

**ADDIS ABABA UNIVERSITY
INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**



**Design of Small-Scale Surface Irrigation
The Case of Jalele, Ethiopia
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A Project Submitted to the School of Graduate Studies of Addis Ababa University Institute of Technology in Partial Fulfilment of the Requirements for the Degree of Master of Engineering in Hydraulic Engineering

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School of Civil and Environmental Engineering

This is to certify that the project prepared by **Gurmu Madessa Ija**, entitled: **Design of Small-Scale Surface Irrigation, the Case of Jalele** and submitted in partial fulfilment of the requirements for the Degree of Master of Engineering (Hydraulic Engineering) complies with the regulations of the university and meets the accepted standards with respect to originality and quality.

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ACRONYMS AND ABBREVIATIONS

E	East
PA	Peasants Association
N	North
FAO	Food, Agriculture Organization
MoWR	Ministry of Water Resource
MoA	Ministry of Agriculture

ABSTRACT

The project Jalele small scale irrigation is located in East Wollega Zone, Sibule Ana, at a distance of about 37km away from Nekemte, nearby Nekemte-Finfinne high way. The area of the catchment (watershed) is 17km² and the project is intending to develop 60 hectares of land by surface irrigation.

The overall objective of the project is to design and analyze of small-scale surface irrigation by using river water (Jalele river). Specifically, design of surface irrigation, design of head work (weir), and selection of weir type will be the major one performed in this project.

The necessary data that was collected and used for this study were primary and secondary data. Data essential for this project work was collected from different sources. The collected data include soil, agronomy, hydrology, geology, surveying that is used for headwork, main canal, and irrigation system design.

During peak flow, the water level attained is equivalent to 0.90m. The water level is obtained from stage ~ discharge curve. After preparing stage ~ discharge curve, the tail water depth is estimated as 0.90m. For stage ~ discharge curve, the discharge was calculated by using Manning equation, $Q = (A/n) * (R^{2/3} I^{1/2})$.

Expected maximum flow for Jalele catchments is estimated by Rational Method $Q_p = 0.00216 CIA^{0.73} = 0.00216 * 0.63 * 96mm * (1700ha)^{0.73} = 29.8 \text{ m}^3/\text{s} \approx 30\text{m}^3/\text{sec}$, based on field data observation, i.e. land slope classification, coverage of the area, and value of runoff coefficient, C. Thus, high flood level of the river (HFL) before construction of the structure is, $HFL = \text{river bed level} + \text{tail water depth} = 1861.30 + 0.90 = 1862.20$.

Ogee weir is selected from broad crested weir for big boulders are coming from upstream mountainous area. Other reason for selecting ogee weir is due to its high coefficient of discharge. Ogee weir is used to pass boulders coming from upstream easily.

Furrow surface irrigation, the most common method of surface irrigation, is recommended for this project, for the following reasons; it is suitable to the soil type of the project area; it has been traditionally exercised by the farmers of the project area; it is easily manageable at farmer's level; and it is suitable to irrigate all crops, which are recommended for the project.

TABLE OF CONTENTS

ACKNOWLEDGMENTS	II
ACRONYMS AND ABBREVIATIONS	III
ABSTRACT.....	IV
1. INTRODUCTION.....	1
1.1. Background	1
1.2. Objective of the Study.....	1
1.2.1. General Objective	1
1.2.2. Specific Objectives	1
2. LITERATURE REVIEW	2
2.1 General	2
2.2 Small Scale Irrigation in Ethiopia	4
3. METHODS AND MATERIALS	6
3.1 Description of the Study Area	6
3.2 Approach and Methodology.....	7
3.2.1 Soil	7
3.2.2 Agronomy	8
3.2.3 Geology.....	8
4. HEAD WORK, AND IRRIGATION SYSTEM DESIGN	9
4.1. Head Work Design	9
4.1.1 Weir Type Selection and Weir Design	9
4.1.2 Determination of Weir Crest Level.....	10
4.1.3 Estimation of Tail Water Depth.....	11
4.1.4 Peak Flood of the River	14
4.1.5 Surface Hydraulics.....	16
4.1.6 Stability Analysis	19
4.1.7 Design of Stilling Basin	22
4.2. Surface Irrigation and Drainage System Design.....	26
4.2.1 Field Irrigation Method.....	26
4.2.2 Proposed Water Distribution and Scheduling.....	27

4.2.3	Water Duty	29
4.2.4	Main Canal Design	30
4.2.5	Secondary Canals Design	33
4.2.6	Field Canals Design	34
4.2.7	Canal Structures Design.....	34
4.2.8	Cross Drainage Structures Design	41
4.2.9	Drainage Canals Design.....	42
4.2.10	Catch Drains.....	42
5.	RESULTS AND DISCUSSIONS	43
6.	CONCLUSION AND RECOMMENDATIONS	50
7.	REFERENCES	52

LIST OF TABLES

Table 1: Selection of an irrigation method based on soil type and net irrigation depth (Source: Jensen, 1983).....	3
Table 2: Furrow lengths in meters as related to soil type, slope, stream size and irrigation depth (Source: Kay, 1986)	4
Table 3: Practical values of maximum furrow lengths in meters depending on soil type, slope, stream size and irrigation depth for small-scale irrigation (Source: FAO, 1988).....	4
Table 4: Average River Bed Slope Estimation	11
Table 5: Elevation and depth data.....	14
Table 6: List of turnouts available in the project	29
Table 7: List of secondary canals location and length.....	33
Table 8: List of culverts on main and secondary canals	35
Table 9: Location of Turnouts	38
Table 10: Location and length of chutes of secondary canals	39
Table 11: Cross drainage structures and their locations	42

1. INTRODUCTION

1.1. Background

The rapidly increasing population of our country can no longer be fed by food crops, which are through traditional and backward agricultural activities. The uneven and erratic distribution of rainfall and the unusual weather changes are among the main reasons for the failure of agricultural productivity.

To overcome the shortage of food crops, proper management and utilization of the existing resources is one of the best solutions.

Jalele Small Scale Irrigation Development Project is proposed on the bases of these facts. Besides producing more food crops for family consumption, the cash income of the beneficiaries is expected to increase; and thus, their living standard would be raised, after the implementation of the project.

The project is located in East Wollega Zone, Sibul Sire Ana, at a distance of about 37km away from Nekemte, nearby Nekemte-Finfinne high way.

The main motivating force to do this project work goes to the benefit of small-scale irrigation. Small scale irrigation is little developed in Jalele area. As a result, designing and analysing small scale irrigation for Jalele is considered. By doing so, the local community particularly small farm holders of the area will be benefit from the project if the project is implemented.

1.2. Objective of the Study

1.2.1. General Objective

The general objective of the project is to design and analyze of small-scale surface irrigation.

1.2.2. Specific Objectives

The specific objectives of the project are:

- designing surface irrigation of Jalele small scale,
- designing head work (weir) for Jalele small scale,
- selecting weir type for Jalele small scale

2. LITERATURE REVIEW

2.1 General

Irrigation is defined as application of artificial water to the living plants for the purpose of food production and overcoming shortage of rainfall and help to stabilize agricultural production and productivity (FAO, 2005). Irrigation in Ethiopia is considered as a basic strategy to alleviate poverty and hence food security. It is useful to transform the rain-fed agricultural system which depends on rainfall into the combined rain-fed and irrigation agricultural system. According MoWR (2012) modern irrigation has been documented in the 1960s where the government designed large irrigation projects in the Awash Valley to produce food crops for domestic consumption and industrial crops for exports. Irrigation development is being suggested as a key strategy to improve agricultural productivity and to encourage economic development (Bhattarai *et al.*, 2007). The adoption of new technology (e.g., irrigation) is the major powerful for agricultural growth and poverty reduction (Norton *et al.*, 2010). The most commonly used and most ancient type is surface irrigation methods (FAO, 2002) through the usage of gravity forces. This was used especially across river sides and it does not depend on mechanized equipments. Nowadays, modernized irrigation systems are mostly used which works based on the pressurized energy system (FAO, 2001).

In order to be in a position to select an irrigation system for a given area, it is important to look at the types of irrigation systems commonly used. Based on the method of applying water to the land, there are four broad classes of irrigation systems: (1) surface irrigation systems, (2) sprinkler irrigation systems, (3) localized irrigation systems and (4) sub-surface irrigation systems.

Surface irrigation is the oldest and most common method of applying water to crops. It involves moving water over the soil in order to wet it completely or partially. The water flows over or ponds on the soil surface and gradually infiltrates to the desired depth. Surface irrigation methods are best suited to soils with low to moderate infiltration capacities and to lands with relatively uniform terrain with slopes less than 2-3% (FAO, 1974).

According to FAO (1989), 95% of the irrigated area in the world is under surface irrigation. Some of the major advantages of surface irrigation systems over other systems are that they are easy to operate and maintain with skilled labour, they are not affected by windy conditions and, with the exception of furrow irrigation, they are good for the leaching of the salts from the root zone. Generally, they are associated with low energy costs.

Soil type	Rooting depth of crop	Net irrigation depth per application (mm)	Surface irrigation method
Sand	Shallow	20-30	Short furrows
	Medium	30-40	Medium furrows, short borders
	Deep	40-50	Long furrows, medium borders, small basins
Loam	Shallow	30-40	Medium furrows, short borders
	Medium	40-50	Long furrows, medium borders, small basins
	Deep	50-60	Long borders, medium basins
Clay	Shallow	40-50	Long furrows, medium borders, small basins
	Medium	50-60	Long borders, medium basins
	Deep	60-70	Large basins

Table 1: Selection of an irrigation method based on soil type and net irrigation depth (Source: Jensen, 1983)

An engineer may have an opportunity to design a surface irrigation system as part of a new irrigation project where surface methods have been selected or when the performance of an existing irrigation system requires improvement by redesign. In either case, the data required fall into six general categories (Walker and Skogerboe, 1987):

- I. the nature of irrigation water supply in terms of the annual allotment, method of delivery and charge, discharge and duration, frequency of use and the quality of the water;
- II. the topography of the land with particular emphasis on major slopes, undulations, locations of water delivery and surface drainage outlets;
- III. the physical and chemical characteristics of the soil, especially the infiltration characteristics, moisture-holding capacities, salinity and internal drainage;
- IV. the cropping pattern, its water requirements, and special considerations given to assure that the irrigation system is workable within the harvesting and cultivation schedule, germination period and the critical growth periods;
- V. the marketing conditions in the area as well as the availability and skill of labour, maintenance and replacement services, funding for construction and operation, and energy, fertilizers, seeds, pesticides, etc.; and
- VI. the cultural practices employed in the farming region especially where they may prohibit a specific element of the design or operation of the system.

According to Michael (1994), rational procedures for predicting the water front advance and tail water recession in furrows, which are applicable to field designs, have not been developed. Various workers have proposed a number of quasi-rational procedures with varying degrees of adaptability. In the absence of more precise information on predicting the

water advance and recession in furrows, general principles regarding stream size, furrow length and furrow slope to obtain efficient irrigation are followed in field design.

Soil type		Clay		Loam			Sand			
Furrow slope %	Maximum stream size (l/sec)	Average irrigation depth (mm)								
		75	150	50	100	150	50	75	100	
0.05	3.0	300	400	120	270	400	60	90	150	
0.10	3.0	340	440	180	340	440	90	120	190	
0.20	2.5	370	470	220	370	470	120	190	250	
0.30	2.0	400	500	280	400	500	150	220	280	
0.50	1.2	400	500	280	370	470	120	190	250	
1.00	0.6	280	400	250	300	370	90	150	220	
1.50	0.5	250	340	220	280	340	80	120	190	
2.00	3.0	220	270	180	250	300	60	90	150	

Table 2: Furrow lengths in meters as related to soil type, slope, stream size and irrigation depth (Source: Kay, 1986)

Soil type		Clay		Loam		Sand	
Furrow slope %	Maximum stream size per furrow (l/sec)	Average irrigation depth (mm)					
		50	75	50	75	50	75
0.00	3.0	100	150	60	90	30	45
0.10	3.0	120	170	90	125	45	60
0.20	2.5	130	180	110	150	60	95
0.30	2.0	150	200	130	170	75	110
0.50	1.2	150	200	130	170	75	110

Table 3: Practical values of maximum furrow lengths in meters depending on soil type, slope, stream size and irrigation depth for small-scale irrigation (Source: FAO, 1988)

2.2 Small Scale Irrigation in Ethiopia

In Ethiopia, traditional irrigation was practiced before centuries (Bekele et al., 2012). Moreover, in the highlands of Ethiopia, irrigation practices have long been in use since ancient times for producing subsistence food crops (Awulachew et al., 2007; Bacha et al., 2011; MoA, 2011a). Different authors such as Awlachew et al (2007), Makombe et al. (2007), Hagos et al. (2009), Bacha et al. (2011) stressed that supplementary irrigation has been practiced by smallholder farmers of Ethiopia for centuries to solve their livelihood challenges.

