unit Weight Of Surcharge(t/m^3)   1.800
Equivalent Surcharge   (m)   1.000

Figure 5 Stability analyses for U/S Retaining walls at Foundation level

Table 8 U/S Retaining Wall Stability Analysis testing at foundation level

Item No.	Vertical (t)	Horizontal (t)	Lever Arm (m)	Mv (t-m)	Mh (t-m)	Remarks
1	2	3	4	5	6	7
1	0.000		0.500	0.000		Front face of wall is vertical
2	3.700		0.750	2.775		
3	8.510		1.767	15.034		Back face is inclined
4	4.560		1.900	8.664		Testing is done at Foundation Level
5	0.000		2.400	0.000		Back inclination starts just at top
6	1.915		1.767	3.383		Back face is inclined
7	5.495		2.975	16.346		
8	1.915		2.917	5.585		Back face is inclined
9	1.665		3.550	5.911		
10	5.040		2.400	12.096		Equivalent surcharge effect
11	0.000		0.000	0.000		
12	0.000		0.000	0.000		
13	0.000		1.900	0.000		Uplift Effect
14	-4.465		2.533	-11.311		Uplift Effect
15		2.520	2.100		5.292	Equivalent surcharge effect
16		1.027	2.967		3.046	
17		2.609	1.175		3.065	
18		0.736	0.783		0.577	
19		2.761	0.783		2.163	
20		0.000	0.000		0.000	
SUM	28.334	9.653		58.482	14.143	
	X-bar(m)=	1.565		Data For Testing Interface		Results
	e(m)=	0.335		(FOUNDATION-SOIL)		(Based on Option-1 or Option-2)
	f1(t/m^2)=	11.402		Option-1	C(t/m^2)=	10.500 O.S.(sliding)= 7.281
	f2(t/m^2)=	3.511			Phi(deg)=	47.000
F.O.S.(overturning)=	4.135			Option-2	"f"=	0.700 F.O.S.(sliding)= 2.055

f = Foundation Pressure on both side is Positive

Max Pressure = 11.402   safe

Min Pressure = 3.511   safe

F.O.S against overturning = 4.135   safe

F.O.S against sliding = 2.055   safe

Downstream Wing Wall (both sides)

The base of the abutment shall be kept 1.73 m below the bed level of the river

The thickness of the foundation concrete = 0.50 m

The EL of junction of concrete and masonry = 1628.40m

The EL of base of concrete = 1627.90m

Top of the abutment = Top of the wearing coat = 1632.15masl

Height of the abutment up to the junction = 3.75 (H)

Assuming the base width = 0.75*H = 2.8125 say 2.9m

Assuming f = 30 and C = 0

Density of stone masonry = 2000   kg/m3

Density of saturated earth = 1800   kg/m3

Testing At Junction
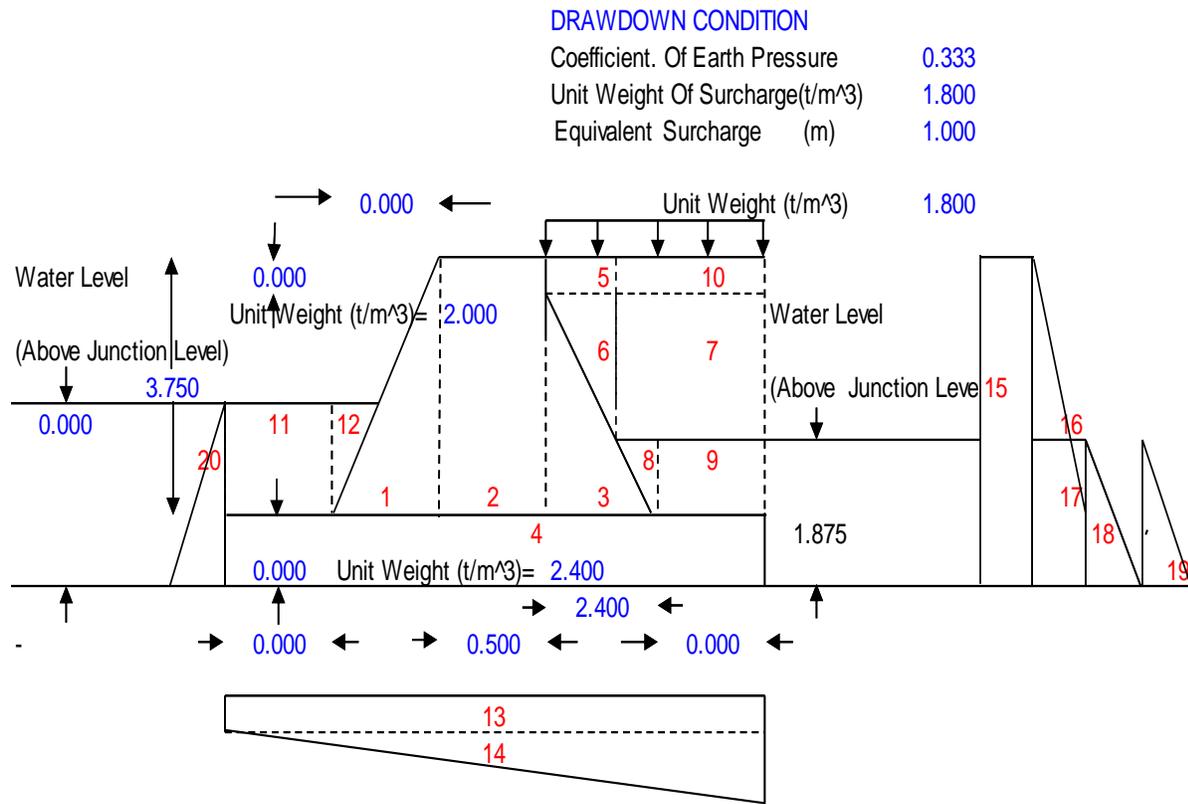
Figure 11 Stability analyses for D/S Retaining walls at Junction

Table 9 D/S Retaining Wall Stability Analysis testing at junction

Item No.	Vertical (t)	Horizontal (t)	Lever Arm (m)	Mv (t-m)	Mh (t-m)	Remarks
1	2	3	4	5	6	7
1	0.000		0.000	0.000		Front face of wall is vertical
2	3.750		0.250	0.938		
3	9.000		1.300	11.700		Back face is inclined
4	0.000		1.450	0.000		Testing is done at Junction Level
5	0.000		1.700	0.000		Back inclination starts just at top
6	2.025		1.300	2.633		Back face is inclined
7	4.050		2.300	9.315		
8	2.025		2.500	5.063		Back face is inclined
9	0.000		2.900	0.000		
10	4.320		1.700	7.344		Equivalent surcharge effect
11	0.000		0.000	0.000		
12	0.000		0.000	0.000		
13	0.000		1.450	0.000		15% Uplift is Considered
14	-0.408		1.933	-0.788		15% Uplift is Considered
15		2.250	1.875		4.219	Equivalent surcharge effect
16		1.055	2.500		2.637	
17		2.109	0.938		1.978	
18		0.469	0.625		0.293	
19		1.758	0.625		1.099	
20		0.000	0.000		0.000	
SUM	24.762	7.641		36.203	10.225	

