TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT ON PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

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Supervisor:

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Professor
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Baishak-2080

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IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR DEGREE IN CIVIL ENGINEERING
(Course Code: CE755)

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CERTIFICATE

This is to certify that this project work entitled "Pre-Feasibility Study of Bhuteni Irrigation Project" has been examined and declared successful for the fulfilment of academic requirement towards the completion of Bachelor Degree in Civil Engineering.

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ABSTRACT

An irrigation system is used to artificially supply water to agricultural land to support plant growth and crop production. Bhuteni irrigation project, a medium sized irrigation project, is selected for the prefeasibility study. There is abundance of land for crop production but due to ineffective irrigation system only a single crop at some area is cultivated. This project aims to explore possibility to increase the command area with three types of crops throughout the year.

Metrological database for the present study was obtained from Department of Hydrology and Meteorology for analyzing flood and monthly flows. Discharge measured was used to calculate the monthly flows using MIP method and these data were used to calculate irrigation water requirement. The structures were designed to meet the requirement.

Bhuteni irrigation project is a medium sized irrigation project located at Goldhap of Jhapa District. Its catchment area at the diversion point is 25.8 km2. The maximum discharge of river is 6.7 m^3/s in August. As per requirement for command area of 750 ha, the maximum required canal discharge is 1.91 m^3/s. The project will have a weir with gated system to pond the level of 1.4m for irrigation. The frontal type of intake is designed to supply water to the canal. For small discharge, triangular canal with round bottom is designed. Settling basin is designed to remove the silt from canal. Sarda fall type drop is designed where there is abrupt change in bed level throughout the canal as per requirement. Cross regulator and head regulator are designed to regulate flow at diversion points. Culvert is designed where a pathway intersects with the canal alignment. The headwork structure can pass flood discharge of 131 m^3/s for 100 years return period safely. About 10% of water will be discharged on downstream through under sluice for water right safety. Every attempt has been made to consider almost all the parts and get most reliable data. Due to time constraint Environmental Impact Assessment has not been done. The total estimated cost of the project is 163.21 million Nepalese Rupees and the net revenue generate from the irrigation system is 72.43 million Nepalese Rupees per year. The annual revenue generated in 20 year time period gives total revenue. The B/C ratio obtained is 2.84. Thus project is feasible. .

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NOTATIONS

Π = 3.14

ρ = density of material

n = manning's coefficient

Υ = specific weight

Ka = active pressure

Kp = pressure pressure

Φ = friction angle

g = acceleration due to gravity

° = degree

°C = degree celcius

σ = Standard deviation

ABBREVIATIONS & ACRONYMS

B	Width
BCR	Benefit -Cost Ratio
CAR	Catchment Area Ratio
C_d	Discharge Coefficient
Cm	Centimeter
CWR	Crop Water Requirement
DEM	Digital Elevation Mode
DHM	Department of Hydrology and Meteorology
D/S	Downstream
d	Diameter
ed	Actual vapor pressure in milibar
EIA	Environment Impact Assessment
FDC	Flow Duration Curve
FSL	Full Supply Level
GoN	Government of Nepal
HFL	High Flood Level
HGL	Hydraulic Gradient Line
Hl	Head Loss
Ht	Height
IRR	Internal Rate of Return
Kc	Crop Coefficient
Km^2	Square Kilometer
Km	Kilometer
KN	Kilo Newton
L	Length
m	Meter
MARR	Minimum Attractive Rate of Return
MHSP	Medium Hydropower Study Project

MIP	Medium Irrigation Project
mm	Millimeter
MWI	Monsoon Wetness Index
NPR	Nepalese Rupees
NRs	Nepalese Rupees
PCC	Plain Cement Concrete
PDSP	Planning and Design Strengthening Project
Q	Discharge
R_H	Relative Humidity
S	Longitudinal slope of canal or river
SB	Settling Basin
TEL	Total Energy Line
U/S	Upstream
WECS	Water and Energy Commission Secretariat

PROJECT SALIENT FEATURES

Location		
Place	:	Garamani
District	:	Jhapa
Zone	:	Mechi
Province No.	:	Province No.
Headwork Latitude	:	26°34'10.01"N
Headwork Longitude	:	87°59'14.79"E
General		
Name of River	:	Bhuteni Khola
Type of Project	:	Medium sized Project
Hydrology		
Catchment Area at Intake @ Bhuteni river	:	25.8 km^2
Design Discharge of canal	:	1.91 m^3/s
Design Flood @ Intake (100 years flood)	:	131 m^3/s
Command area	:	725 ha
Diversion		
Type	:	Weir-Gated System
No. of Gate	:	3
Type of Gate	:	Sluice Gate
Weir Height	:	0.7 m
Gate Height	:	0.7 m
Width of diversion	:	
Canal head regulator		
Type	:	Frontal type