Irrigated agriculture is being practiced under smallholders, medium and large-scale farming. Many authors such as Awlachew et al. (2007), Makomb et al., (2007), Hagos et al. (2009), Bacha et al. (2011) were used government based irrigation schemes classification systems

for their description during their studies. According to Ministry of Water Resources of Ethiopia (MoWR, 2002), irrigation development in Ethiopia is classified based on the size of the command area, in three types:

1. Small-scale irrigation systems (<200 hectares ha)
2. Medium-scale irrigation systems (200-3,000 ha)
3. Large-scale irrigation systems (>3,000 ha)

This classification system is the most common in Ethiopia. Accordingly, 46% of proposed irrigation developments are in the small-scale irrigation category (Makombe et al., 2011).

Small scale irrigation is a type of irrigation defined as irrigation, on small plots, in which farmers have the controlling influence and must be involved in the design process and decisions about boundaries (Tafesse, 2007). Small scale irrigation is an important strategy in reducing risks associated with rainfall variability and increasing income of rural farm households.

3. METHODS AND MATERIALS

3.1 Description of the Study Area

Climatic Characteristics

The study area is categorized under warm sub-humid agro climatic zone with average annual rain fall of 1342mm. The daily average maximum temperature and average minimum temperature are 28°C and 18°C respectively. The dry months of the year are from December to March whereas the rain months are from April to October. The elevation of the project is about 1875m a.m.s.l.

Hydrologic Characteristics

The area has available water resources in the form of surface and ground water. River and small streams are the main surface water resource of the area. Jalele is one of the catchments of Didesa river basin, which drains from north to south direction crossing the way of main asphalt road of Addis Ababa to Nekemte.

Grid coordinate is 9° 04' N ~ 9° 08'N latitude, 36° 44'E ~ 36° 48'E longitude from the upper most divide to the out let (the proposed weir site.) The area of the enclosed drainage is about 17Km² with large number of perennial springs, which enhance good flow potential to the main stream of Jalele (it has high stream density).

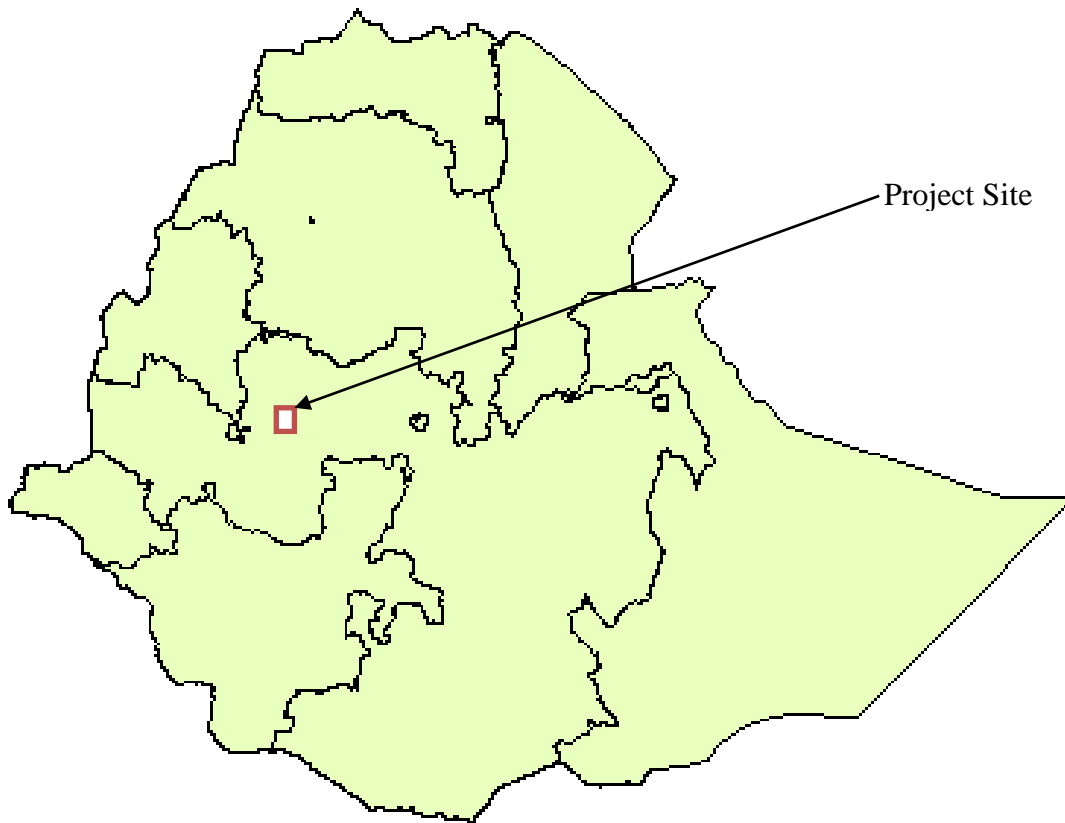


Figure 1: Study area location

3.2 Approach and Methodology

The necessary data that was collected and used for this study were classified into two: primary and secondary data. Data necessary for this project work was collected from different sources. Data collection include: soil, agronomy, hydrology, geology, surveying that is used for headwork, main canal, and irrigation system design.

3.2.1 Soil

For the study of physical and chemical properties of the command area and other associated studies, general field visit and reconnaissance was made and the command area has been divided into three soil units and field and laboratory studies have been carried out.

Concerning the main canal route (to know textural composition, infiltration, seepage condition and other associated studies), the laboratory and field studies of the samples have been carried out.

At every 200m distance, soil samples for laboratory analysis were collected at 0 ~ 30cm and 30 ~ 60cm depth. The samples collected are expected to represent the whole canal line. The vegetation, slope change, geology and other conditions of the area have been considered to collect the representative samples.

3.2.2 Agronomy

The project area farmers are practicing mixed system of farming, crop production and animal husbandry. Improved agricultural practices and mechanized agricultural technology are not well developed. However, fertilizer applications and using selected seed (package) are practiced by some farmers only on some crops.

3.2.3 Geology

It was not possible to get any written material about Jalele River in general and about the project site in particular.

To study and describe the geology of the site top- map of the area (scale 1:50,000) geological compass and geological hammer were used. Observations were made in the catchment, command area and streams and along the main canal. Geologic test pits were dug at right and left side edge of the river along the weir axis.

Information was gathered from the Zonal Water, Mines and Energy Resources Development Department concerning the boreholes dug in the areas so as to have information about the ground water.

4. HEAD WORK, AND IRRIGATION SYSTEM DESIGN

Head work, irrigation and drainage system design involves determination of more efficient and reliable hydraulic structures of the headwork including weir, stilling basin, and design of on farm structures.

4.1. Head Work Design

A weir is a concrete or masonry structure which is constructed across the open channel (such as a river) to change its water flow characteristics. Weirs are constructed as an obstruction to flow of water. In my project, the weir is a concrete structure constructed across Jalele river.

Weirs are classified according to shape of the opening, shape of the crest and effect of the sides on the emerging nappe.

- 1) Types of weirs based on shape of the opening
 - a. Rectangular weir
 - b. Triangular weir
 - c. Trapezoidal weir

- 2) Types of weirs based on shape of the crest
 - a. Sharp-crested weir
 - b. Broad- crested weir
 - c. Narrow-crested weir
 - d. Ogee-shaped weir: *type of weir for Jalele project.*

- 3) Types of weirs based on effect of the sides on the emerging nappe
 - a. Weir with end contraction (contracted weir)
 - b. Weir without end contraction (suppressed weir)

4.1.1 Weir Type Selection and Weir Design

The geology of the weir site is composed of alluvial deposits from the size of boulders up to small sized gravel around the riverbed. From the test pits taken on the axis of the weir (on both edges of the river), the profile is characterized by boulders with admixture of silty clay.

The upstream of the weir site that is catchment area is v ~ shaped valley and there are mountains of domes cliff forms. There are big boulders that comes from the catchment area.

Ogee weir is selected for the reason that big boulders are coming from upstream mountainous area. Ogee weir is the appropriate weir in transporting boulders from upstream to downstream smoothly. Other reason for selecting ogee weir is due to its high coefficient of discharge, as flow over an ogee weir is dependent on the discharge coefficient. Ogee weir, as the name says, represents the shape of the downstream face of the weir. The downstream face of the weir is constructed corresponding to the shape of lower nappe of freely falling water jet which is in ogee shape. The ogee shape of the downstream face is designed on the basis of the principle of a projectile. In general, the shape of lower nappe of the water jet is not constant for all water heads hence, the shape obtained for the maximum head is taken into account while designing ogee weir.

Data used for Design

- Peak Flood Discharge: The maximum probable flood discharge is $30.0\text{m}^3/\text{sec}$.
- Rating Curve: The water level attained during the floods before the construction of the weir is obtained from the stage-discharge curve, and the depth is equal to 0.90m.
- Cross Section of the River: The width of the river at the weir axis is measured to be 12.0m.
- Weir Crest Length: From field observations and design considerations, a weir length of 16.0m is fixed.
- The shape of the ogee weir depends upon the head, the inclination of the upstream face of the overflow section, and the height of the overflow section above the floor of the entrance channel.

4.1.2 Determination of Weir Crest Level

Average elevation of the highest field in the command area = 1860.70.

Water depth required = 0.31m.

Head loss across head regulator = 0.15m.

Head loss along the main canal = 0.807m.

Head loss at turnout = 0.05m.

Entrance, friction and exit loss = 0.0803m.

Free board = 0.10m.

Average river bed elevation = 1861.30m.

Total head loss = 1.50m.

Therefore, weir crest level = $1860.70 + 1.50 = 1862.20$.

Therefore, crest height, $h = 1862.20 - 1861.30 = 0.90\text{m}$.

4.1.3 Estimation of Tail Water Depth

The design peak flood discharge for the specific catchment is 30.0m³/sec.

No.	Station	Relative Dist (m)	Elevation	Cumulative Ht (m)	Area (m ²)	Remark
1	0+00	0	1859.53	0.00	0.000	
2	0+03	3	1859.83	0.30	0.450	
3	0+10	7	1860.24	0.71	3.535	
4	0+25	15	1860.75	1.22	14.475	
5	0+34	9	1860.82	1.29	11.295	
6	0+37	3	1861.02	1.49	4.170	
7	0+41	4	1861.36	1.83	6.640	Weir Axis
8	0+49	8	1861.77	2.24	16.280	
9	0+54	5	1862.24	2.71	12.375	
10	0+62	8	1862.52	2.99	22.800	
11	0+75	13	1862.68	3.15	39.910	
12	0+84	9	1862.69	3.16	28.395	
13	0+94	10	1863.41	3.88	35.200	
Total		94			195.525	