X-bar(m)=	1.049	Data For Testing Interface		Results
e(m)=	0.401	(MASONARY-FOUNDATION)		(Based on Option-1 or Option-2)
f1(t/m^2)=	15.621	Option-1 C(t/m^2)=	10.500	O.S.(sliding)= 7.461
f2(t/m^2)=	1.457	Phi(deg)=	47.000	
F.O.S.(overturning)=	3.541	Option-2 "f"=	0.700	O.S.(sliding)= 2.269

f = Foundation Pressure on both side is Positive

Max Pressure = 15.621   safe

Min Pressure = 1.457 safe

F.O.S against overturning = 3.541   safe

F.O.S against sliding = 2.269   safe
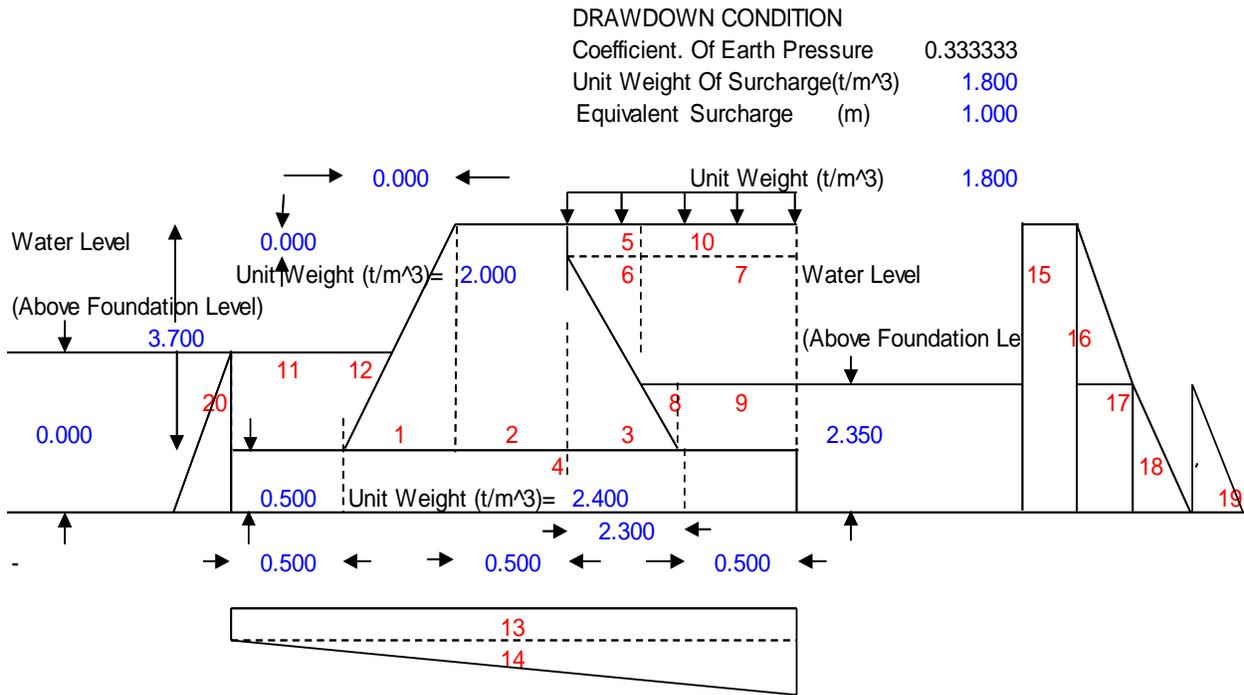
Testing at Foundation level
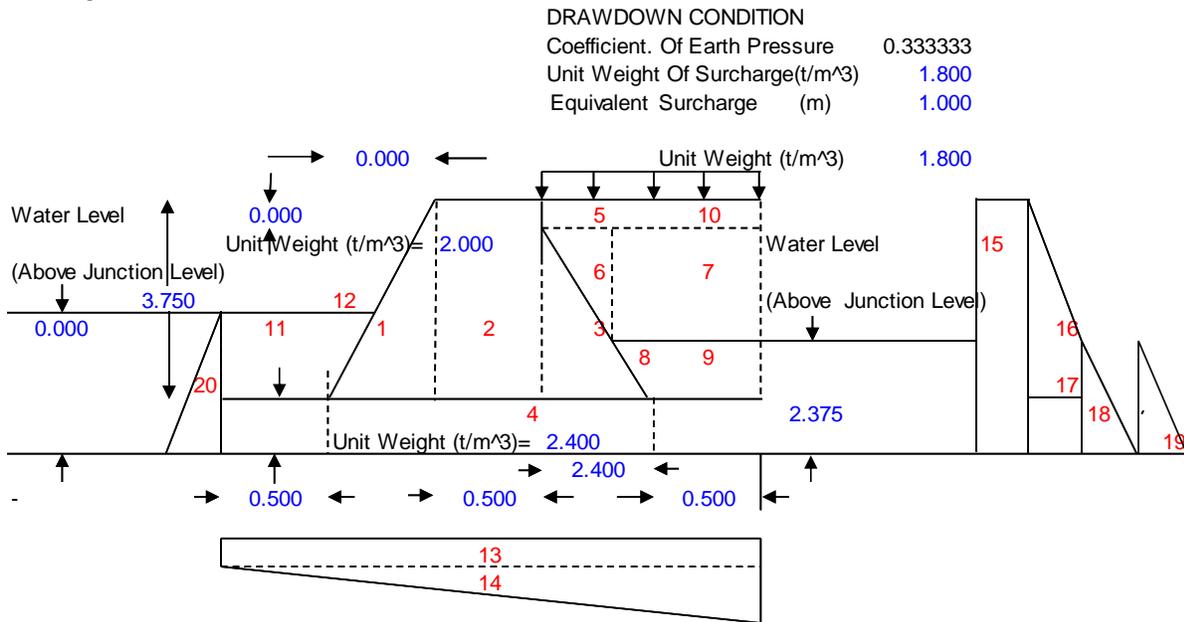
Figure 62 Stability analyses for D/S Retaining walls at Foundation level

Table 10 D/S Retaining Wall Stability Analysis testing at foundation level

Item No.	Vertical (t)	Horizontal (t)	Lever Arm (m)	Mv (t-m)	Mh (t-m)	Remarks
1	2	3	4	5	6	7
1	0.000		0.500	0.000		Front face of wall is vertical
2	3.750		0.750	2.813		
3	9.000		1.800	16.200		Back face is inclined
4	0.000		1.950	0.000		Testing is done at Junction Level
5	0.000		2.450	0.000		Back inclination starts just at top
6	1.089		1.587	1.728		Back face is inclined
7	5.000		2.890	14.449		
8	3.249		2.893	9.400		Back face is inclined
9	2.138		3.650	7.802		
10	5.220		2.450	12.789		Equivalent surcharge effect
11	0.000		0.000	0.000		
12	0.000		0.000	0.000		
13	0.000		1.950	0.000		15% Uplift is Considered
14	-0.517		2.600	-1.343		15% Uplift is Considered
15		2.250	1.875		4.219	Equivalent surcharge effect
16		0.567	2.833		1.607	
17		1.959	1.188		2.327	
18		0.752	0.792		0.595	
19		2.820	0.792		2.233	
20		0.000	0.000		0.000	
SUM	28.928	8.349		63.837	10.981	

X-bar(m)=	1.827	Data For Testing Interface	Results
e(m)=	0.123	(MASONARY-FOUNDATION)	(Based on Option-1 or Option-2)
f1(t/m^2)=	8.820	Option-1 C(t/m^2)= 10.500	O.S.(sliding)= 8.620
f2(t/m^2)=	6.016	Phi(deg)= 47.000	
F.O.S.(overturning)=	5.814	Option-2 "f"= 0.700	O.S.(sliding)= 2.425