No. of gate	:	1
Width	;	3.2 m
Canal		
Type	:	Triangular (round bottom)
Length	:	7.5 km
Wall Thickness	:	0.1 m
Depth	:	1 m
Settling Basin		
Type	:	Intermittent Automatic Flushing
No.	:	1
Length	:	34 m
Width	:	3.4 m
Height	:	2.8 m
Wall Thickness	:	0.2 m
Bridge		
No.	:	3
Length	:	7 m
Width	:	6 m
Height of Abutment	:	3.9 m
Culvert		
No.	:	7
Length	:	11 m
Width	:	3.8 m
Total Height	:	1.2 m
Drop		
Type	:	Rectangular Shaped Crest Wall

No.	:	3
Crest ht above U/s bed	:	0.23 m
Top width	:	0.6 m
Width of drop	:	1.9 m

Distributary Cross Regulator

No.	:	5
Width	:	1.9 m
No. of Gate	:	1
Canal Wall Thichness	:	0.2 m

Distributary head regulator

No.	:	5
Width	:	1.6 m
No. of Gate	:	1
Canal Wall Thickness	:	0.2 m

Cropping Pattern

3 crops	:	Paddy, Pulses and Vegetable

1. Introduction

1.1 Background

Bhuteni Khola Irrigation Project, a Medium Irrigation Project is located at Goldhap VDC in Jhapa District. The source of water is Bhuteni Khola which is a perennial river. The headwork of this project is 1km south of Sainik Bazar. This irrigation project irrigates about 725 ha of land of Goldhap. It has been found that local demand for water rights is persistent so the project is designed such that about 10% of water is let to be discharged downstream of the weir. The excess water of the Bhuteni Khola is accumulated by the weir which is then used for irrigating the land at the southern and western part of the headwork.

The temperature within this region is warm and suitable for irrigated agriculture. Without this irrigation project the local farmers cultivated only paddy at monsoon season and during another seasons millet was cultivated. But this project aims to change the cropping pattern in which paddy is cultivated during monsoon which is followed by winter vegetables and pulses. To assess the performance of the Bhuteni Khola Irrigation Project, various parameters such as cropping pattern, crop water requirement, conveyance losses, economic and financial returns need to be evaluated. The feasibility of the project needs to be examined, including its financial and economic viability, to ensure its sustainability and effectiveness. By conducting a comprehensive evaluation, we can identify the shortcomings of the project and recommend measures to improve its efficiency and productivity.

1.2 Need of study

This study is mainly focused on improving the agricultural production, efficiency, practice in the region of the project. The Goldhap region including its neighboring locations has been found to be largely in scarcity of irrigation system that has hampered in the agricultural yield of crops. This study will help to understand the current irrigation problems and find out the most feasible solution to the problem. By conducting a comprehensive evaluation, the study will be able to identify the areas that require attention and recommend measures to improve the efficiency and productivity of the irrigation system. Ultimately, the study will contribute to ensuring the sustainability of the project and its ability to meet the agricultural demand of the area.

1.3 Objectives

The proposed project is aimed at providing reliable irrigation facility to 750 ha of command area of Goldhap village in Jhapa. It is expected that the project will play a significant role to increase in agriculture productivity and to upgrade the living standard of the farmers and the community. The objectives & scope of the project are as follows:

i. To assess the irrigation water requirement and required canal discharge.

ii. To design the headworks and canal structures.

iii To examine the financial returns of the project.

1.4 Scope

Beside these scopes the additional scope of the study is:

- Field visit to the project site for data collection on cropping pattern, crop water requirements, and financial returns.
- Analysis of data collected from the project site and secondary sources. These data include the canal survey data, headwork survey data, interview with farmers and the soil type.
- Evaluation of the economic and financial feasibility of the project.
- Study of crop-water requirement of different crops and find out the most feasible and economically beneficial crop.
- Study of the catchment area and its delineation.

1.5 Approach and methodology

The main approaches are interaction with local beneficiaries, water users & farmers and ensure active participation of them in all phases of project development. The farmers have been involved during the entire period of fieldwork. Due attentions have been made to the suggestions and opinions given by the beneficiaries during various meeting with them. In order to collect information, meetings with the beneficiaries were conducted several times. Further, the farmers had been informed about the guidelines and procedures for the project approval & about their contributions for the implementation of the project.

The design, drawing, rate analysis and cost estimate etc. have been prepared as per the standard norms, design manual and usual practice in the field. For computation of water requirement of different crops, the PDSP manual M3 has been adopted.