Table 4: Average River Bed Slope Estimation

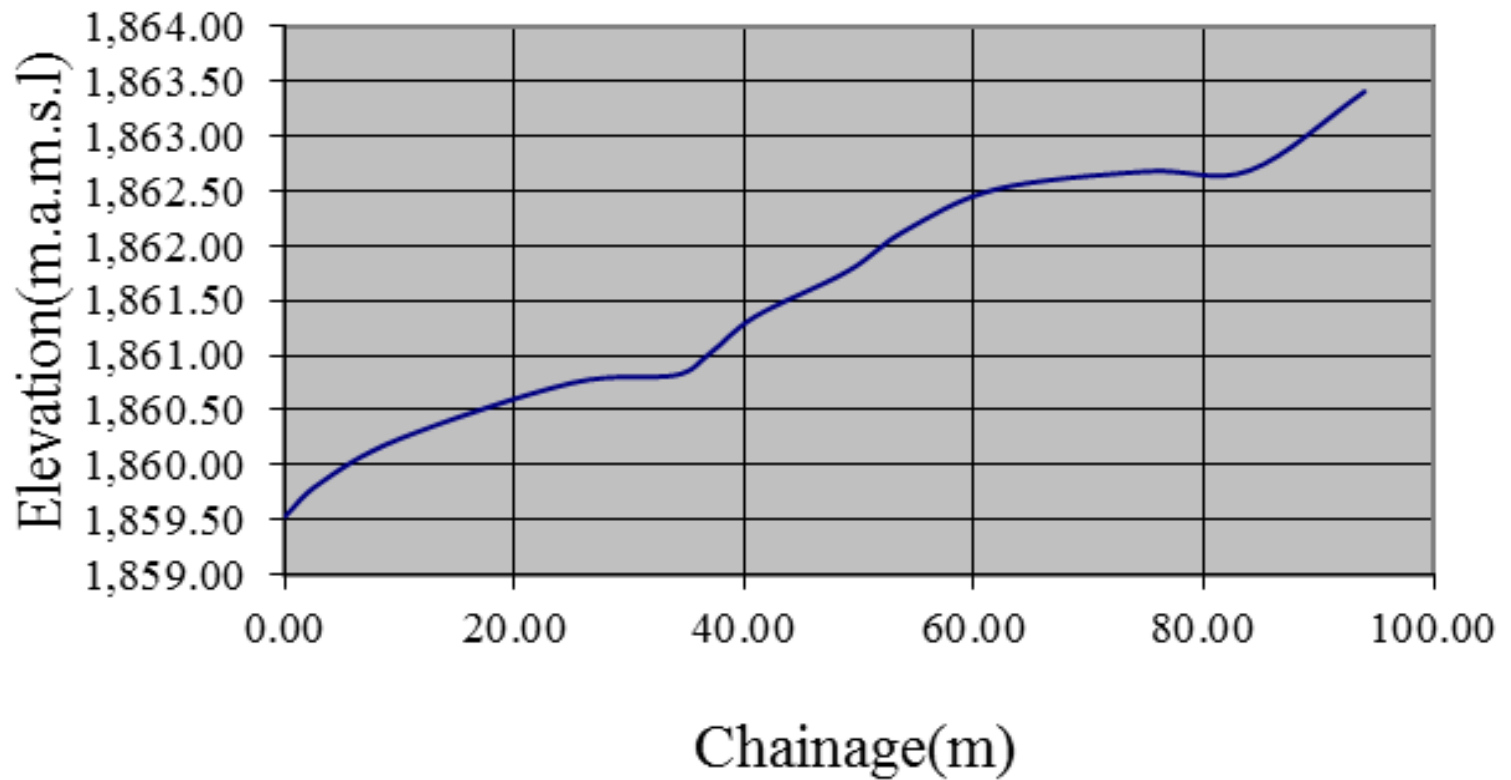
Individual Area, $A_n = \{[H_n + H_{(n-1)}]/2\} * L_n$

Cumulative Height, $E_n - E_0$

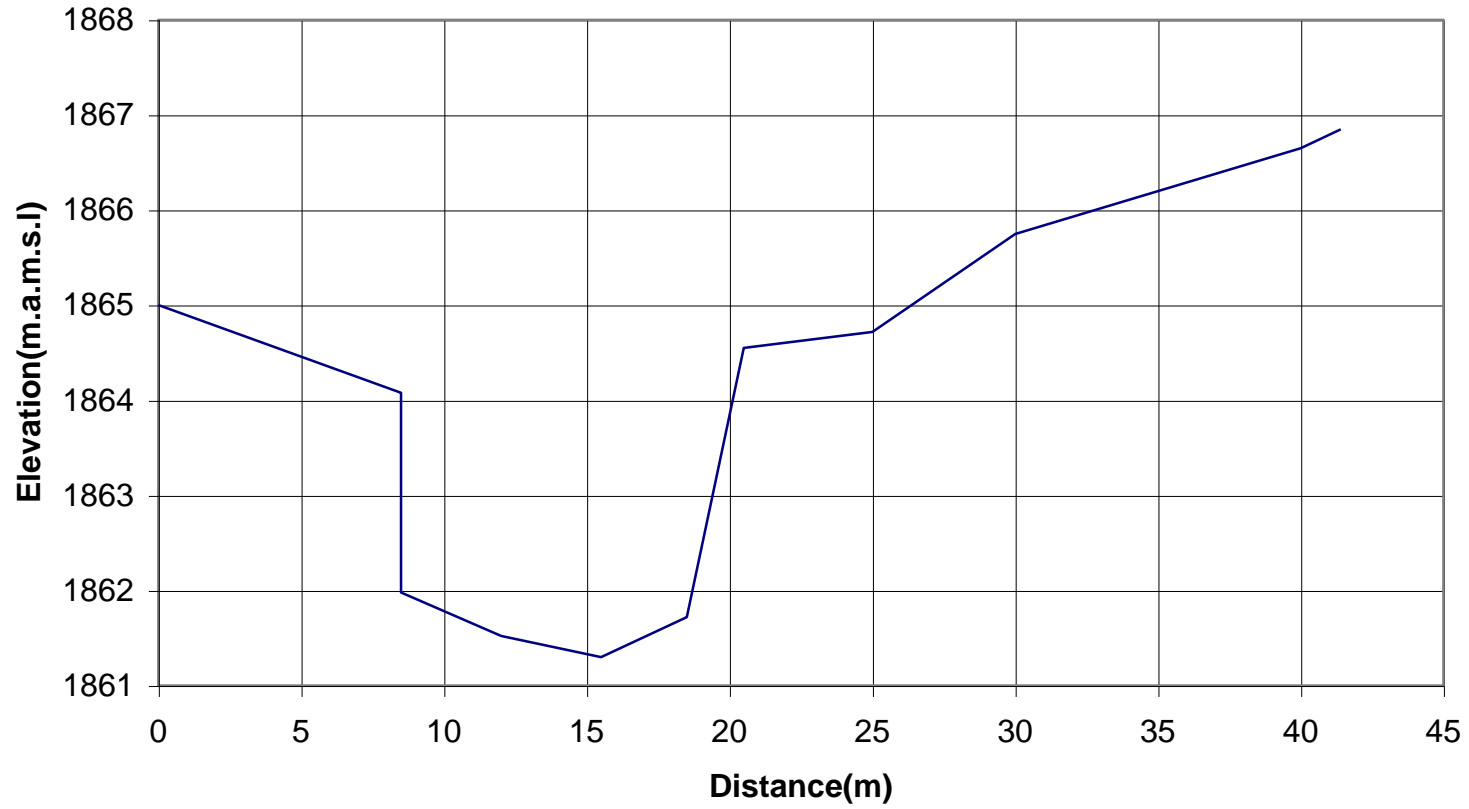
Average Height, $H_{av} = 2A/2 = (2*195.525m^2)/94 = 4.16m$.

Average River Bed Slope, $I_{av} = H_{av}/L = 4.16m/94m = 0.04425 = 4.425\%$

Longitudinal Cross Section of Jalele River



Transversal Cross Section of Jalele River(Weir Axis)



4.1.4 Peak Flood of the River

Elevation	Depth, d (m)	Water Area, A (m ²)	Wetted Perimeter, P (m)	Hydraulic Radius, R (m)	Velocity, V (m/sec)	Discharge, Q (m ³ /sec)	Remark
1861.30	0.00	0.000	0.000	0.000	0.000	0.000	Center of River
1861.52	0.22	0.558	5.090	0.109	1.371	0.765	
1861.72	0.42	1.867	8.066	0.231	2.265	4.236	
1861.98	0.68	4.233	10.379	0.408	3.305	13.991	
1864.08	2.78	25.614	14.579	1.756	8.751	224.146	

Table 5: Elevation and depth data

The discharge is calculated from the Manning's Equation,

$$Q = (A/n) * (R^{2/3} I^{1/2})$$

Where Q = Discharge (m³/sec)

A = Water Area (m²)

N = Manning's Roughness Coefficient

R = Hydraulic Radius (m)

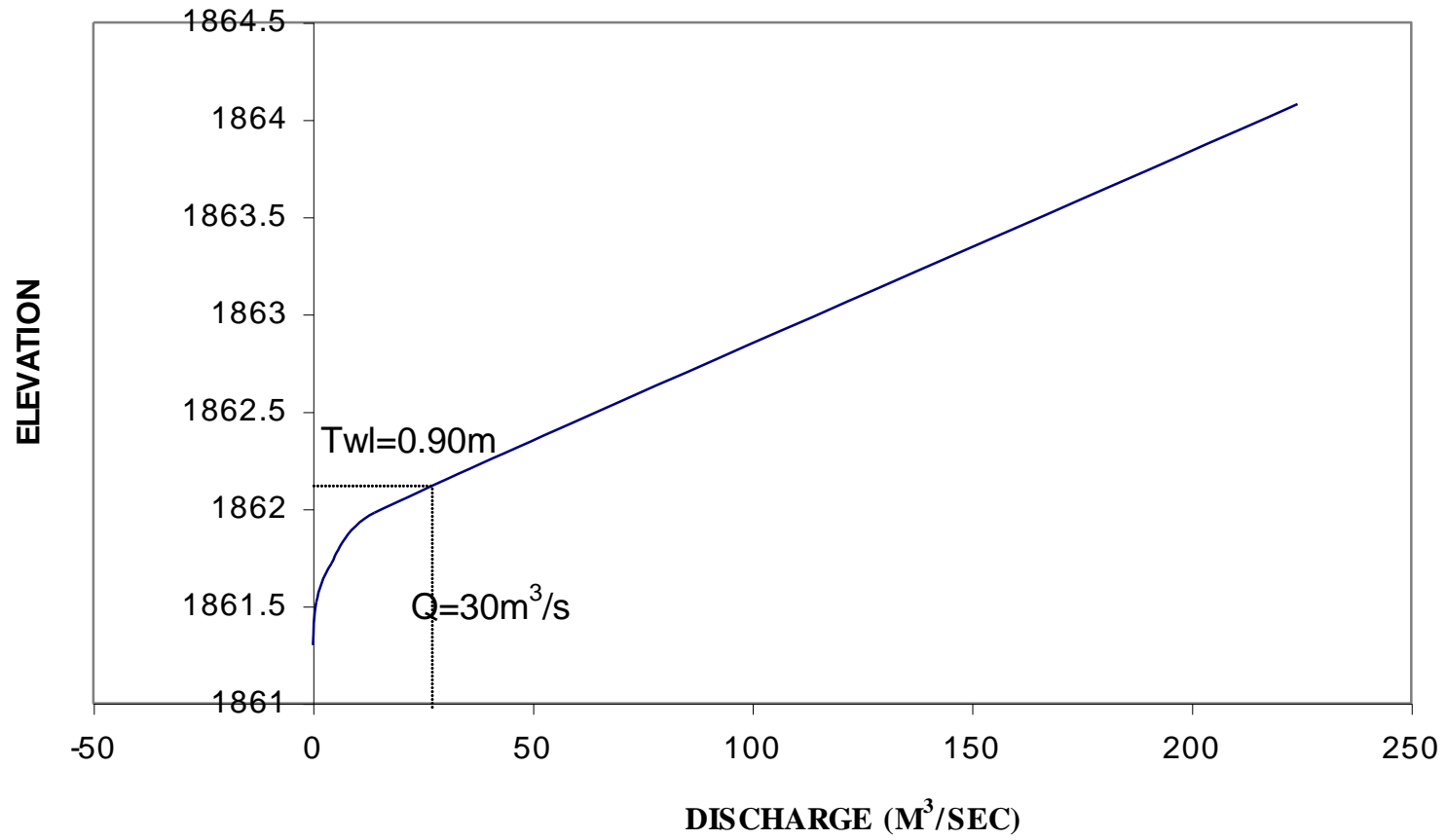
I = Average River Bed Slope

After preparing the Rating Curve (plotting discharge versus depth from table 5), the tail water depth (depth of water during the expected maximum flow) is estimated as 0.90m.

Therefore, high flood level of the river (HFL) before construction of the structure is:

$$\text{HFL} = 1861.30 + 0.90 = 1862.20$$

THE RATING CURVE



4.1.5 Surface Hydraulics

Water Depth on the Crest

Shape of the Weir: Considering boulder coming from the upstream of the weir and taking into account its high discharge coefficient it is decided to design an ogee shaped weir with inclined upstream face.

Crest Length of the Weir: From physical field observations, the weir crest length should be equal to 16.0m.

Discharge over the weir is generally expressed as:

$$Q = CLH_e^{3/2}, \text{ where } Q = \text{Peak flood (m}^3/\text{sec)}/50 \text{ years return period/}$$

$$L = \text{Weir crest length (m)}$$

$$C = \text{Coefficient of discharge} = 2.22$$

$$H_e = \text{Height of energy line above the crest (m)}$$

$$\text{Therefore, } H_e = [Q/CL]^{2/3} \Rightarrow H_e = [30/(2.22*16)] = 0.893\text{m.}$$

$$H_e = 0.893\text{m}$$

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

$$V_a = \{30/[16(0.9 - 0.4) + H_d]\} = [1.875/(0.5 + H_d)]$$

Therefore, $h_v = V_a^2/2g = \{(30)^2/[(16(0.5 + H_d))]^2\} * 1/2g$

$$\Rightarrow h_v = [0.17918/(0.5+H_d)^2] \text{ ----- (i)}$$

$$\text{But } H_e = H_d + h_v \Rightarrow h_v = H_e - H_d$$

$$\Rightarrow h_v = 0.893 - H_d \text{ ----- (ii)}$$

Equating (i) & (ii),

$$[0.17918/(0.5+H_d)^2] = 0.893 - H_d$$

H _d	Left Side	Right Side	Remark
0.75	0.1146	0.143	
0.78	0.109	0.113	
0.785	0.108	0.108	Ok

Therefore, H_d = 0.785m, h_v = 0.108m.