f = Foundation Pressure on both side is Positive

Max Pressure = 8.82   safe

Min Pressure = 6.016   safe

F.O.S against overturning = 5.814   safe

F.O.S against sliding = 2.425   safe

1.1.14 *Left & Right Side River Bank Protection embankments*

Due to provision of weir across the river, water level will rise on the upstream side of the structure. The structure is designed and provided in restricted width of river. Raised water level may cause out flanking of riverbanks near structure if the banks are not above afflux level but the river bank is much above the afflux level so no need of river bank protection work.
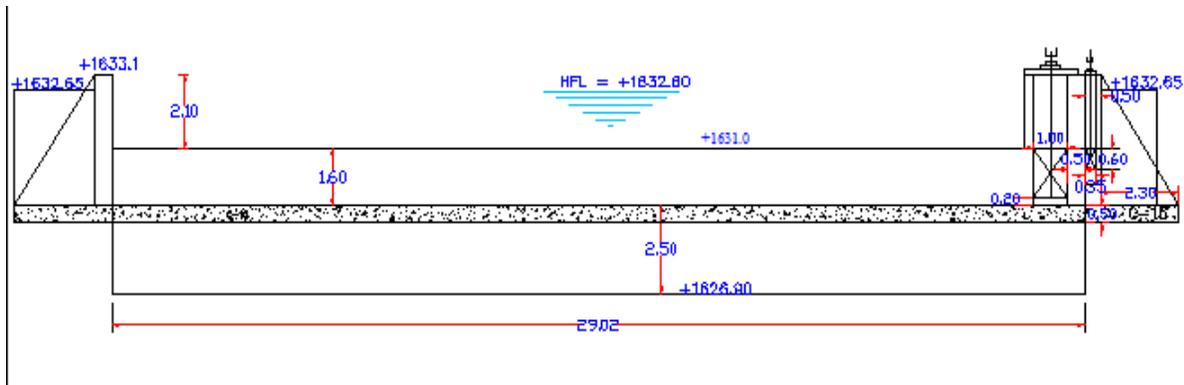
Figure 13 Cross Section the river along the Weir Axis.
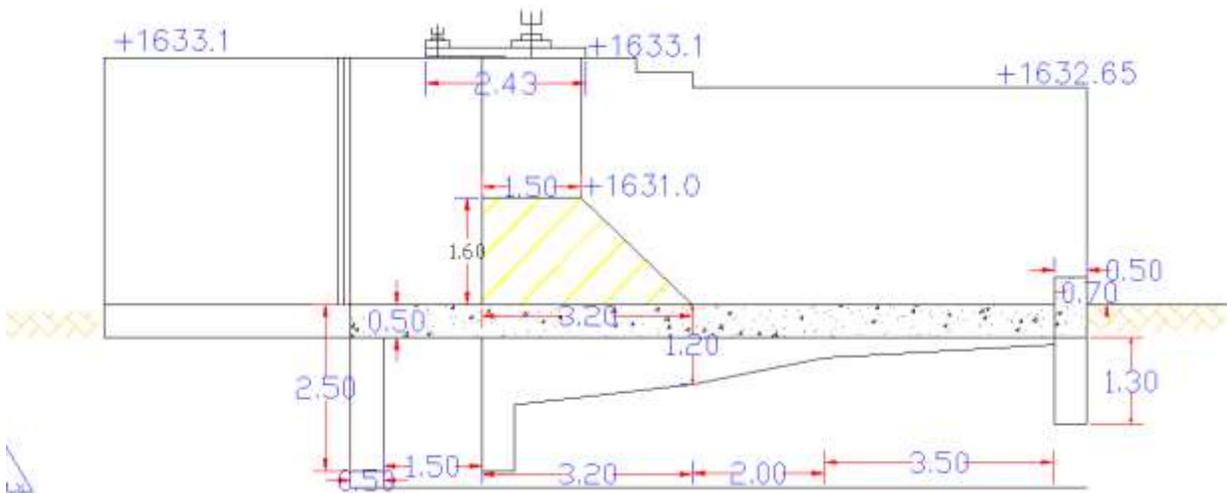
Figure 7 Section view of the weir at left side of the river bank
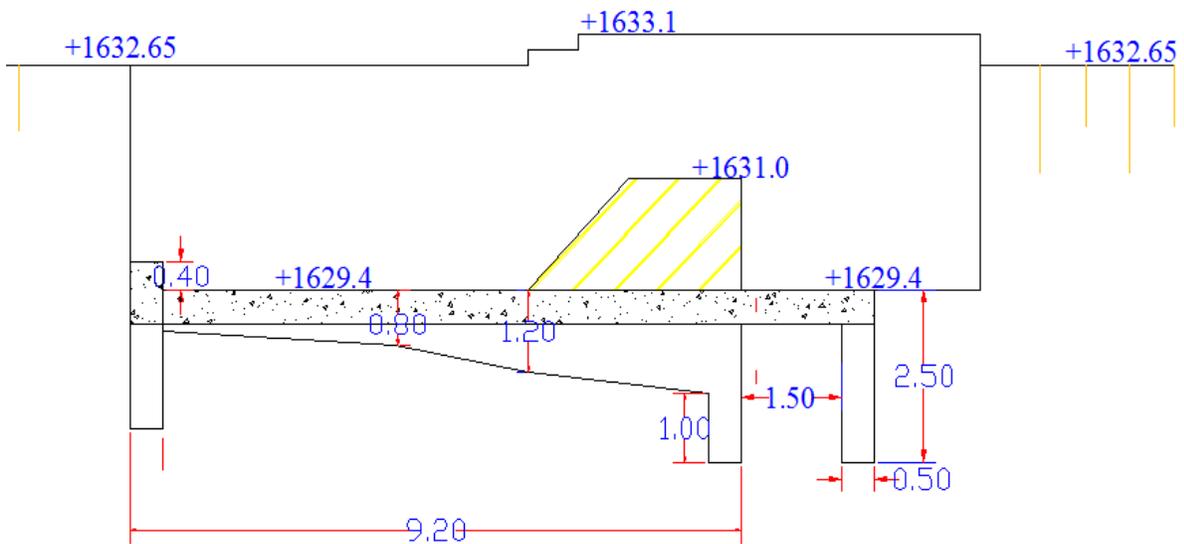
Figure 15 Section view of the weir at Right side of the river bank
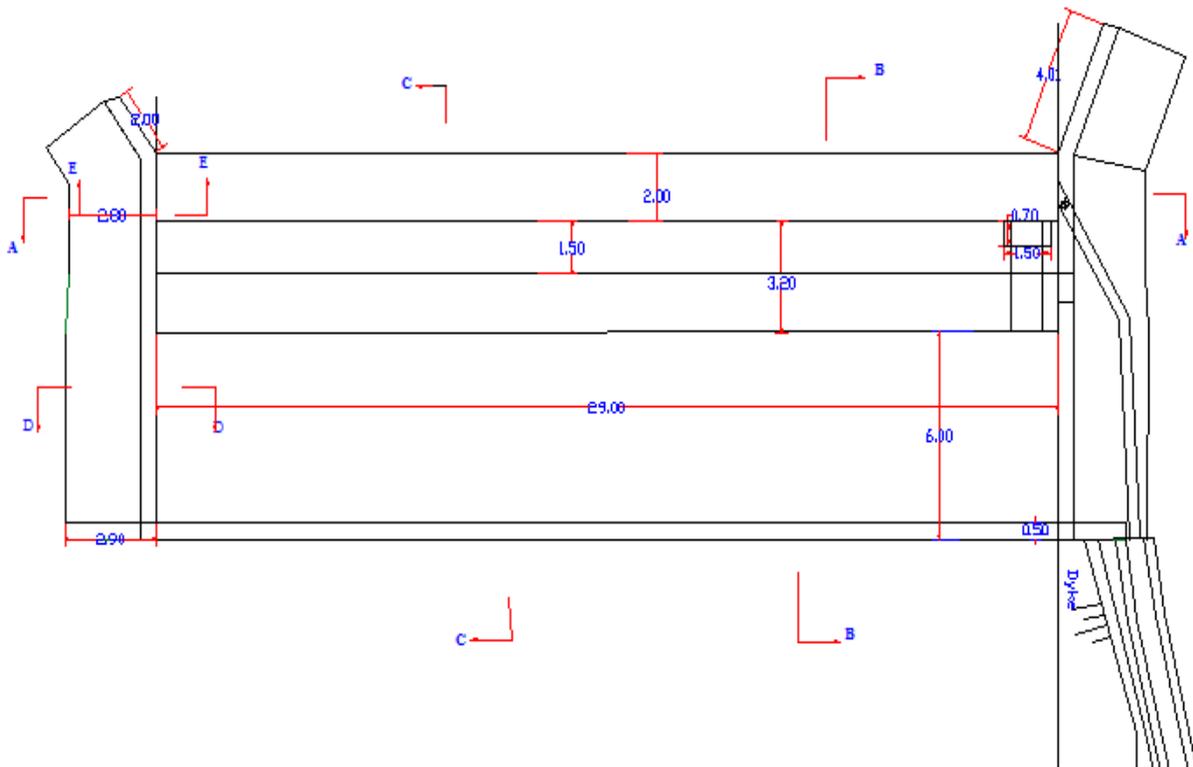
Figure 16 Head Work Plan

2. IRRIGATION DISTRIBUTION SYSTEM DESIGN

2.1   CANAL SYSTEM

2.1.1   Description of Water Distribution System

The diverted water from the Kolu River is to be distributed to the farm through a network of canals by gravity flow. The physical purpose of irrigation is to satisfy the demands of crop water requirement by increasing the moisture content of the soil in the root zone of the crop. For this canal distribution system should be planned to convey the required amount of water from the outlet as described above. In the case of the 'Laga kolu' irrigation scheme since gravity led irrigation practice is designed and furrow irrigation method is selected. Each command area with small channels (furrow) water reaches over the soil surface.