The equation of ogee- shaped weirs as of the Water Ways Experiment Station is expressed as:

$$X^n = K_0 H_d^{n-1} Y,$$

Where:

X and Y are co-ordinates of the crest profile with the origin at the highest point of the crest, H_d is the design head excluding the head due to the velocity of approach and, K_0 and n are parameters depending on the slope of the upstream face. For our case,

The upstream face has a slope of 3Vertical:2Horizontal.

$$K_0 = 1.939$$

$$n = 1.810$$

Substituting the values in the above equation, we get the following formula for the weir crest geometry;

$$Y = -X^{1.810}/1.5937$$

The shape can be determined by assigning different values to X and calculating the corresponding co-ordinate, Y.

No.	X	$Y = -X^{1.810}/1.5937$
1	0.000	0.000
2	0.250	-0.051
3	0.500	-0.179
4	0.750	-0.372
5	1.000	-0.627
6	1.225	-0.905

Determination of the Radius of the Base Bucket

The toe of the weir should be connected to the base by a circular curve.

$$V = \sqrt{[2g(h + 0.5H_d)]}, \text{ where } V = \text{velocity at the toe of the weir}$$

$$H = \text{weir height} = 0.90\text{m.}$$

$$H_d = \text{design head} = 0.785\text{m.}$$

$$\Rightarrow V = \sqrt{[2*9.81(0.90 + 0.5*0.785)]} = 5.035\text{m/sec}$$

$$K = [(V + 6.40H + 4.88)/(3.6H_d + 19.50)]$$

$$= [(5.035+6.40*0.893 + 4.88)/(3.60*0.785 + 19.50)]$$

$$\Rightarrow K = 0.70$$

$$R_0 = 0.305 \cdot 10^k = 0.305 \cdot 10^{0.70}$$

$$\Rightarrow R_0 = 1.528 \text{ m}$$

The arc at the base should be tangent to the weir face (slope of the tangent is 1V:0.75H).

In considering the reverse, the ogee curve is continuous up to the value of $X = 1.225$; thus the Y value becomes -0.905 .

$$\text{Let } f(X) = -X^{1.810}/1.5937 \text{ and } g(X) = -0.905.$$

f and g are continuous on $[0, 1.225]$ and $f(X) \geq g(X)$. Then the area A of the region R between the graphs of f and g on $[0, 1.225]$ is given by:

$$\begin{aligned} A &= \int_0^{1.225} [f(X) - g(X)] dx \\ &= \int_0^{1.225} [(-X^{1.810}/1.5937) - (-0.905)] dx \\ &= (-1/1.5937) \int_0^{1.225} X^{1.810} dx + 0.905 \int_0^{1.225} dx \\ &= (-1/1.5937) [X^{(1.810+1)} / (1.810+1)]_0^{1.225} + 0.905 [X]_0^{1.225} \\ &= -0.3949 + 1.108 \\ \Rightarrow A &= 0.713 \text{ m}^2 \end{aligned}$$

Then the moments M_x and M_y about the axes are given by:

$$\begin{aligned} M_x &= \int_0^{1.225} \{ [f(X)]^2 - [g(X)]^2 \} dx \\ &= \int_0^{1.225} \{ [-X^{1.810}/1.5937]^2 - (-0.905)^2 \} dx \\ &= (1/5.079) [(X^{3.62+1}) / (3.62+1)]_0^{1.225} - (1/2)(0.819) [X]_0^{1.225} \\ \Rightarrow M_x &= -0.3928 \end{aligned}$$

$$Y = M_x/A = -0.3928/0.713 = -0.55 \text{ m.}$$

$$M_y = \int_0^{1.225} X[f(X) - g(X)]dx$$

$$M_y = \int_0^{1.225} X[-X^{1.810}/1.5937 + 0.905]dx$$

$$= -0.3568 + 0.679$$

$$\Rightarrow M_y = 0.322m.$$

$$X = M_y / A = 0.322/0.713 = 0.451m$$

Therefore, Center of Gravity of the ogee geometry, (X,Y) = (0.451,-0.55)

4.1.6 Stability Analysis

For the analysis of stability, the following parameters are considered:

Unit weight of concrete = 2.3T/M³

Unit weight of Masonry = 2.1T/M³

Unit weight of Water = 1.0T/M³

Unit weight of Silt = 1.36T/M³

I. Static Case

No.	Name & Description of Force	Magnitude of Force (T)		Arm Length (M)	Moment about Toe (TM)	
		Vertical	Horizontal		MR (+)	MOO(-)
1	Self-Weight – W ₁	0.529	-	2.840	1.502	-
2	Self-Weight – W ₂	0.388	-	2.530	0.983	-
3	Self-Weight – W ₃	0.015	-	2.490	0.037	-
4	Self-Weight – W ₄	1.640	-	1.969	3.229	-
5	Self-Weight – W ₅	1.050	-	2.990	3.139	-
6	Self-Weight – W ₆	1.622	-	1.967	3.199	-
7	Self-Weight – W ₇	0.158	-	2.640	0.416	-
8	Self-Weight – W ₈	0.040	-	1.078	0.043	-
9	Self-Weight – W ₉	0.110	-	1.080	0.120	-

10	Self-Weight – W ₁₀	0.839	-	1.250	1.046	-
11	Self-Weight – W ₁₁	0.166	-	0.860	0.142	-
12	Self-Weight – W ₁₂	0.074	-	0.780	0.058	-
13	Self-Weight – W ₁₃	0.048	-	0.260	0.013	-
14	Self-Weight – W ₁₄	2.100	-	0.500	1.050	-
14	Water Pressure -W _p	-	0.405	1.873	-	0.758
15	Silt Pressure – SP ₁	-	0.109	1.706	-	0.186
16	Water Weight – WW ₁	0.405	-	2.940	1.191	-
17	Uplift Pressure – UP ₁	3.771	-	1.620	-	6.109
18	Uplift Pressure – UP ₂	1.019	-	2.160	-	2.201

Summation of all Vertical Forces, $\Sigma F_v = 4.393$ T.

Summation of all Horizontal Forces, $\Sigma F_h = 0.5138$ T.

Summation of all Balancing Moments, $\Sigma M(r) = 16.167$ T.M.

Summation of all Overturning Moments, $\Sigma M(o) = 9.2536$ T.M.

Summation of all Moments, $\Sigma M = 6.913$ T.M.

Check for Stability

Check for Sliding

Factor of safety against sliding, $F.S/s = \mu \Sigma F_v / \Sigma F_h > 1.50$.

$$F.S/s = 0.65(4.393)/0.5138 = 5.55 > 1.50. \text{----- Ok.}$$

Check for Overturning

Factor of safety against overturning, $F.S/ot = \Sigma M(r) / \Sigma M(o) > 1.50$.

$$F.S/ot = 16.167/9.2536 = 1.747 > 1.50. \text{----- Ok.}$$

Check for Overstress

$$X = \Sigma M / \Sigma F_v = 6.913 / 4.393 = 1.573m.$$

$$\text{Eccentricity, } e = /1.573 - L/2/ = /1.573 - 3.24/2/ = 0.047m.$$

$$\text{Allowable Eccentricity, } e = L/6 = 3.24/6 = 0.54m.$$

$0.047 < 0.54$. Hence, no tension will develop anywhere in the structure.

II. Dynamic Case

No.	Name & Description of Force	Magnitude of Force (T)		Arm Length (M)	Moment about Toe (TM)	
		Vertical	Horizontal		MR (+)	MO (-)
1	Self-Weight – W ₁	0.529	-	2.840	1.502	-
2	Self-Weight – W ₂	0.388	-	2.530	0.983	-
3	Self-Weight – W ₃	0.015	-	2.490	0.037	-
4	Self-Weight – W ₄	1.640	-	1.969	3.229	-
5	Self-Weight – W ₅	1.050	-	2.990	3.139	-
6	Self-Weight – W ₆	1.622	-	1.967	3.199	-
7	Self-Weight – W ₇	0.158	-	2.640	0.416	-
8	Self-Weight – W ₈	0.402	-	1.078	0.043	-
9	Self-Weight – W ₉	0.110	-	1.080	0.120	-
10	Self-Weight – W ₁₀	0.839	-	1.250	1.046	-
11	Self-Weight – W ₁₁	0.166	-	0.860	0.142	-
12	Self-Weight – W ₁₂	0.739	-	0.780	0.058	-
13	Self-Weight – W ₁₃	2.100	-	0.260	0.013	-
14	Self-Weight – W ₁₄	3.295	-	0.500	1.050	-
15	Up lift pressure – UP ₁	-	-	1.620	-	5.338
16	Up lift pressure – UP ₂	1.082	-	2.160	-	2.337
17	Weight of water – WW ₁	0.405	-	1.873	1.191	-
18	Water pressure – WP ₁	-	0.707	2.023	-	1.429
19	Water pressure – WP ₂	-	0.405	1.873	-	0.758
20	Silt pressure – SP	-	0.109	1.973	-	0.21

Summation of all Vertical Forces, $\sum F_v = 5.1685$ T.

Summation of all Horizontal Forces, $\sum F_h = 1.220$ T.

Summation of all Balancing Moments, $\sum M(r) = 16.16738$ T.M.

Summation of all Overturning Moments, $\sum M(o) = 10.0766$ T.M.

Summation of all Moments, $\sum M = 6.0907$ T.M.

Check for Stability

i. Check for Sliding

Factor of safety against sliding, $F.S/s = \mu \sum F_v / \sum F_h > 1.50$.

$$F.S/s = 0.65(5.1685)/1.22 = 2.75. >1.50. \text{----- Ok.}$$

ii. Check for Overturning

Factor of safety against overturning, $F.S/ot = \sum M(r) / \sum M(o) > 1.50.$

$$F.S/ot = 16.16738/10.0766 = 1.604 > 1.50. \text{----- Ok.}$$

iii. Check for Overstress

$$X = \sum M / \sum F_v = 6.0907/5.1685 = 1.1.178m.$$

$$\text{Eccentricity, } e = /1.178 - L/2/ = /1.178 - 3.24/2/ = 0.441m.$$

$$\text{Allowable Eccentricity, } e = L/6 = 3.24/6 = 0.54m.$$

$0.441 < 0.54.$ Hence, no tension will develop anywhere in the structure

4.1.7 Design of Stilling Basin

Design Information of Stilling Basin

Hydraulic Jump

Average Riverbed Elevation = 1861.30.

Weir Crest Level = 1862.20.

Downstream Water Level = 1862.0.

The bed level of stilling basin can be determined by trial-and-error method. Assume the floor level is 0.80m lower than the riverbed level.

Therefore, assume $EL_1 = EL_3 - 0.80m.$

$$= 1861.30 - 0.80.$$

$$= 1860.50.$$

Total energy head, $He = E_0 = E_1$

$$Z = EL_0 - \text{Assumed } EL_1$$

$$= 1862.20 - 1860.50.$$

$$= 1.70m.$$

$$H = He + Z$$

$$= 0.893 + 1.70.$$

$$= 2.593m.$$

Discharge intensity over the weir crest, $q = 1.875m^3/sec/m.$

$$E_0 = E_1 = d_1 + h_{v_1} = d_1 + v_1^2/2g$$

But $v_1 = q/d_1$.