2.1.2   Canal Alignment (Canal Layout)

The Laga Kolu site has sufficient and suitable irrigable area on Left sides of the river (see Fig. 17 the layout of the command area). The command area of this Irrigation project is 105.50 ha and stretched along the left side of the main river. The proposed canal system of the scheme consists of on main canal (MC-6169.62m long), five secondary canals (SC1- 116.5m, SC2-88.77m, SC3-159.25m, SC4-155.63m, SC5-309.55 long and 29 TC-9601m long). SCs branch from the main canal at chainage 2+033,2+856,4+586,5+182 and 6+158 with Grid Coordinates of (628446.18,931822.97),(628733.39 ,931247.52),(629585.92,929924.02),(629877.50 929466.75)and (630319.77,928950.41) to irrigate 9.18ha, 6.15ha,16.27ha,19.55ha and 39.07ha of land respectively.

From MC, SC and TC more than 63 turnouts and 34 offtakes are proposed. Along the whole canal system there are 6 major drain crossings which require two Drainage Culvert (super passage) and 4 Flume designed. As the main canal system passes through villages and social infrastructures, more than 25 footpath crossings are required.

Canal layout is marked on the field and profile is taken for each rout of main ,secondary and secondary canal. The following points are considered while preparing the canal layout:

❖ The layout is done based on block areas to be irrigated by taking drains and ridges into consideration.

❖ Deep cutting and high banking is tried to be avoided

❖ The alignment of Main canal follows almost on the periphery of the command area as a contour canal.

2.2 Design of Canals

Canal design is used to decide the parameters like the depth of flow, bed width, flow velocity, etc. These depend on design discharge, bed slope, and coefficient of roughness and side slope of canals if the cross section is trapezoidal and Rectangular. Design discharge is the maximum discharge that the canal is required to carry in any year .It depends on irrigation intensity, cropping pattern, size of command area, duty and time factor.

Design Discharge Determination
Duty, q = 1.8 lit/sec/ha
Command Area, A = 105.5 ha
Design Discharge, Q d = 0.19m3/sec
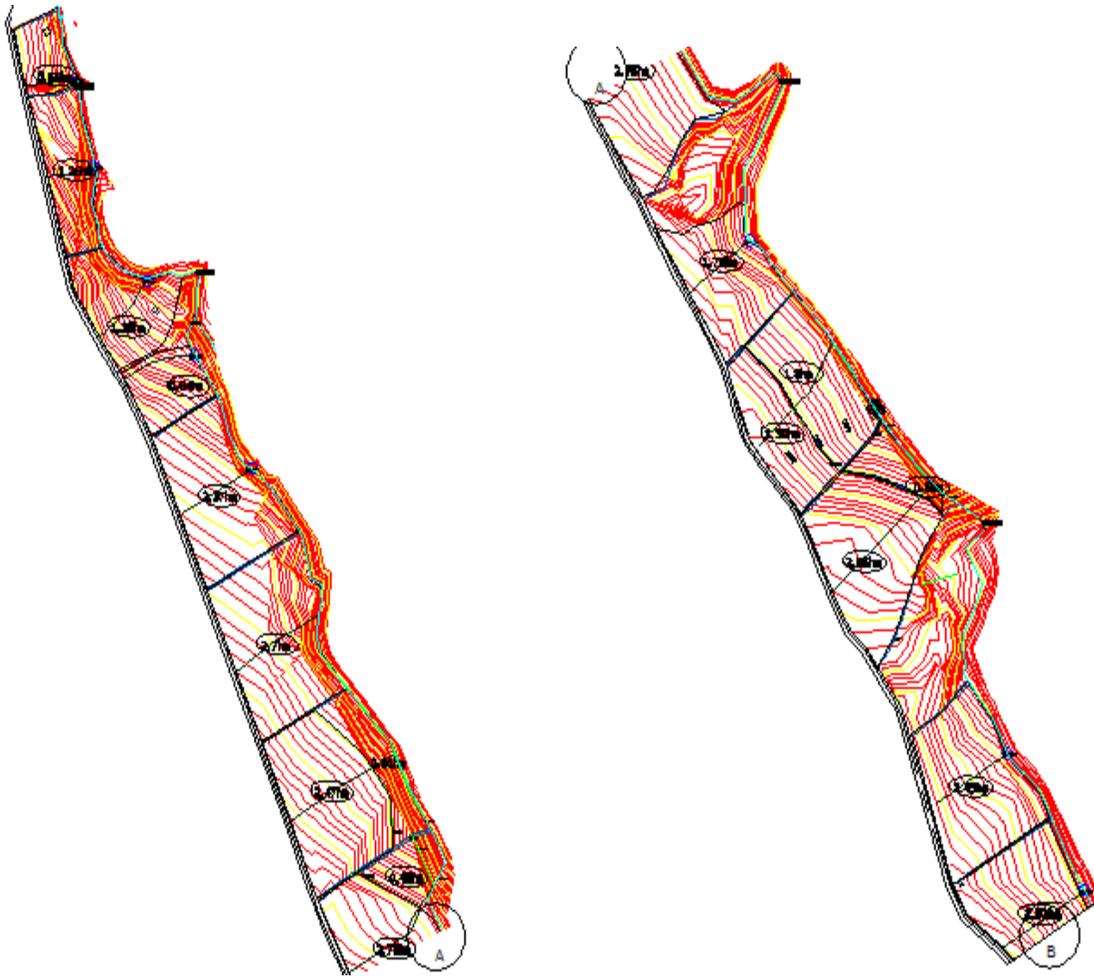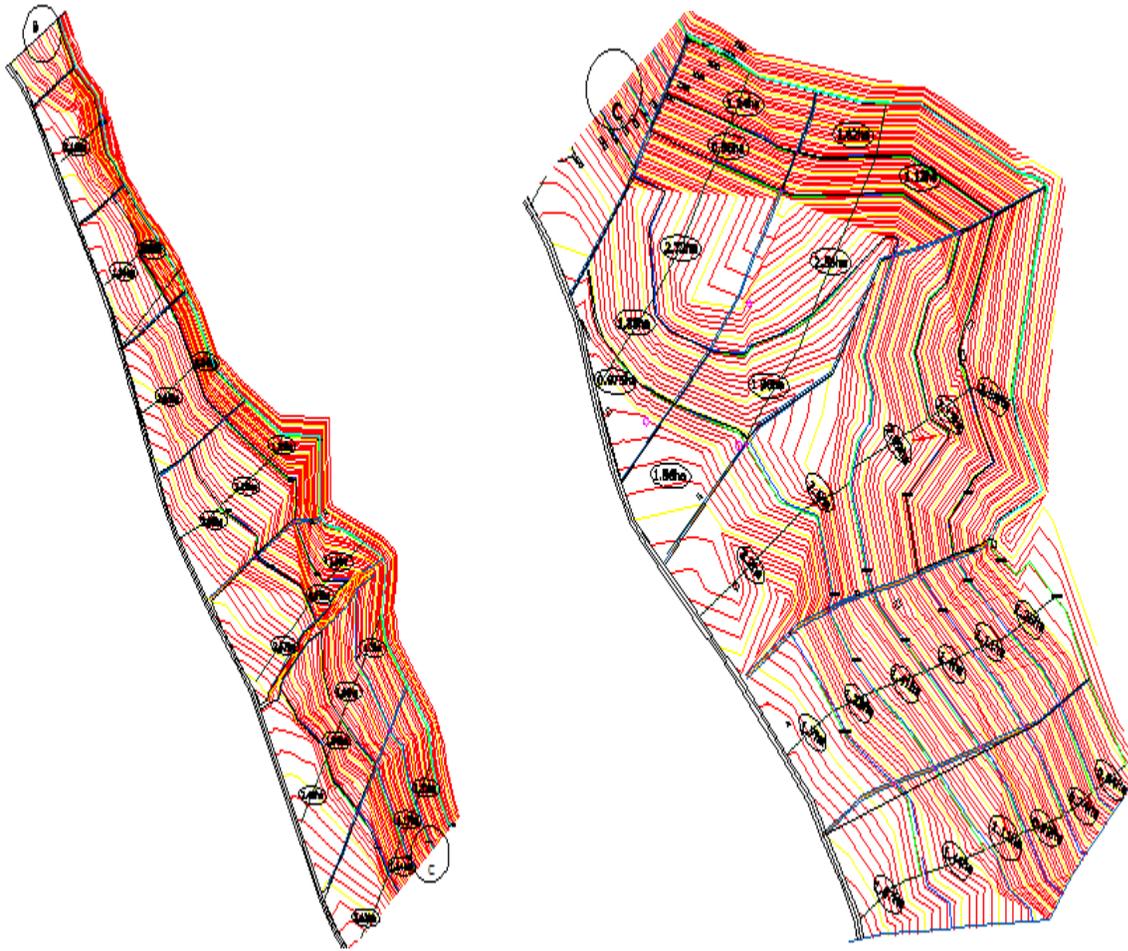
Figure 17 General Layout of Laga Kolu irrigation project

    2.2.1   Main Canal

Design Discharge Determination

Duty, q = 1.80 lit/sec/ha

Command Area, A = 105.50ha

Design Discharge, Qd = 0.19m3/sec
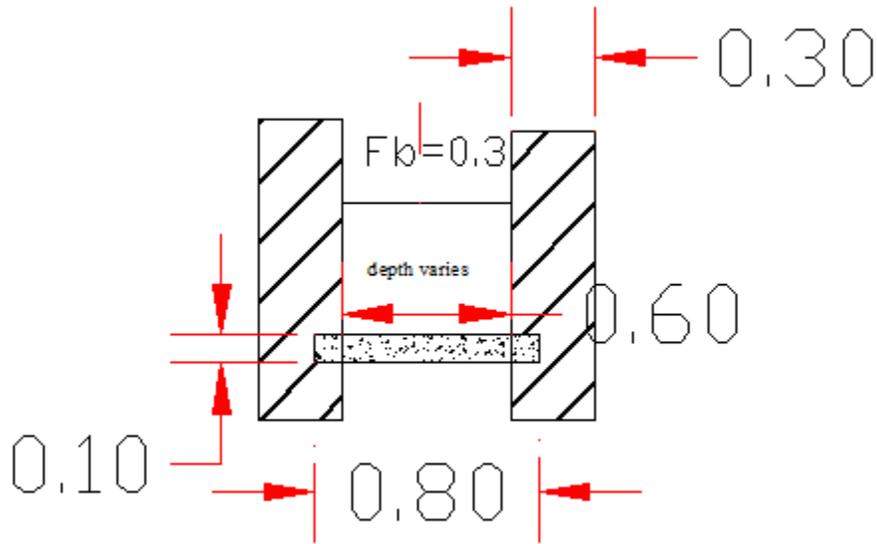
Figure 18 Sections of Canals

Table 11 Design Parameters of Main Canal

Canal Design	For Rectangular	
Canal bed width, b	0.60	m
Water Depth, d	0.47	m
Canal Side Slope, m		m/m
Water area, A	0.28	m2
Wetted Perimeters, P	1.54	m
Hydraulic Radius, R	0.18	m
Roughness Coefficient, n	0.015	
Canal bed slope, I	0.0010	m/m
Flow Velocity, V	0.68	m/s
Flow Discharge, Q	0.19	m3/s

2.2.2   Secondary Canals

Duty, q = 1.8   lit/sec/ha

Design Discharge, Qd = duty*Command Area

Table 12 Design Parameters of rectangular Secondary Canal

Canal Design	unit	SC-1	SC-2	SC-3	SC-4	SC-5
Command Area	ha	9.18	6.15	16.27	19.55	39.065
Canal bed width, b	m	0.25	0.25	0.25	0.25	0.3
Water Depth, d	m	0.02	0.02	0.05	0.05	0.17
Canal Side Slope, m	m/m	0.00	0.00	0.00	0.00	0.00
Water area, A	m2	0.01	0.01	0.01	0.01	0.05
Wetted Perimeter, P	m	0.29	0.29	0.35	0.35	0.64
Hydraulic Radius, R	m	0.02	0.02	0.04	0.04	0.08
Roughness Coefficient, n		0.015	0.015	0.015	0.015	0.015
Canal bed slope, I	m/m	0.6	0.3	0.1	0.15	0.015
Flow Velocity, V	m/s	3.45	2.44	2.33	2.85	1.51
Flow Discharge, Q	m3/s	0.0165	0.011	0.0293	0.035	0.07

2.2.3   Longitudinal Section of Canals

The longitudinal section along the canals is prepared with levels at intervals based on the natural topographic changes indicating the following details:

- ❖ Chain age
- ❖ Existing ground level (OGL)
- ❖ Canal bed level (CBL)
- ❖ Full supply level (FSL)
- ❖ Canal top level (CTL)
- ❖ Bed slope, Bed width (S, B)
- ❖ Discharge, velocity (Q, V)
- ❖ Location of turnouts and other related structures

The longitudinal profile of the main, secondary, and tertiary canals is shown in the drawings with canal section details.

2.3 CANAL STRUCTURES

There are many canal structures are on the main, secondary and tertiary canals such as:-

- ❖ Four Flume and Two Drainage crossing(super passage)
- ❖ 25 foot paths on MC and 40 field road For TC crossing
- ❖ 63 Turnouts and 34 off takes
- ❖ 3 chutes

And their design are prepared based on the canals design parameters which they are sited and their hydraulic characteristics, the detail working drawings each structures are attached with this engineering design report.

2.3.1 Drainage Culverts

There is one stream crossings which the canal alignment crosses at chainage 0 + 40, which needs cross drainage structures to pass the irrigation water through main canal under the natural drain, such that the drainage water runs above the canal freely. Hence, the BTL of the Canal is sufficiently below the bottom of the drain, so that the drainage water flows under gravity, the irrigation water flow through pipe culvert.
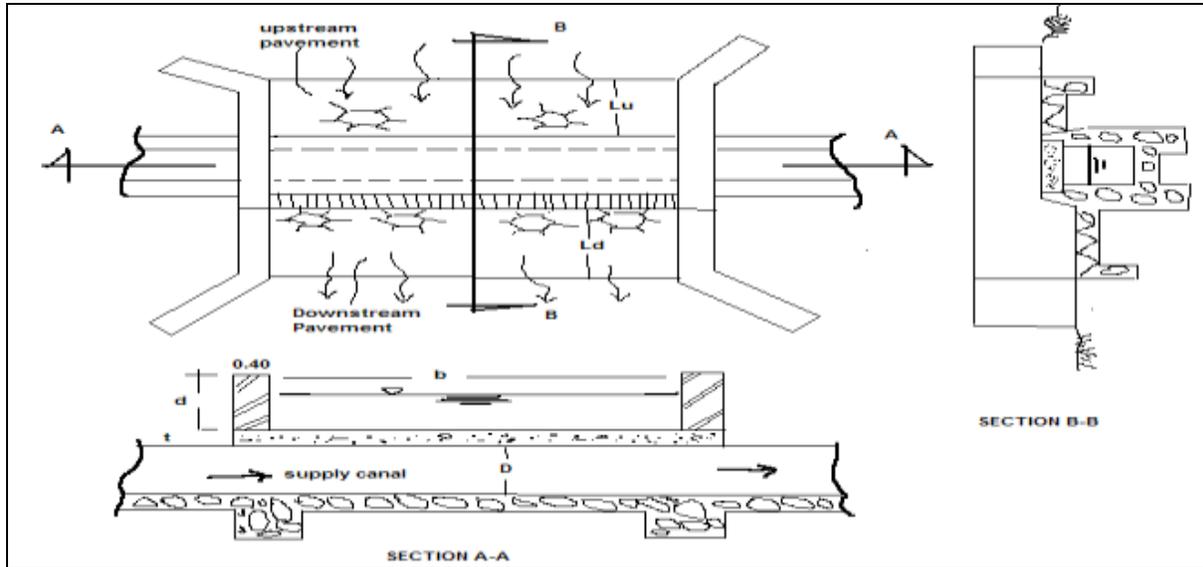
Figure 19 cross Section of Drainage super passage

Protection dyke is necessary as flood incoming is large and the flow channel is very wide and flat.