By the principle of conservation of energy,

$$d_1 + v_1^2/2g = 2.593.$$

$$d_1 = 2.593 - 0.17918/d_1^2$$

d₁	2.593 – 0.17918/d₁²	Remark
0.270	0.135	
0.278	0.274	OK
0.280	0.307	

Therefore, $d_1 = 0.278\text{m}$.

$$V_1 = q/d_1 = 1.875/0.278 = 6.744\text{m/sec.}$$

$$\text{Froud Number, } Fr = v_1/\sqrt{(gd_1)} = 6.744/\sqrt{(9.81*0.278)} = 4.084.$$

$$d_2 = (d_1/2) \sqrt{[(1+8Fr^2) - 1]} = 1.473\text{m.}$$

If the water surface of the jump is 0.20m higher than the downstream water level,

$$EL = WL_3 + 0.20 = 1862.20 + 0.20 = 1862.40.$$

$$1862.40 - 1.473 = 1860.927 > 1860.500.$$

The sequent depth, d_2 is greater than the tail water depth by 0.573m. Hence, it is decided to depress the floor level to an elevation of 1860.727.

$$\begin{aligned} \text{Length of basin, } L &= 5(d_2 - d_1) \\ &= 5(1.473 - 0.278) \\ &= 5.975\text{m.} \approx 6.0\text{m.} \end{aligned}$$

Scour Depth

The depth of scour may be calculated from Lacey's Formula, as follows:

$$R = 1.35[q^2/f]^{1/3}, \text{ Where } R = \text{Depth of scour below the highest flood in meter.}$$

q = highest flood discharge of the river in m^3/sec per meter

length of the point of consideration.

f = silt factor = 1.

$$R = 1.35[(1.875)^2/ 1]^{1/3} = 2.05\text{m.}$$

- Upstream scour depth = $1.25*R = 2.566\text{m.} \approx 2.57\text{m.}$

- Downstream scour depth = $1.5*R = 3.075\text{m} \approx 3.08\text{m.}$

Exit Gradient

The factor of safety for exit gradient for soil types of our case (as of the geologic information of the weir axis) is selected to be 1/5.

The general formula adopted for exit gradient is

$$EG = (H/d) * 1/(\pi\sqrt{\lambda})$$

A graph based on Khosla's theory is placed to indicate the correction between the floor length, b and cut off depth, d i.e,

σ and $1/(\pi\sqrt{\lambda})$ to determine exit gradient.

$$\lambda = (1 + \sqrt{1 + \sigma^2})/2, \quad \sigma = b/d$$

for b=9.24 and d=1.00m.

$$\sigma = 9.24/1.00 = 9.24$$

For $\sigma = 9.24$ from the graph, $1/(\pi\sqrt{\lambda}) = 0.146$

H_{\max} – is for the static case and equal to 0.90m.

$$\begin{aligned} EG &= 0.90/1.00 * 1/(\pi\sqrt{\lambda}) \\ &= 0.90 * 0.146 \\ &= 0.1314 < 0.20 \text{-----safe.} \end{aligned}$$

Thickness of the floor

For the downstream apron the thickness to be determined depends on whether a static or dynamic case is being considered.

1. Static case:

Head on the structure

$$= 1862.200 - 1861.300$$

$$= 0.900\text{m}$$

2. Dynamic case

Head on the structure

$$= \text{Upstream TEL} - \text{Downstream TEL}$$

$$= 1863.093 - 1862.282$$

$$= 0.810\text{m}$$

The Maximum static head should be used for designing purposes.

Weighted creep length

According to Lane's method, weighted creep ratio is recommended in the form,

$$L > CH \text{ where } L: \text{ percolation distance (m.)}$$

$$H: \text{ maximum head, } = 0.90$$

$$C: \text{ Lane's creep ratio } = 3.20$$

$$L > 3.20 \times 0.90$$

$$L > 2.880$$

$$L = \sum l_v + 1/3 l_h \geq CH$$

$$L_c = (1.00 + .50 + 1.08 + 1.10) + (0.50 + 0.70 + 1.00 + 6) 1/3$$

$$= 3.68 + 2.733$$

$$= 6.413$$

Weighted creep ratio = $L_c / \text{head on the structure}$

$$= 6.413 / 0.90 = 7.220$$

Static case ($H_{\max} = 0.90$)

Point	Weight creep length			Hmax [1-LA/LC]	(TWL-WLA)	t(Safety factor/rm-1)
Case	H	V	L			
A	2.200	2.580	3.313	0.435	0.200	0.635
B	4.200	2.580	3.979	0.341	0.200	0.541
C	0.000	2.580	4.646	0.247	0.200	0.447
	Dynamic case ($H_{\max} = 0.81$)					
A	2.30	2.58	3.313	0.392	0.598	$0.989 \approx 1.000$
B		2.58		0.306	0.000	0.306
C		2.58		0.222	0.000	0.222

Checking the thickness at each point

Point A $1.00 > 0.989$ ----- Ok (dynamic case)

Point B $0.55 > 0.540$ ----- Ok (static case)

Point C $0.45 > 0.447$ ----- Ok (static case)

4.2. Surface Irrigation and Drainage System Design

In order to be in a position to select an irrigation system for a given area, it is important to look at the types of irrigation systems commonly used. Based on the method of applying water to the land, there are four broad classes of irrigation systems:

- 1) Surface irrigation systems,
- 2) Sprinkler irrigation systems,
- 3) Localized irrigation systems and
- 4) Sub-surface irrigation systems.

Surface irrigation is the application of water by gravity flow to the surface of the field. Either the entire field is flooded (basin irrigation) or the water is fed into small channels (furrows) or strips of land (borders).

In sprinkler irrigation systems, water is conveyed and distributed through pressurized pipe networks before being sprayed onto the land. There are several sprinkler irrigation systems, which can broadly be divided into set systems and continuous move systems.

In localized irrigation systems, a pipe distribution network is used to distribute and deliver filtered water (and fertilizer) to a predetermined point. The three main categories of localized irrigation methods are drip, spray and bubbler. More recently, drip irrigation systems have been developed whereby the laterals are buried in the root zone of the crop.

Sub-surface irrigation systems rely on the raising or lowering of the water table in order to effect groundwater flow to the root zone. As such, they are drainage flow systems.

4.2.1 Field Irrigation Method

Surface irrigation schemes are designed and operated to satisfy the irrigation water requirements of each field while controlling deep percolation, runoff, evaporation and operational losses. The performance of the system is determined by the efficiency with which water is conveyed to the scheme from the headworks, distributed within the scheme and applied to the field, and by the adequacy and uniformity of application in each field.

The classification of surface methods is perhaps somewhat arbitrary in technical literature. This has been compounded by the fact that a single method is often referred to with different names. Surface methods are classified by the slope, the size and shape of the field, the end conditions, and how water flows into and over the field. Each surface system has unique advantages and disadvantages depending on such factors like:

- 1) Initial cost;
- 2) Size and shape of fields;
- 3) Soil characteristics;
- 4) Nature and availability of the water supply;
- 5) Climate;
- 6) Cropping patterns;
- 7) Social preferences and structures;
- 8) Historical experiences; and
- 9) Influences external to the surface irrigation system.

Furrow irrigation, the most traditional method of surface irrigation, is recommended for this project, for the following reasons; it is suitable to the soil type of the project area; it has been traditionally exercised by the farmers of the project area; it is easily manageable at farmer's level; it is suitable to irrigate all crops, which are recommended for the project. Furthermore, furrow irrigation avoids flooding the entire field surface by channelling the flow along the primary direction of the field using 'furrows,' 'creases,' or 'corrugations'. Water infiltrates through the wetted perimeter and spreads vertically and horizontally to refill the soil reservoir. Furrows are often employed in basins and borders to reduce the effects of topographical variation and crusting. The distinctive feature of furrow irrigation is that the flow into each furrow is independently set and controlled as opposed to borders and basins where the flow is set and controlled on a border by border or basin by basin basis.

4.2.2 Proposed Water Distribution and Scheduling

Unit Flow

The water duty of the project is calculated as 2.16lit/sec/ha. By multiplying this figure by the unit area of land under each turnout, the unit flow is obtained.

Scheduling

The average irrigation interval (recommended by the agronomist) is 7 days. If we have to irrigate 60ha of land in seven days, therefore, we have to irrigate $60/7 = 8.57\text{ha/day}$. Therefore, the supply of water would be:

$$130\text{lt/sec}/8.57\text{ha} = 15.17\text{lt/sec/ha.}$$

In order to satisfy this condition, the command area is divided into 7 blocks with areas nearly equal to 8.57ha each, and turnouts with the same schedule of irrigation are identified by similar symbols (♠, ♣, ♥, etc). See the table 6 below.

Canal	Turnout No	Area (ha)	Discharge, (m ³ /sec)	Remark
Main Canal	T-1	0.45	0.97	♠
Main Canal	T-2	1.29	2.79	♠
Main Canal	T-3	1.07	2.31	♠
Main Canal	T-4	1.49	3.22	♠
Main Canal	T-5	0.42	0.91	♠
Main Canal	T-6	0.17	0.37	♠
Main Canal	T-7	2.03	4.38	♠
Main Canal	T-8	1.69	3.65	♠
Main Canal	T-9	1.47	3.18	♣
Main Canal	T-10	0.71	1.53	♣
Main Canal	T-11	0.32	0.69	♣
Main Canal	T-12	1.16	2.51	♣
Main Canal	T-13	2.02	4.36	♣
Main Canal	T-14	3.54	7.65	♣
Main Canal	T-15	3.58	7.73	♥
Main Canal	T-16	2.5	5.4	♥
Secondary Canal-1	T-1,1	1.5	3.24	♥
Secondary Canal-1	T-1,2	1.98	4.28	♦
Secondary Canal-1	T-1,3	0.82	1.77	♥
Secondary Canal-1	T-1,4	1.29	2.79	♦
Secondary Canal-1	T-1,5	0.69	1.49	♦
Secondary Canal-1	T-1,6	0.65	1.4	♦
Secondary Canal-1	T-1,7	0.55	1.19	♦
Secondary Canal-1	T-1,8	0.53	1.15	♦
Secondary Canal-1	T-1,9	0.91	1.97	♦
Secondary Canal-1	T-1,10	0.37	0.8	♦

Secondary Canal-1	T-1,11	0.55	1.19	◆
Secondary Canal-2	T-2,1	0.17	0.37	◆
Secondary Canal-2	T-2,2	0.23	0.5	◆
Secondary Canal-2	T-2,3	0.51	1.1	◆
Secondary Canal-2	T-2,4	0.48	1.04	▲
Secondary Canal-2	T-2,5	0.2	0.43	▲
Secondary Canal-2	T-2,6	0.35	0.76	▲
Secondary Canal-2	T-2,8	0.27	0.58	▲
Secondary Canal-2	T-2,9	0.39	0.84	▲
Secondary Canal-2	T-2,10	0.43	0.93	▲
Secondary Canal-2	T-2,11	1.99	4.3	▲
Secondary Canal-2	T-2,12	0.59	1.27	▲
Secondary Canal-2	T-2,13	0.97	2.1	▲
Secondary Canal-2	T-2,14	0.67	1.45	▲
Secondary Canal-2	T-2,15	0.85	1.84	▲
Secondary Canal-2	T-2,16	1.93	4.17	■
Secondary Canal-2	T-2,17	0.62	1.34	▲
Secondary Canal-2	T-2,18	2.29	4.95	■
Secondary Canal-2	T-2,19	1.95	4.21	■
Secondary Canal-2	T-2,20	2.26	4.88	■
Secondary Canal-2	T-2,21	1	2.16	☹
Secondary Canal -3	T-3,1	1.48	3.2	☹
Secondary Canal -3	T-3,2	1.73	3.74	☹
Secondary Canal -3	T-3,3	1.36	2.94	☹
Secondary Canal -3	T-3,4	1.42	3.07	☹
Secondary Canal -3	T-3,5	1	2.16	☹

Table 6: List of turnouts available in the project

4.2.3 Water Duty

The maximum total irrigation water requirement (for Onion) is calculated as 6,760m³/ha for the growing period of 5 months. But the maximum irrigation water requirement (in the month of March) is 2,340m³/ha. This figure is used to calculate the water duty of the project.

Taking the farmers past experience in to consideration, 10 hours irrigation per day is considered to calculate the water duty.

Hence, the water duty would be $2340\text{m}^3/\text{ha} \times 10^3\text{lt}/\text{m}^3/\text{month} = 234 \times 10^4\text{lt}/\text{ha}/30 \times 24 \times 60 \times 60 = 0.9\text{lt}/\text{sec}/\text{ha}$. This figure is for 24 hours irrigation.

Therefore, for 10 hours irrigation, $Q = 0.9 \times 24/10 = 2.16\text{lt}/\text{sec}/\text{ha}$. This figure satisfies all of the crops recommended for this project.

4.2.4 Main Canal Design

The main canal has a total length of 3.54km. Of this length, 12.00m is closed conduit (diameter of pipe = 50cm), 38.00m is box canal (a box with masonry lining), 65.00m is box conduit (with cover slab), 524.50m is line canal (masonry lining), and 2900.50m is earthen canal with a longitudinal slope of 1/560. The Manning's Equation is used to design the canal.

The main canal is a contour canal throughout its length unless otherwise unavoidable obstacles are reached, it is tried to maintain a constant depth of cut of 0.60m. The new canal joins the old main canal at a chainage of 2+750. Drop structures are provided on the canal after it joins the old main canal so as to maintain the alignment of the later.