Hydraulic design principle is according to Lacey's regime perimeter

$P = 4.75\sqrt{Q}$ thus the total perimeter of the water way is chosen equal to P. And Q is the drainage passing above the supply canal and $Q = 5m^3/s$

$P = 4.75\sqrt{5} = 10.6$ too large

Hence span of supper passage can be taken to be 3m and calculate the flood height over the box like that of broad crested weir and check with Lacey's wetted perimeter.

    b=3m

$Q = 1.7bH3/2$ and thus $H = [Q/(1.7b)]2/3$

H=0.7m

For this condition the Lacey's parameter is fulfilled thus providing free board of 0.3m for height of guide wall above the crest will be 1.0m

Table 13 Location of Cross Drainage Structures on Main Canal

Chainage	Grid Coordinate		Type Of Canal	Structure	Pipe Diameter (m)	Elevation				Remark
	X	Y				OGL (m)	DBL (m)	FSL (m)	BTL (m)	
0+040	627739.32	933440.45	Main Canal	CD - 1	0.40	1630.63	1629.96	1630.43	1630.73	Stream 1

   2.3.2 Crossing Culverts

Road (Foot Path) Crossing

There are 25 major footpath and 40 field road crossings at the tertiary canal system of the Laga Kolu irrigation scheme. The width, (w) of the road depends on the width of existing footpath width (local foot path width), which is used to cross cattle from one area to another, the average width of crossing is 3.00m. Concrete slab thickness of 15cm, the width is equal to the lined canal width plus wall thickness.

2.3.3   Turnout

The turnouts are designed to divert flow to each field canal on proportional bases during the maximum flow and on rotational bases during the low flow (when the flow is less than the design flow of the canal). As it has already mentioned before, since field canals are not provided, simple turnouts at the main and tertiary canals, which directly take flow to the fields are provided. These turnouts are provided on the main, Tertiary canals based on the available command area and topography. At every 50 -100m distance between two field canals, one turn out is provided. The water surface level in tertiary canals at off take points of canals is maintained 10 to 15cm higher than the existing ground levels in the irrigable area for siphoning. The flow at each turn out (to each field canal) is controlled by gate operation. There are 63 numbers of turnouts and they are designed proportionally according to discharges in both parent and off take canals as follows.
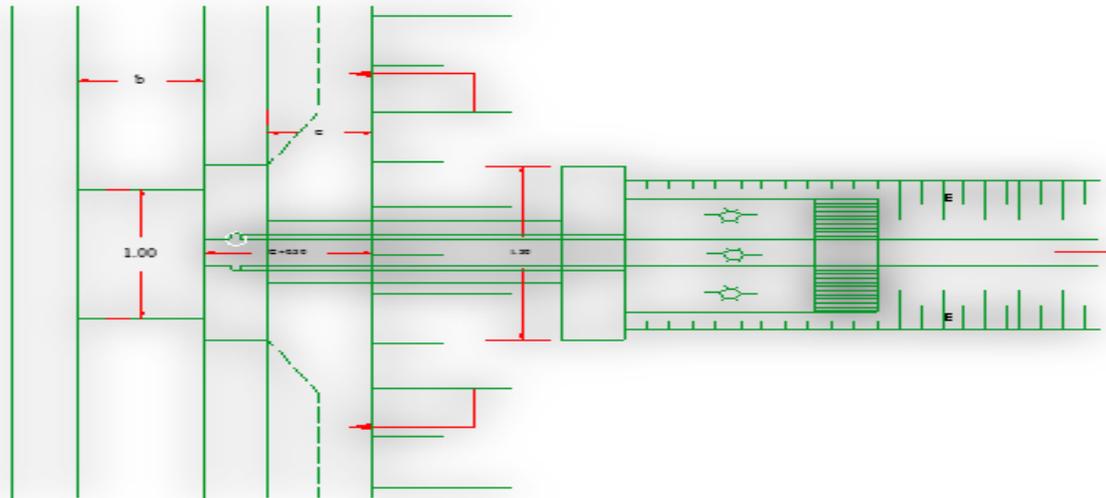
Figure 8 Layout of turnout

Plan of Turnout

Available data:-

Using orifice formula $Q = C_d \cdot A \cdot \sqrt{(2gHL)}$

Where, Q - Full Supply Discharge (m3/sec)

Discharge Coefficient, $C_d = 0.73$ for submerged flow

HL – working head or loss of head between u/s and d/s = 0.05 m

A - Area of pipe $= \pi d^2/4$

g = 9.81

Table 14 Irrigation outlet on Main Canal (MC)

No	Kind of canal	Turnout	Grid Coordinate		Station	Farm unit area	Area	Water Duty	Discharge Required	Pipe Diameter	Driving Head
		No	x	y	No	A	(Ha)	(m3/s/ha)	m3/sec	(m)	(m)
1	Main Canal	TO-1	627947.94	932737.86	869	MA – 1	0.55	0.0018	0.0010	0.047	0.05
2		TO-2	627973.17	932631.33	980	MA – 2	1.26		0.0023	0.071	0.05
3		TO-3	628047.26	932491.69	1161	MA – 3	1.38		0.0025	0.075	0.05
4		TO-4	628114.55	932401.87	1337	MA – 4	0.84		0.0015	0.058	0.05
5		TO-5	628193.34	932260.87	1501	MA – 5	2.37		0.0043	0.098	0.05
6		TO-6	628554.54	931466.28	2572	MA – 7	1.75		0.0032	0.084	0.05
7		TO-7	628905.44	930830.88	3375	MA – 9	2.45		0.0044	0.099	0.05
8		TO-8	629010.73	930661.19	3577	MA – 10	2.52		0.0045	0.101	0.05
9		TO-9	629113.10	930495.15	3777	MA – 11	2.16		0.0039	0.0934	0.05

Table 15 Irrigation outlet on Tertiary Canal (TC)