Dry discharge of Jalele River is estimated as 125lt/sec (by floating method). Releasing 20lt/sec. For downstream users, the can portion from the head structure up to Leku River is designed for a discharge of 105lt/sec.

$$Q = 105\text{lt}/\text{sec}$$

$$S = 1/560 \text{ (for the carton part of the canal)}$$

$$n = 0.025$$

$$b:d = 1:1$$

Canal side slope = 1:1 (for the earthen part)

$$A = (b+T)/(2) \cdot d = (b+b+b+b)/(a) \cdot b = 4b^2/2 = 2b^2 = 2d^2$$

$$P = 2\sqrt{b^2+b^2+b} = 2\sqrt{2b^2+b} = 2b\sqrt{2+b} = 2d\sqrt{2+d}$$

$$R = A/P = 2d^2/d(2\sqrt{2+1}) = 2/(2\sqrt{2+1}) \cdot d$$

$$Q = VA, V = 1/n R^{2/3} S^{1/2} \cdot A$$

$$= 1/0.025 \times 2/(2\sqrt{2+1})^{2/3} (1/560)^{1/2} \cdot A$$

$$= 1.096 \cdot d^{2/3} \cdot 2d^2$$

$$0.13 = 2 \times 1.09 d^{8/3}$$

$$D = 0.347 \approx 0.35\text{m}$$

Free board, $f = 0.25\text{m}$

$$\Rightarrow D = 0.60\text{m}$$

$$\Rightarrow A = 0.24\text{m}^2, P = 1.327\text{m}, R = 0.181\text{m}$$

$$V = 1/0.025 (0.181)^{2/3} (1/560)^{1/2} \\ = 0.54\text{m/sec}$$

Design of Lined Canal

$$(0+12+0+50)$$

$$Q = 130\text{lt/sec}$$

$$S = 0.1/38 = 1/380$$

$$N = 0.017$$

$$B:d = 1$$

$$A = bd = 1.5d^2$$

$$P = 2d + 1.5d = 3.5d$$

$$R = A/P = 1.5d^2/3.5d = 0.429d$$

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

$$0.13 = 1.5d^2/0.017 (0.429d)^{2/3} (1/380)^{1/3}$$

$$0.051 = d^{8/3}$$

$$\Rightarrow d = 0.33\text{m}$$

$$\Rightarrow b = 1.5d = 0.49\text{m} \approx 0.50\text{m}$$

$$Q = VA \Rightarrow V = Q/A = 0.13/0.33 \times 0.49 = 0.80\text{m/sec}$$

Let free board, $f = 0.17\text{m}$

$$\Rightarrow D = 0.50\text{m}$$

$$0+50-0+115$$

Box contract (Box with cover slab)

$$Q = 130\text{lt/sec}$$

$$S = 1/25$$

$$n = 0.017$$

$$b:d = 1.5$$

$$A = bd = 1.5d^2$$

$$R = 0.429d$$

$$Q = a/n R^{2/d} \Rightarrow 0.13 \cdot 1.5d^2 / 0.017 (0.429d)^{2/3} (1/250)^{1/2}$$

$$\Rightarrow 0.041 = d^{8/3}$$

$$\Rightarrow d = 0.30\text{m}$$

$$\Rightarrow b = 1.5d = 0.45\text{m}$$

$$Q = VA \Rightarrow V = Q/A = 0.13 / 0.30 \times 0.45 = 0.96\text{m/sec}$$

Let free board, $f = 0.20\text{m}$

$$\Rightarrow D = 0.50\text{m}$$

Design of Lined canal (629-1125, 1575-1610.50)

$$Q = 130\text{lt/sec}$$

$$S = 1/560$$

$$n = 0.017$$

$$b:d = 1.5$$

$$b = 1.5d$$

$$A = bd = 1.5d^2$$

$$P = 2d + 1.5d = 3.5d$$

$$R = A/P = 1.5d^2 / 3.5d = 0.429d$$

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

$$0.13 = 1.5d^2 / 0.017 (0.429d)^{2/3} (1/560)^{1/2}$$

$$\Rightarrow d = 0.35\text{m}$$

$$\Rightarrow b = 0.52\text{m}$$

Free board, $f = 0.15\text{m}$

$$\Rightarrow \text{Total depth of lined canal} = 0.50\text{m}$$

$$Q = VA \Rightarrow V = Q/A = 0.13 / 0.35 \times 0.52$$

$$= 0.72\text{m/sec}$$

4.2.5 Secondary Canals Design

Three secondary canals, namely secondary canal -1, 2 & 3 are designed for the scheme. The points at which these canals depart from the main canal and their respective lengths are shown on table 7.

No	Type of secondary Canal	Length (km)	Point of Departure from the main canal
1	Secondary Canal -1	0.800	1+750
2	Secondary Canal -2	2.581	2+345
3	Secondary Canal -3	0.636	2+870

Table 7: List of secondary canals location and length

Towards the beginning of them, all of the secondary canals run on steeply slope. Therefore, they are equipped with chute structures. And secondary canals 1 and 2 have got chute structures at the middle of their lengths due to the unavoidable steeply slope which they run through.

The Manning Equation is used to design the secondary canals, too. The hydraulic parameters of the canals are shown on the following sketches.

Design of Secondary Canal-1

Secondary canal 2 and 3 are also designed in the same way as secondary canal 1.

Design slope, $S = 1/400$

$n = 0.025$

$b:d = 1$

Canal side slope = 1

$$A = (b + T)/(2) d = 2d^2$$

$$P = 2\sqrt{2b^2 + b} = 2b\sqrt{2+6} = 2d\sqrt{2+d}$$

$$R = A/P = 2/2\sqrt{2+1} d$$

$$Q = VA \quad V = 1/n R^{2/3} S^{1/2}$$

$$= 1.30d^{2/3}$$

$$0.02125 = 2d^2 (1.3d^{2/3})$$

$$= 2.50 d^{8/3}$$

$\Rightarrow d = 0.17m$. Make $b = 0.20m$ for the sake of ease of construction

$F_b = 0.13m$

$$\Rightarrow D = 0.30\text{m}$$

$$R = \frac{2}{2r_2 + 1} d = 0.089\text{m}$$

$$V = \frac{1}{0.025} (0.089)^{2/3} (1/400)^{1/2} = 0.40\text{m/sec}$$

Lined canal on secondary canal-1

$$(0+650-0+800)$$

Length = 150m

$$\text{Slope} = \frac{1805.50 - 1799.7}{150} = 0.0387 \approx 3.87\%$$

$$Q = 0.02125\text{m}^3/\text{sec}$$

$$n = 0.017$$

$$b = 2d$$

$$A = bd = 2d^2$$

$$P = 2d + d = 4d$$

$$R = \frac{A}{P} = \frac{2d^2}{4d} = \frac{d}{2}$$

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

$$0.02125 = \frac{2d^2}{(0.017)} \left(\frac{d}{2}\right)^{2/3} (0.0387)^{1/2}$$

$$0.00146 = d^{8/3}$$

$$D = 0.086\text{m}$$

$$\Rightarrow b = 2d = 0.17\text{m}$$

$$\text{Free board, } f_b = 0.114\text{m}$$

$$\Rightarrow D = 0.20\text{m}$$

4.2.6 Field Canals Design

The amount of water supplied to the field through turnouts is very small (not more than 4.88lt/sec). Therefore, it is believed that the farmer's furrow field canals without any slope-adjusting structures are sufficient for this purpose.

4.2.7 Canal Structures Design

A. Drop Structures

There are eight (8) vertical drops of 1.0m height (USBR Type) on the main canal. These are provided to make the slope of the old main canal compatible with our design slope.

- On Secondary canal-1, there is one drop of 0.50m height.

- On secondary canal –2, there are eight drops of 0.80m height, two drops of 0.10m height and one drops of 0.50m height, which amounts to the total of eleven drops.
- On secondary canal –3, there are four drops of 0.80m height. All of the drops are vertical drops (USBR type).

Design of vertical drops (H=1m) (Main canal)

Remaining drops are done in the same manner on main and secondary canals.

Design discharge, $Q= 0.130\text{m}^3/\text{sec}$

Height of drop, $H= 1.0\text{m}$

Width of drop, $b_c = 0.734Q/(d_1)^{3/2} = 0.734(0.13)/(0.313)^{3/2} = 0.55\text{m}$

Unit discharge, $q = Q/b_c = 0.236\text{m}^3/\text{sec}/\text{m}$

Critical depth, $d_c = [q^2/q]^{1/3} = 0.18\text{m}$

Stilling basin

Lip height, $a= d_c/\partial = 0.09\text{m}$ use $a= 0.15\text{m}$

Length, $L = [2-5+1.1d_c/H+0.7(d_c/H)^3]\sqrt{Hd_c} = 1.15\text{m} \approx 1.2\text{m}$

Width , $B= 18.46\sqrt{Q/Q+9.91} = 0.66\text{m} = 0.70\text{m}$

B. Culverts

The culverts on the main and secondary canals with their structural dimension are described by table 8

No	Canal	Length of culvert (m)	Pipe Diameter (cm)	Number of culverts
1	Main canal	6	40	4
2	Main canal	5	40	7
3	SC-1	6	20	1
4	SC-2	5	30	6
5	SC-3	6	20	1

Table 8: List of culverts on main and secondary canals

Design of 5 m culvert-Main Canal

Remaining culverts on main and secondary canals are designed in the same.

Design velocity in canal = 0.535m/sec

Pipe diameter = 0.40m

Area of pipe, $H/r = 1.6 \Rightarrow A/r = 2.694 \Rightarrow A = 0.10776\text{m}^2$

Velocity of water in pipe = $Q/A = 0.130/0.10776 = 1.21\text{m/sec}$

Velocity head in pipe = $V^2/2g = 0.075\text{m}$

Wetted perimeter, $H/r = 1.6 \Rightarrow P/r = 4.428 \Rightarrow P = 0.8856\text{m}$

Hydraulic radius, $H/r = 1.6 \Rightarrow R/r = 0.608 \Rightarrow R = 0.1216\text{m}$

Value of $n = 0.018$

Frictional loss in pipe, $h_f = (Vn/R^{2/3})^2 \times L$

$$H = 0.8D$$

$$= 0.32\text{m}$$

$$H/r = 0.32/0.2 = 1.6$$

$$\Rightarrow P = 0.8856\text{m}$$

$$H/r = 1.6 \Rightarrow R/r = 0.608$$

$$\Rightarrow R = 0.1216\text{m}$$

$$\Rightarrow h_f = [0.46 \times 0.018 / (0.23)^{2/3}]^2 \times 5 = 0.04\text{m}$$

Water surface change at the u/s 8 d/s of culvert

Drop of water surface at the inlet of the culvert

$$Dh_1 = 1.5(V_2^2 - V_1^2/2g) = 0.09\text{m} \approx 0.10\text{m}$$

Rise of water surface at d/s of canal section

$$Dh_2 = 0.3(V_2^2/2g) = 0.018\text{m} \approx 0.02\text{m}$$

C. Turnouts

A free pipe outlet is designed. The discharge can be computed by using the equation

$Q = C_d A \sqrt{2gH_o}$, where

C_d = Coefficient of discharge = 0.62

H_o = Head on upstream side measured from FSL of distributary up to center of pipe outlet.

A = Area of cross section of pipe.

The calculation is made for the maximum area of land served by a single turnout, and this one is adopted for the rest of the outlets. The maximum area of land served by a single turnout is 3.58ha (T-15)

$$\Rightarrow q = \text{Duty} \times 3.58 = 2.16\text{lt/sec/ha} \times 3.58\text{ha}$$

$$= 7.73\text{lt/sec}$$

Let $H_o = 17\text{cm} = 0.17\text{m}$ (for outlets on the main canal)

$$7.73 \times 10^{-3} = 0.62A\sqrt{2 \times 9.81 \times 0.17}$$

$$\Rightarrow A = 6.83 \times 10^{-3} \text{ m}^2$$

$$\Rightarrow Y = 0.05\text{m.}$$

Therefore, a pipe out let (concrete pipe) of diameter 10cm would be used moreover; a check structure of 5.10cm height is arranged in front of every outlet in order to assure continuous and sufficient supply of water into the field. The discharge of the off taken would be controlled by wooden/steel gates of specified dimensions.

Secondary canals-1 and 3 take off by a pipe of diameter 15 cm, while secondary canal-2 takes off by a pipe of diameter 20cm.

The following table (table 9) describes the location of turnouts on each type of canal (i.e., main and secondary canals).

No	Canal	Turnout No	Location
1	MC	SC-1	1+075
2	MC	SC-2	2+345
3	MC	SC-3	2+870
4	MC	T-1	0+350
5	MC	T-2	0+425
6	MC	T-3	0+525
7	MC	T-4	1+175
8	MC	T-5	1+275
9	MC	T-6	1+950
10	MC	T-7	2+095
11	MC	T-8	2+300
12	MC	T-9	2+400
13	MC	T-10	2+545
14	MC	T-11	2+600
15	MC	T-12	2+900
16	MC	T-13	3+100
17	MC	T-14	3+300
18	MC	T-15	3+400

No	Canal	Turnout No	Location
29	SC-1	T-1,10	0+700
30	SC-1	T-1,11	0+800
31	SC-2	T-2,1	0+225
32	SC-2	T-2,2	0+325
33	SC-2	T-2,3	0+425
34	SC-2	T-2,4	0+475
35	SC-2	T-2,5	0+575
36	SC-2	T-2,6	0+725
37	SC-2	T-2,7	0+875
38	SC-2	T-2,8	1+025
39	SC-2	T-2,9	1+075
40	SC-2	T-2,10	1+100
41	SC-2	T-2,11	1+175
42	SC-2	T-2,12	1+30
43	SC-2	T-2,13	1+375
44	SC-2	T-2,14	1+485
45	SC-2	T-2,15	1+656
46	SC-2	T-2,16	1+906

19	MC	T-16	3+523	47	SC-2	T-2,17	2+006
20	SC-1	T-1,1	0+175	48	SC-2	T-2,18	2+181
21	SC-1	T-1,2	0+325	49	SC-2	T-2,19	2+256
22	SC-1	T-1,3	0+450	50	SC-2	T-2,20	2+331
23	SC-1	T-1,4	0+450	51	SC-2	T-2,21	2+431
24	SC-1	T-1,5	0+525	52	SC3	T-3,1	0+211
25	SC-1	T-1,6	0+525	53	SC3	T-3,2	0+286
26	SC-1	T-1,7	0+600	54	SC3	T-3,3	0+386
27	SC-1	T-1,8	0+600	55	SC3	T-3,4	0+486
28	SC-1	T-1,9	0+700	56	SC3	T-3,5	0+586

Table 9: Location of Turnouts

D. Chutes

Towards their beginning and at some of their middle chainage, the secondary canals run through profiles of steeply slopes. Therefore, chute structures are provided to these canals at these steeply slope profiles. The lengths and chainages of the chute structures on the secondary canals are described by the following table (table 10):

No.	Canal	Chute No.	Location	Length (m)
1	SC-1	Ch-1,1	0+000-0+075	75
2	SC-1	Ch-1,2	0+275-0+320	45
3	SC-1	Ch-1,3	0+400-0+450	50
4	SC-1	Ch-1,4	0+450-0+525	75
5	SC-1	Ch-1,5	0+525-0+600	75
6	SC-1	Ch-1,6	0+600-0+650	50
7	SC-2	Ch-2,1	0+000-0+125	125
8	SC-2	Ch-2,2	0+125-0+169	44
9	SC-2	Ch-2,3	0+430-0+457	27
10	SC-2	Ch-2,4	0+475-0+525	50
11	SC-2	Ch-2,5	0+575-0+625	50
12	SC-2	Ch-s,6	0+575-0+625	50
13	SC-2	Ch-2,7	0+625-0+720	95
14	SC-2	Ch-2,8	0+725-0+750	25
15	SC-2	Ch-2,9	1+220-1+300	80
16	SC-2	Ch-2,10	1+300-1+341	41

17	SC-3	Ch-3,1	0+000-0+061	61
18	SC-3	Ch-3,2	0+061-0+136	75
19	SC-3	Ch-3,3	0+136-0+211	75

Table 10: Location and length of chutes of secondary canals

Design of chute 1, 1 (secondary canal 1) (D+00-0+75)

Remaining chutes are designed in the same way as this one.

Discharge, $Q = 0.02125 \text{ m}^3/\text{sec}$

Width of notch, $b_c = 0.734Q/(d\partial)^{3/2} = 0.22 \text{ m}$

Unit discharge, $q = Q/b_c = 0.096 \text{ m}^3/\text{sec}/\text{m}$

Critical depth, $d_c = [q^2/q]^{1/3} = 0.098 \text{ m}$

Critical velocity, $A_c = q/d_c = 0.98 \text{ m}/\text{sec}$

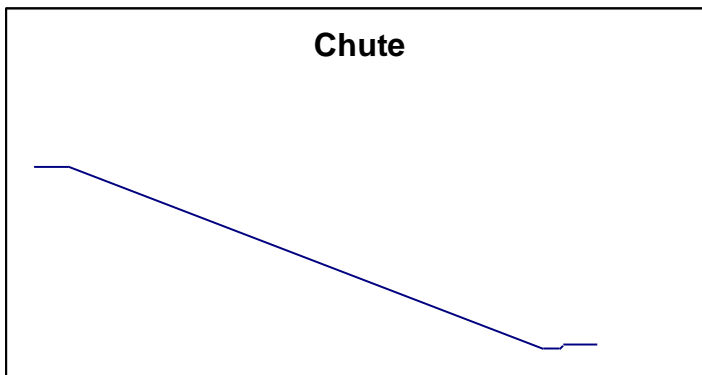
Velocity head, $h_{vc} = V_c^2/2g = 0.049 \text{ m}$

Water area, $A_c = b_c d_c = 0.0216 \text{ m}^2$

Wetted perimeter, $P_c = b_c + 2d_c = 0.416 \text{ m}$

Hydraulic radius, $R_c = A_c/P_c = 0.052 \text{ m}$

Water surface, $I_c = [(n^{v_c}) / (R_c)^{2/3}]^2 = 0.0143$



Energy calculation

At section (c)

$F = EL.A - EL.B = 1858.13 - 1840.10 = 18.03 \text{ m}$

$E_c = d_c + h_{vc} + F = 0.098 + 0.049 + 18.03 = 18.177 \text{ m}$

Energy at section (C)

Designation	Results of the calculation		
	1	2	3
Trial Number	1	2	3
Assumed depth, d_1	0.05	0.03	0.02975
$b_1 = b_c$	0.22	0.22	0.22
$A_1 = b_1 d_c$	0.011	0.0066	0.006545
$V_1 = Q/A_1$	1.932	3.22	3.247
$H_{v1} = V_1^2/2g$	0.19	0.528	0.537
$P_1 = b_1 + 2d_1$	0.32	0.28	0.2795
$R_1 = A_1/P_1$	0.0344	0.0236	0.0234
$I_1 = (n^2 V_1^3 / R_1^2)^2$	0.0964	0.443	0.455
$I_m = (I_c + I_1)/2$	0.0554	0.229	0.235
$H_{fl} = I_m \cdot L$	4.152	17.159	17.589
$E_1 = 18.177$	4.392	17.718	18.156

Froude Number, $Fr = \sqrt{V_1/gd_1} = \sqrt{[3.247/(9.81*0.02975)]} = 6.01$

Conjugate Depth, $d_2 = \{(d_1/2)*[\sqrt{(1+8Fr^2)} - 1]\} = 0.24m$

Stilling Basin

Length, $L = 4d_2 = 0.96m \approx 1m$.

Width, $B = 18.4b\sqrt{Q/Q+9.91} = 0.27m$

Bottom Elevation (E.L.C)

$V_2 = q/d_2 = 0.096/0.24 = 0.4m/sec$

$H_{v2} = V_2^2/2g = 0.0082m$

$E_2 = d_2 + h_{v2} = 0.25m$

$a = E_2 - E_3 = 0.25 - 0.174 = 0.08m = 0.10m$

Design of chute 2, 1 (0+00-0+125)

$Q = 0.04161m^3/sec$

Width of notch, $bc = 0.734Q/(d_0)^{3/2} = 0.30m$

$Q = Q/b_c = 0.141m^3/sec/m$

$$d_c = (q^2/g)^{1/3} = 0.126\text{m}$$

$$V_c = q/d_c = 1.12\text{m/sec}$$

$$h_{v_c} = V_c^2/2g = 0.064\text{m}$$

$$A_c = b_c d_c = 0.0378\text{m}^2$$

$$P_c = b_c + 2d_c = 0.552\text{m}$$

$$R_c = A_c/P_c = 0.069\text{m}$$

$$I_c = [n v_c / (R_c)^{2/3}]^2 = 0.0129$$

Energy calculation

$$F = 1855.19 - 1837.40 = 17.79\text{m}$$

$$E_c = d_c + h_{v_c} + F = 17.98\text{m}$$

Energy at section (1)

D_1 is solved by trial and error as

$$D_1 = 0.0437\text{m}$$

$$\Rightarrow V_1 = 3.174\text{m/sec}$$

$$F_r = V_1 / \sqrt{g d_1} = 4.848$$

$$D_2 = d_1/2 (\sqrt{1+8F_r^2} - 1) = 0.28\text{m}$$

Stilling basin, $L = 4d_2 = 1.12\text{m} \approx 1.20\text{m}$

$$B = (18.46) \sqrt{Q/(Q+9.91)} = 0.38$$

Bottom elevation (E.L.C)

$$V_2 = q/d_2 = 0.141/0.28 = 0.504\text{m/sec.}$$

$$h_{v_2} = V_2^2/2g = 0.013\text{m}$$

$$E_2 = d_2 + h_{v_2} = 0.293\text{m}$$

$$E^3 = d_3 + h_{v_3} = 0.22 + (0.468)^2/2 \times 9.81 = 0.23\text{m}$$

$$A = E_2 - E_3 = 0.63\text{m, make } a = 0.10\text{m}$$

4.2.8 Cross Drainage Structures Design

As it was mentioned above, the main canal is contour canal throughout its length. The drainage water, which comes from uphill, will be intercepted by interceptor drain and it will be safely passed down wards through cross drainages structures. The same arrangement of cross drainages is performed for secondary canals, too.

The cross-drainage arrangement is in such a way that the irrigation water will be passing through a culvert of 5m length, while the drainage water will be passing over the culvert. The design calculation of a 5m culvert which had been performed for a crossing culvert of the same length will apply for this case.

The location of cross drainage structure on each canal is described by the following table (table 11).

No.	Canal	Cross Drainage No.	Location
1	Main canal	CD-1	0+800
2	Main canal	CD-2	2+500
3	Secondary Canal ~ 2	CD-2,1	0+350
4	Secondary Canal ~ 2	CD-2,2	0+875
5	Secondary Canal ~ 2	CD-2,3	1+670
6	Secondary Canal ~ 2	CD-2,4	2+097

Table 11: Cross drainage structures and their locations

4.2.9 Drainage Canals Design

The alignment of field drains and main drain is shown on the irrigation system layout. The water drained out of the field, i.e., water which is surplus of the irrigation requirement, is very small, and hence it is not necessary to design a field drain. The farmers' furrows like drains are believed to be able to safely drain out the surplus water out of the field.

On the other hand, the main drain is aligned along natural drainage lines of the area. Therefore, these too do not require special design procedure.

4.2.10 Catch Drains

As it is already stated above, the main canal runs along a hill side of a ridge, i.e. it is a contour canal. Therefore, a catch drain facility should be arranged in order to safe guard the canals and the command area against flood hazards of the uphill land.

5. RESULTS AND DISCUSSIONS

Surface irrigation scheme is designed and operated to satisfy the irrigation water requirements of each field while controlling deep percolation, runoff, evaporation and operational losses. The performance of the system is determined by the efficiency with which water is conveyed to the scheme from the headworks, distributed within the scheme and applied to the field, and by the adequacy and uniformity of application in each field. A furrow irrigation system consists of furrows and ridges. The water is applied by means of small channels or furrows, which follow a uniform longitudinal slope. The method is best suited to row crops proposed on the project such as maize, potatoes, onions, tomatoes.

The project consists of 7,567m length of canal of which 3,540m is main canal (contour canal) and the remaining is secondary canals. Concerning main canal 12m is closed conduit, 38m is box canal, 65m is box conduit, 524.50m is masonry lined canal, and 2900.50m is earthen canal. For both main and secondary canals designing, Manning's Equation is used.

On main canal, a change of ground level encountered at eight different locations that necessitates the provision of eight vertical drops (falls) to lower down its bed level to maintain the designed slope. Eleven drops of different heights are provided on secondary canals to keep the designed slope of the canals. Towards their beginning and at some of their middle chainages, the secondary canals run through profiles of steeply slopes. Therefore, chutes are required to reduce the bottom slope of canals lying on steeply sloping land in order to avoid high velocity of the flow and risk of erosion. Out of nineteen chutes of different length, six are located on secondary canal one, ten on secondary canal two and three on secondary canal three.