No	Kind of canal	Turnout No	Grid Coordinate x	Grid Coordinate y	Station No	Farm unit area A	Area (Ha)	Water Duty (m3/s/ha)	Discharge Required m3/sec	Pipe Diameter (m)	Driving Head (m)
1	TC-1-1-1	TO-1	628395.57	931913.83	104	TCA-1	0.81		0.0015	0.057	0.05
2		TO-2	628288.03	932084.75	315	TCA-2	2.70		0.0049	0.104	0.05
3	TC-1-1-2	TO-3	628450.35	931755.34	67	TCA-3	0.45		0.0008	0.043	0.05
4	TC-1-1-3	TO-4	628362.46	931897.25	107	TCA-4	2.47		0.0044	0.100	0.05
5	TC-1-1-4	TO-5	628430.29	931725.69	99	TCA-5	2.75		0.0050	0.105	0.05
6	TC-1-2-1	TO-6	628666.81	931341.32	116	TCA-6	1.80		0.0032	0.085	0.05
7	TC-1-2-2	TO-7	628804.17	931165.40	108	TCA-7	0.50		0.0009	0.045	0.05
8	TC-1-2-3	TO-8	628612.56	931256.96	101	TCA-8	1.30		0.0023	0.072	0.05
9	TC-1-2-4	TO-9	628787.67	931149.00	115	TCA-9	2.55		0.0046	0.101	0.05
10		TO-10	629502.95	930041.88	196	TCA-10	1.35		0.0024	0.074	0.05
11	TC-1-3-1	TO-11	629342.20	930160.50	398	TCA-11	0.84		0.0015	0.058	0.05
12		TO-12	629229.74	930326.04	599	TCA-12	0.70		0.0013	0.053	0.05
13	TC-1-3-2	TO-13	629673.62	929887.55	99	TCA-13	1.00		0.0018	0.064	0.05
14		TO-14	629470.91	930005.17	127	TCA-14	2.25		0.0041	0.095	0.05
15	TC-1-3-3	TO-15	629310.72	930140.71	342	TCA-15	2.65	0.0018	0.0048	0.103	0.05
16		TO-16	629192.30	930302.51	543	TCA-16	1.94		0.0035	0.089	0.05
17	TC-1-3-4	TO-17	629594.53	929830.34	113	TCA-17	0.91		0.0016	0.061	0.05
18	TC-1-3-5	TO-18	629366.40	929939.49	119	TCA-18	2.06		0.0037	0.091	0.05
19	TC-1-3-6	TO-19	629557.92	929799.00	139	TCA-19	2.57		0.0046	0.102	0.05
20	TC-1-4-1	TO-20	629823.57	929571.62	120	TCA-20	1.32		0.0024	0.073	0.05
21		TO-21	629715.66	929782.96	363	TCA-21	1.30		0.0023	0.072	0.05
22		TO-22	629970.26	929429.04	100	TCA-22	1.24		0.0022	0.071	0.05
23	TC-1-4-2	TO-23	630167.58	929396.29	301	TCA-23	1.82		0.0033	0.086	0.05
24		TO-24	630349.66	929091.77	779	TCA-24	2.58		0.0046	0.102	0.05
25	TC-1-4-3	TO-25	629788.04	929509.89	112	TCA-25	1.17		0.0021	0.069	0.05
26		TO-26	629669.48	929703.91	344	TCA-26	1.64		0.0030	0.081	0.05
27	TC-1-4-4	TO-27	629758.42	929459.02	109	TCA-27	1.14		0.0021	0.068	0.05
28		TO-28	629628.08	929632.67	331	TCA-28	1.84		0.0033	0.086	0.05
29	TC-1-4-5	TO-29	629730.05	929410.14	107	TCA-29	3.02		0.0054	0.110	0.05
30		TO-30	629594.69	929575.22	324	TCA-30	2.48		0.0045	0.100	0.05

No	Kind of canal	Turnout No	Grid Coordinate x	Grid Coordinate y	Station No	Farm unit area A	Area (Ha)	Water Duty (m3/s/ha)	Discharge Required m3/sec	Pipe Diameter (m)	Driving Head (m)
31	TC-1-5-1	TO-31	630413.95	928890.60	131	TCA-31	1.25		0.0023	0.071	0.05
32		TO-32	630517.50	928719.50	333	TCA-32	0.64		0.0012	0.051	0.05
33	TC-1-5-2	TO-33	630288.93	929102.89	212	TCA-33	2.15		0.0039	0.093	0.05
34		TO-34	630142.86	929339.97	600	TCA-34	1.13		0.0020	0.068	0.05
35		TO-35	629940.36	929373.94	808	TCA-35	0.86		0.0015	0.059	0.05
36	TC-1-5-3	TO-36	630348.81	928851.75	115	TCA-36	1.11		0.0020	0.067	0.05
37		TO-37	630469.16	928690.15	320	TCA-37	0.74		0.0013	0.055	0.05
38	TC-1-5-4	TO-38	630231.12	929072.87	201	TCA-38	2.89		0.0052	0.108	0.05
39		TO-39	630129.44	929301.26	570	TCA-39	2.56		0.0046	0.102	0.05
40		TO-40	629917.75	929335.04	787	TCA-40	2.72		0.0049	0.105	0.05
41	TC-1-5-5	TO-41	630292.67	928834.52	112	TCA-41	1.40		0.0025	0.075	0.05
42		TO-42	630426.35	928666.34	329	TCA-42	0.99		0.0018	0.063	0.05
43	TC-1-5-6	TO-43	630115.08	929006.82	115	TCA-43	2.50		0.0045	0.100	0.05
44		TO-44	630057.97	929175.91	540	TCA-44	1.98		0.0036	0.089	0.05
45		TO-45	629828.59	929195.97	802	TCA-45	1.85		0.0033	0.086	0.05
46	TC-1-5-7	TO-46	630224.38	928813.57	114	TCA-46	1.41		0.0025	0.075	0.05
47		TO-47	630368.26	928648.52	335	TCA-47	1.11		0.0020	0.067	0.05
48	TC-1-5-8	TO-48	630044.52	928977.51	122	TCA-48	3.20		0.0058	0.114	0.05
49		TO-49	629961.95	929048.93	235	TCA-49	1.56		0.0028	0.079	0.05
50		TO-50	629774.57	929135.25	445	TCA-50	0.98		0.0018	0.063	0.05
51	TC-1-5-9	TO-51	630149.89	928793.64	107	TCA-51	1.23		0.0022	0.070	0.05
52		TO-52	630312.32	928631.35	337	TCA-52	1.44		0.0026	0.076	0.05
53	TC-1-5-10	TO-53	630090.65	928773.59	102	TCA-53	1.70		0.0031	0.083	0.05
54		TO-54	630218.54	928605.01	325	TCA-54	1.67		0.0030	0.082	0.05

2.3.4   Main canal protection

Due to main canal routes in the river valley for more than 50m, river flood will damage it. So to protect the canal from damage, dyke of 1.5H: 1V with height equal to downstream retaining wall should have to construct extending up to where canal leave the valley

2.3.5   REFERENCE

1. *Farmers and Administrational Institutions in the nearby of the site.*

2. *Irrigation Engineering and Hydraulic structures by S.K.Garg*

3. *Irrigation Engineering and Hydraulics structure by S.R. Saharabuthe, 1994*

BILLOFF QUANTITY

Item No.	Description of work	Unit	Qty	Unit cost	Total cost
**Bill of Quantities for Laga Kolu S.S.I.P.**				
1	Access Road Construction				
1.1	Access Road Maintenance cutting to an average depth of 0.2m, with 6m width	km	7	7000	49,000.00
1.2	Field Road construction cutting to an average depth of 0.15m with 3m width	km	6.6	3500	23,100.00
**Total Carried Summary Birr**					**72,100.00**
2	**Camping (3m x 13.85m office & bed room, 4m x 6m kitchen & Cafeteria, 5m x 5m store, 4mx2m Toilet & Shower, 2m x 2m guard house**				
2.1	Site clearing	m2	175	2.92	511.00
2.2	Excavation	m3	63.336	57.2	3,622.82
2.3	Cart away all excess excavated material for safe place with a radius of more than 500m	m3	88.52	31.2	2,761.82
2.4	25cm thick hard core	m3	89.7	36.4	3,265.08
2.5	Masonry work with 1:3 mortar mix	m3	38.404	858.54	32,971.37
2.6	5cm thick mass concrete (1:2:4 mix ratio)	m3	12.71	1698.84	21,592.26
2.7	2cm cement screed	m2	91	90.64	8,248.24
2.8	CIS walling G-32	m2	337	142.91	48,160.67
2.9	CIS roofing G-32	m2	194.5	211.94	41,222.33
2.1	Chip wood wall ceiling	m2	256	129.33	33,108.48
2.11	Supply, assemble and fix in position eucalyptus wall post of length 3 m with span length of 1.2m	No	161	225	36,225.00
2.12	Supply and fix purlin in Eucalyptus wood size 50 x 70 mm nailed into eucalyptus truss	m	586	69.375	40,653.75
2.13	Supply, assemble and fix in position eucalyptus roof truss	No	36	225	8,100.00
2.14	Supply and fix purlin in zigba wood size 50 x 70 mm nailed into eucalyptus truss including three coats of anti - termite external treatment	m	190	69.375	13,181.25

Bill of Quantities for Laga Kolu S.S.I.P.