Turnouts are provided to divert irrigation water from main and secondary canals to field. The calculation is made for the maximum area of land served by a single turnout, and this one is adopted for the rest of the outlets. The maximum area of land served by a single turnout is 3.58ha (T-15). Therefore, a pipe outlet of diameter 10cm would be used. Moreover, a check structure of 5.10cm height is arranged in front of every outlet in order to assure continuous and sufficient supply of water into the field. The discharge of the offtake would be controlled by wooden/steel gates.

Water is diverted from the tertiary canal into furrows by means of siphons placed over the side of the ditch or canal bank and be allowed to flow downstream along the furrow. The water level in the canal must be raised to a sufficient height above the level of the furrows by using a piece of wood, check plates, or canvas filled with sand. This creates a head difference between the water level in the field ditch and the furrow, which is necessary for the water flow. The water is gradually absorbed into the soil and spreads laterally to wet the area between the furrows. With furrow irrigation, water is mainly lost by deep percolation at the head end of furrow and runoff at the tail end.

Furrow design is an iterative process that should consider the shape of the furrow, the spacing between furrows, with the furrow length determined, amongst other factors, by the stream size to apply and its application time, the soil type and the slope.

Concerning the design of weir that includes the design of stilling basin, the geology of the weir site as well as the catchment of the project dictate the type of weir to be ogee. That is to say, alluvial deposits from the size of boulders up to small sized gravel around the riverbed is one of the factors. Furthermore, big boulders are coming from upstream mountainous area that need smooth passage from upstream to downstream. In this regard, ogee weir is the best option and that is why ogee weir is selected and designed.

Average river bed slope around the weir site, $I_{av} = H_{av}/L = (2 \times 195.525\text{m}^2)/94 = 4.16\text{m}/94\text{m} = 0.04425 = 4.425\%$. The water level is obtained from stage ~ discharge curve. After preparing stage ~ discharge curve (i.e., plotting discharge versus depth), the tail water depth (depth of water during the expected maximum flow) is estimated as 0.90m. For stage ~ discharge curve, the discharge was calculated by using Manning equation, $Q = (A/n) \cdot (R^{2/3} I^{1/2})$. Thus, high flood level of the river (HFL) before construction of weir structure is; $\text{HFL} = 1861.30\text{m} + 0.90\text{m} = 1862.20\text{m}$

Expected maximum flow for Jalele catchments is estimated by Rational Method $Q_p = 0.00216 \text{CIA}^{0.73} = 0.00216 * 0.63 * 96\text{mm} * (1700\text{ha})^{0.73} = 29.8 \text{ m}^3/\text{s} \approx 30\text{m}^3/\text{sec}$, based on field data observation, i.e., land slope classification, coverage of the area, and value of runoff coefficient, C.

The crest length of the ogee weir is decided to be 16.00m from both observation and experience point of views.

Ogee weir crest level was determined from average elevation of the highest field in the command area. The highest field elevation in the command area is 1860.70m. Total head loss is the

summation of head loss across head regulator, along main canal, at turnout, at entrance and exit. The value of total head loss = 1.50m; is from

- ✓ Water depth required = 0.31m.
- ✓ Head loss across head regulator = 0.15m.
- ✓ Head loss along the main canal = 0.807m.
- ✓ Head loss at turnout = 0.05m.
- ✓ Entrance, friction and exit loss = 0.0803m.
- ✓ Free board = 0.10m.

Since average river bed elevation = 1861.30m and total head loss = 1.50m, ogee weir crest level which is the summation of highest field in the command and total head loss is equal to 1862.20 = 1860.70 + 1.50. Thus, ogee weir crest height was determined by taking the difference between ogee crest level and average river bed level (i.e., $h = 1862.20 - 1861.30 = 0.90\text{m}$).

The equation of ogee-shaped weirs as per the Water Ways Experiment Station is expressed as:

$$X^n = K_0 H_d^{n-1} Y,$$

Where:

X and Y are co-ordinates of the crest profile with the origin at the highest point of the crest,

H_d is the design head excluding the head due to the velocity of approach and,

K_0 and n are parameters depending on the slope of the upstream face.

For Jalele ogee weir, the upstream face has a slope of 3 Vertical to 2 Horizontal (3V:2H). This implies that

$$K_0 = 1.939$$

$$n = 1.810.$$

After substitution, we get the following formula for Jalele weir crest geometry;

$$Y = -X^{1.810}/1.5937$$

The shape of the weir is determined by assigning different values to co-ordinate X and calculating the corresponding co-ordinate, Y.

$Y = -X^{1.810}/1.5937$		
Serial Nr.	X	$Y = -X^{1.810}/1.5937$
1	0.000	0.000
2	0.250	-0.051
3	0.500	-0.179
4	0.750	-0.372

5	1.000	-0.627
6	1.225	-0.905

Since the toe of the weir should be connected to the base by a circular curve, the radius of the base bucket $R_0 = 1.528\text{m}$. The center of gravity of the ogee geometry, $(X, Y) = (0.451, -0.550)$

The stability of the ogee weir structure was analyzed from sliding, overturning and overstress point of view under both static and dynamic conditions. The outcomes show that the ogee weir is safe against sliding, overturning and overstress.

The bed level of stilling basin is determined by trial-and-error method. The hydraulic jump, d_1 is computed by the principle of conservation of energy and the value is equal to 0.278.

d_1	$2.593 - 0.17918/d_1^2$	Remark
0.27	0.135	
0.278	0.274	Almost equal (OK)
0.28	0.307	

By using Froud Number, $Fr = v_1/\sqrt{(gd_1)} = 6.744/\sqrt{(9.81*0.278)} = 4.084$.

$$d_2 = (d_1/2) \sqrt{[(1+8Fr^2) - 1]} = 1.473\text{m}.$$

If the water surface of the jump is 0.20m higher than the downstream water level,

$$EL = WL_3 + 0.20 = 1862.20 + 0.20 = 1862.40.$$

$$1862.40 - 1.473 = 1860.927 > 1860.500.$$

The sequent depth, d_2 is greater than the tail water depth by 0.573m. Hence, it is decided to depress the floor level to an elevation of 1860.727.

$$\begin{aligned} \text{Length of basin, } L &= 5(d_2 - d_1) \\ &= 5(1.473 - 0.278) \\ &= 5.975\text{m} \approx 6.0\text{m}. \end{aligned}$$

Scour depth, exit gradient and thickness of the floor under static and dynamic cases

Scour depth

The depth of scour was calculated from Lacey's Formula as:

$$R = 1.35[q^2/f]^{1/3},$$

Where R = Depth of scour below the highest flood in meter.

q = highest flood discharge of the river in m³/sec per meter length of the point of consideration.

f = silt factor = 1.

$$R = 1.35[(1.875)^2 / 1]^{1/3} = 2.05\text{m.}$$

- ✓ Upstream scour depth = 1.25R = 2.566m. \approx 2.57m.
- ✓ Downstream scour depth = 1.5R = 3.075m \approx 3.08m.

Exit Gradient

The factor of safety for exit gradient for soil types of our case (as of the geologic information of the weir axis) is selected to be 1/5.

The general formula adopted for exit gradient is

$$EG = (H/d) * 1/(\pi \sqrt{\lambda})$$

A graph based on Khosla's theory is placed to indicate the correction between the floor length, b and cut off depth, d i.e.,

δ and $1/(\pi \sqrt{\lambda})$ to determine exit gradient.

$$\lambda = (1 + \sqrt{1 + \delta^2}) / 2, \quad \delta = b/d$$

for b = 9.24 and d = 1.00m.

$$\delta = 9.24/1.00 = 9.24$$

For $\delta = 9.24$ from the graph, $1/(\pi \sqrt{\lambda}) = 0.146$

H_{max} – is for the static case and equal to 0.90m.

$$EG = 0.90/1.00 * 1/(\pi \sqrt{\lambda})$$

$$= 0.90 * 0.146$$

$$= 0.1314 < 0.20 \text{ ----- safe.}$$

Thickness of the floor

For the downstream apron, the thickness to be determined depends on whether a static or dynamic case is being considered.

1. Static case:

$$\begin{aligned} &\text{Head on the structure} \\ &= 1862.200 - 1861.300 \\ &= 0.90\text{m} \end{aligned}$$

2. Dynamic case:

$$\begin{aligned} &\text{Head on the structure} \\ &= \text{Upstream TEL} - \text{Downstream TEL} \\ &= 1863.093 - 1862.282 \\ &= 0.81\text{m} \end{aligned}$$

The Maximum static head that is 0.090m was used for designing purposes.

Weighted creep length

According to Lane's method, weighted creep ratio is recommended in the form,

$$L > CH$$

where L: percolation distance (m.)

H: maximum head, = 0.90

C: Lane's creep ratio = 3.20

$$L > 3.20 \times 0.90$$

$$L > 2.880$$

$$L = \sum l_v + 1/3 l_h \geq CH$$

$$L_c = (1.00 + 0.50 + 1.08 + 1.10) + (0.50 + 0.70 + 1.00 + 6)1/3$$

$$= 3.68 + 2.733$$

$$= 6.41$$

Weighted creep ratio = L_c /head on the structure

$$= 6.41/0.90 = 7.72$$

Static case ($H_{max} = 0.90$)						
Point	Weight creep length			Hmax [1-LA/LC]	(TWL-WLA)	t(Safety factor/rm-1)
Case	H	V	L			
A	2.20	2.58	3.31	0.435	0.200	0.635
B	4.20	2.58	3.98	0.341	0.200	0.541
C	-	2.58	4.65	0.247	0.200	0.447
Dynamic case ($H_{max} = 0.81$)						
Point	Weight creep length			Hmax [1-LA/LC]	(TWL-WLA)	t(Safety factor/rm-1)
Case	H	V	L			
A	2.30	2.58	3.31	0.392	0.598	0.989
B	-	2.58	-	0.306	-	0.306
C	-	2.58	-	0.222	-	0.222

Checking the thickness at each point shows satisfactory results. The results are

Point A $1.00 > 0.989$ ----- Ok (Dynamic case)

Point B $0.55 > 0.540$ ----- Ok (Static case)

Point C $0.45 > 0.447$ ----- Ok (Static case)

Finally, furrow surface irrigation and ogee weir are successfully design for Jalele project. The project is small scale irrigation.

6. CONCLUSION AND RECOMMENDATIONS

To be effective and bear fruit land use, land management, soil and water resources conservation should be implemented in the project vicinity.

Liming (from locally available material like wood ashes) application is recommended to increase the availability of phosphorus and nitrogen, to furnish calcium and magnesium for plant nutrition, to improve soil properties like bulk density, infiltration capacity and percolation.

Green manuring and application of other organic matters is suggested to improve soil fertility, infiltration capacity, permeability and available water holding capacity of the soil

Where irregular rock outcrops and surface stones are encountered on canal route it should be lined as it is difficult for excavation and create the loss of irrigation water through seepage.

To protect command area from erosion as the topography is sloppy; terracing, ridge cultivation, planting fruits and crops around ridges is recommended.

Jalele river flows on sloppy land especially on its upstream catchment area. During rainy season the river transports boulders and big stones that have big enough to affect the weir structure. Ogee weir was recommended and the design was done by considering this recommendation.

Right and left side of the river is gently sloping, so it can be easily eroded; for this reason, any possible care should be given to protect the area from erosion.

Where there is sandy clay on the canal route (for example after crossing Leku river) attention should be given on its designing for sliding may be the problem.

Ogee weir was designed by using Waterways Experimental Station Standard having 2H:3V upstream face. The downstream profile was done by the equation $x^n = KH_d^{n-1}y$ where (x, y) are the coordinates of the points on the crest profile with the origin at the highest point of the crest, called the apex. H_d is the design head including the velocity head, K and n are constants depending upon the slope of the upstream face. Ogee weir of 0.90m above river bed was placed on 16m width of the

river section. Ogee weir of 2H:3V up stream face was recommended as the river flood water are full of boulders and big stones.

A main canal of length 3550m was designed. On main canal there are closed and box conduit, box canal, trapezoidal lined canal, and trapezoidal earthen canal. The hydraulic parameters of the main, secondary and tertiary canals are designed by using Manning's Equation.

Where the topography of the area dictates, drop and chute were provided to be compatible with elevations of the area. Drops were provided to convey water from a higher to a lower elevation and to dissipate excess energy resulting from this drop. A canal along this same terrain would ordinarily be steep enough to cause severe erosion in earth canals or disruptive flow in hard surface lined canals. The water must therefore be conveyed with a drop structure designed to safely dissipate the excess energy. Chutes designed were used to convey water from a higher elevation to a lower elevation. Chutes are similar to drops except that they carry the water over longer distances, over flatter slopes, and through greater changes in grades.

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