Item No.	Description of work	Unit	Qty	Unit cost	Total cost
2.15	Supply and fix CIS doors size 1.0x2.10m with accessaries	No	14	3139.25	43,949.50
2.16	Supply and fix CIS windows size 1x1.2m	No	9	2150.25	19,352.25
2.17	Fence 2.0m height & 15cm φ eucalyptus poles placed every 2m with barbed wire at 20cm vertical interval & erected in 0.6m depth embedded with concrete	LS	1	84943.2	84,943.20
	Total Carried Summary Birr				**513,969.02**
3	Head Work				
3.1	Weir , wing wall &Stilling Basin Including Piles				
3.1.1	Soil excavation 30%	m3	200.1	57.2	11,445.72
3.1.2	Soft Rock Excavation50%	m3	333.5	108.2	36,084.70
3.1.3	Hard Rock excavation10%	m3	66.7	277	
3.1.3	Masonry work	m3	275.462	1456	401,072.67
3.1.4	Concrete works	m3	217.11	2055.6	446,291.32
3.1.5	Plastering work	m2	251.07	123.23	30,939.36
	Total Carried Summary Birr				**1,439,802.78**
3.2	Intake & Under sluice Gates		2		
3.2.1	Concrete work	m3	7.22	2002.72	14,459.64
3.2.2	Reinforcement Iron Bars	kg	26.54	85.5	2,269.17
3.2.2	30 cm Ø Pipe Placing	m	2.2	450	990.00
3.2.3	Fix Intake gate 0.60mX0.40m with 6mm thick sheet plate	pcs	1	10000	10,000.00
3.2.4	Fix under sluice gate 1m X 1.4m with 6mm thick sheet plate	pcs	1	15000	15,000.00
	Total Carried Summary Birr				**1,482,521.59**
4	Main Canal & Related Structures				
4.1	Main Canal	Km	6.158		
4.1.1	Lined	km	6.158		
4.1.1.1	Soil excavation 70%	m3	28006.37	57.2	1,601,964.36
4.1.1.2	Soft Rock Excavation 20%	m3	8001.82	108.2	865,796.92
4.1.1.3	Hard Rock excavation 10%	m3	4000.91	277	1,108,252.07

Item No.	Description of work	Unit	Qty	Unit cost	Total cost
	Bill of Quantities for Laga Kolu S.S.I.P.				
4.1.1.3	Fill & compaction	m3	1612	143.44	231,225.28
4.1.1.4	Masonry Work	m3	3990.384	1456	5,809,999.10
4.1.1.5	Concrete Work	m3	381.796	2002.72	764,630.49
4.1.1.6	Plastering work	m2	8251.72	123.23	1,016,859.46
	Main canal protection work				
4.1.1.7	Dyke L=69m in river coarse fill & compaction	m3	692.52		
	Total Carried Summary Birr				**12,881,249.27**
4.1.2	Drainage Culvert -super passage	no	2		
4.1.2.1	Soil excavation 70%	m3	17.5	57.2	1,001.00
4.1.2.2	Soft Rock Excavation 30%	m3	7.5	108.2	811.50
4.1.2.3	Backfill & Compaction	m3	6.25	143.44	896.50
4.1.2.4	Masonry Work	m3	15.2	1456	22,131.20
4.1.2.5	Concrete Work	m3	1.23	2002.72	2,463.35
4.1.2.6	Plastering	m2	22	123.23	2,711.06
	Total Carried Summary Birr				**12,911,263.88**
4.1.3	Flume on Main Canal L=52m	no	4		
4.1.3.1	Soil excavation 60%	m3	32.256	57.2	1,845.04
4.1.3.2	Soft Rock Excavation 20%	m3	10.752	108.2	1,163.37
4.1.3.3	Hard Rock 20%		10.752	277	2,978.30
4.1.3.4	Backfill & Compaction	m3	13.44	143.44	1,927.83
4.1.3.5	Masonry Work	m3	6.84	1456	9,959.04
4.1.3.6	Formwork	m2	247.368	243.5	60,234.11
4.1.3.7	Eucalyptus pole 12cm diam.	pcs	216	180	38,880.00
	Concrete Work				
4.1.3.6	C-15	m3	7.15	2002.72	14,319.45
4.1.3.7	C-25	m3	29.84	2185.1	65,203.38
4.1.3.8	Reinforcement Iron Bars	kg	1761.837	85.5	150,637.06

**Bill of Quantities for Laga Kolu S.S.I.P.**
Item No.	**Description of work**	**Unit**	**Qty**	**Unit cost**	**Total cost**
4.1.3.9	Plastering	m2	293.768	123.23	36,201.03
	Total Carried Summary Birr				**13,294,612.50**
4.1.4	chutes on main canal-L=170m	no	3		
4.1.4.1	Soil excavation 70%	m3	90.6696	57.2	5,186.30
4.1.4.2	Soft Rock Excavation20%	m3	22.6674	108.2	2,452.61
4.1.4.3	Backfill & Compaction	m3	28.33425	143.44	4,064.26
4.1.4.4	Masonry Work	m3	52.484	1456	76,416.70
4.1.4.5	Concrete Work (C-15)	m3	9.3	2002.72	18,625.30
4.1.4.6	Plastering	m2	102	123.23	12,569.46
	Total Carried Summary Birr				**13,413,927.14**
4.1.5	Turnouts	Pcs	9		
4.1.5.1	Soil excavation 70%	m3	22.68	57.2	1,297.30
4.1.5.2	Soft Rock Excavation 30%	m3	9.72	108.2	1,051.70
4.1.5.3	Masonry Work	m3	15.1714286	1456	22,089.60
4.1.5.4	Concrete Work	m3	5.67	2002.72	11,355.42
4.1.5.5	Pipe Placing Dia.100mm.	pcs	9	250	2,250.00
4.1.6	Foot bridge				
4.1.6.1	Masonry Work	m3	18.72	1456	27,256.32
4.1.6.2	Concrete Work	m3	35.1	2002.72	70,295.47
	Total Carried Summary Birr				**13,549,522.95**
5	Secondary Canals, & Related Structures	no	5		
5.1	Secondary Canals,	Km	0.83		
5.1.1	Lined	Km	0.83		
5.1.1.1	Soil excavation 80%	m3	140.12	57.2	8,014.86
5.1.1.2	Soft Rock Excavation 20%	m3	35.03	108.2	3,790.25
5.1.1.3	Fill & compaction	m3	7	143.44	1,004.08
5.1.1.4	Masonry Work	m3	286.35	1456	416,925.60
5.1.1.5	Concrete Work C-15	m3	24.9	2002.72	

**Bill of Quantities for Laga Kolu S.S.I.P.**

Item No.	Description of work	Unit	Qty	Unit cost	Total cost
					49,867.73
5.1.1.6	Plastering work	m2	498	123.23	61,368.54
	Total Carried Summary Birr				**14,090,494.01**
6	**Tertiary Canal & Related Structures**	no.	29		
6.1	Tertiary Canal (TC)	Km	9.6		
6.1.1	Lined	Km	9.6		
6.1.1.1	Soil excavation 70%	m3	4565	57.2	261,118.00
6.1.1.2	Fill & compaction	m3	160	143.44	22,950.40
6.1.1.3	Masonry Work	m3	4896	1456	7,128,576.00
6.1.1.4	Concrete Work	m3	288	2002.72	576,783.36
6.1.1.5	Plastering	m2	5760		
6.1.1.6	Plastering work	m2	352	123.23	43,376.96
	Total Carried Summary Birr				**22,123,298.73**
6.1.2	Turnouts	Pcs	54		
6.1.2.1	Soil excavation 70%	m3	136.08	47.91	6,519.59
6.1.2.2	Soft Rock Excavation 30%	m3	58.32	117.85	6,873.01
6.1.2.3	Masonry Work	m3	91.0285714	1456	132,537.60
6.1.2.4	Concrete Work	m3	34.02	2002.72	68,132.53
6.1.2.5	Pipe Placing Dia.100mm.	pcs	108	250	27,000.00
	Total Carried Summary Birr				**22,364,361.47**
7	**Total Project Cost**	Birr			**22,364,361.47**
8	Overhead cost =5% of Total Project Cost	Birr			1,118,218.07
9	Total Carried Summary Birr	Birr			**23,482,579.55**
10	For Supervision Cost = 5% of final Total Summary Birr	Birr			1,174,128.98
11	Final Total Carried Summary Birr	Birr			**24,656,708.52**
12	Vat 15%	Birr			3,698,506.28
13	Grand Total	Birr			**28,355,214.80**