1.5.1 Desk study

a. GIS, Google Earth Surveys and Topo-Graphic Map

The layout of the system was prepared on the basis of the Google Earth, which clearly shows the canal network. The order of the canal system was confirmed with the system users. The L-section and X-section of the canals have been surveyed by Level and cross section drawn at 50 m interval. Only this year rehabilitation section was surveyed and remaining was walkover with user's committees. GIS was also used to delineate the catchment area, but the resolution of Aster data that we had used was relatively low (30m) which couldn't match the catchment area delineated using Google earth and topographic map. This is because of sink cells and the low resolution of the Aster data used. Similarly, supervised classification of land use type was done using google earth to find out the net command area.

b. Estimation of Flow at Diversion/Intake

The flood was estimated using the following methods:

- Modified Dicken's
- DHM 2004
- PCJ 1996
- MHSP 1997
- WECS/DHM 1990
- RATIONAL

Similarly, for monthly flow following approaches were utilized:

- DHM 2004
- WECS/DHM 1990
- MHSP 1997
- MIP1990

The maximum daily rainfall at hydro-meteorological station was evaluated using:

- Gumbel
- Log -Normal

- Log-Pearson III

1.5.2 Site investigations

In the field level investigations particular attention were given to:

- The topography of area and best suited site for the head work.
- Proposed canal alignment.
- Location of distributary structures.

a. Walkthrough Survey

Walk through surveys were conducted by a joint team of project members and the locals. The project team was composed of students. The joint team walked along the main canal alignment and branch canal alignment from head to tail. Special attention was given to the main problems confirmed in discussions with the local. The team determined that the intake structure need to be constructed in safe area and all canal section must be lined with concrete to improve the system efficiency.

b. Agro-Economic and Social Survey

Household surveys and discussions with Key Informants were carried out to collect the farm level information. A complete list of beneficiary households from head, middle and tail of the command area was obtained from Subproject Request Form. A combined household level questionnaire (comprising questions on agriculture, livelihood, gender and other socio-institutional aspects) was used to interview the selected samples.

1.5.3 Office work

The data and information obtained from the field works have been used to prepare this report. Data from topographical survey has been used for preparation of map of the headwork site. The map has been drawn, showing demarcation of the both banks of the river, flood lines on either side of the river, lines with spot levels along which the bed slopes of the river is taken traverse lines, bench marks reference lines or points with respect to which the present topo map is prepared. In like manner longitudinal section of canal, cross-section of the canal at every 50 m, plan has also been drawn.

In view of topographical condition, nature of soil, cropping pattern, water available in the river, nature and catchments of the river etc., design of diversion structures, canal and its structures such as distribution boxes, foot bridges etc. have been proposed.

1.5.4 Cost estimation and financial analysis

As the data for the study is not enough and the exact information about the whole canal network system is not available so the cost estimation won't be accurate. So, we have used the unit rate approach for the cost estimation of the overall project. Benefit -Cost analysis method is applied to find out the economic feasibility of project.

2. Hydrological Analysis

For catchment area and command area delineation topographical maps, GIS and Google earth were used. They are multiple methods; result was obtained using them and the one with more precise according to method and topography is used for further computation as mentioned below.

2.1 Delineation of catchment area and commanded area:

2.1.1 Catchment area:

A catchment is simply an area of land where runoff is collected and then supplies to a large river, lake or ocean. Catchment area can vary in size; they can be as small as a palm or large enough to encompass all the land that drains water into river. There is a system of drain in catchment area to drain out to a common point. It combines with other catchment areas to form a network of rivers and streams that progressively drain into larger water areas. The word catchment area sometimes gets interchanged with drainage basin. Ridges and hills that separate two catchment areas are called the drainage divide. The catchment area consists of surface water like lakes, streams, reservoirs, wetlands and all underlying groundwater.

Methods for catchment area delineation:

a. Topographic Map

The procedures to calculate catchment area is:

- The outlet point is fixed and marked.
- Ridge line is identified which separates the two watersheds.
- The spot height points are identified because they separate the flow.
- The ridge points are then connected to draw the catchment area. Eventually it will connect with the point from which you started. At this point you have delineated the watershed of the wetland being evaluated. The delineation appears as a solid line around the watercourse. Generally, surface water runoff from rain falling anywhere in this area flows into and out of the wetland being evaluated. This means that the wetland has the potential to modify and attenuate sediment and nutrient loads from flooding.

b. Google Earth

- A latest version of Google Earth Pro was installed in the computer.

- Add place mark command was used and coordinate of outlet was given.
- The 2.5D map was viewed and watershed was determined to be marked.
- From the Tools tab, Ruler was selected: To measure the catchment area, Polygon Option was selected. The polygon was drawn though the watershed boundary, the measured area was noted and was saved. To draw the polygon, the procedure is to move perpendicular to the contours to reach the ridge line of the hills, as the water flows from only a side of the ridge the catchment polygon should surround that side only. After reaching the peak's end it should be returned surrounding the other side forming a polygon.
- The Path Option was selected from the Ruler and the Lc was measured.: Mouse was hovered around the catchment for the determination of maximum elevation point. In this way, area of Catchment, was measured as shown in Figure 2.1 The catchment area is found out to be 25.8 square kilometers

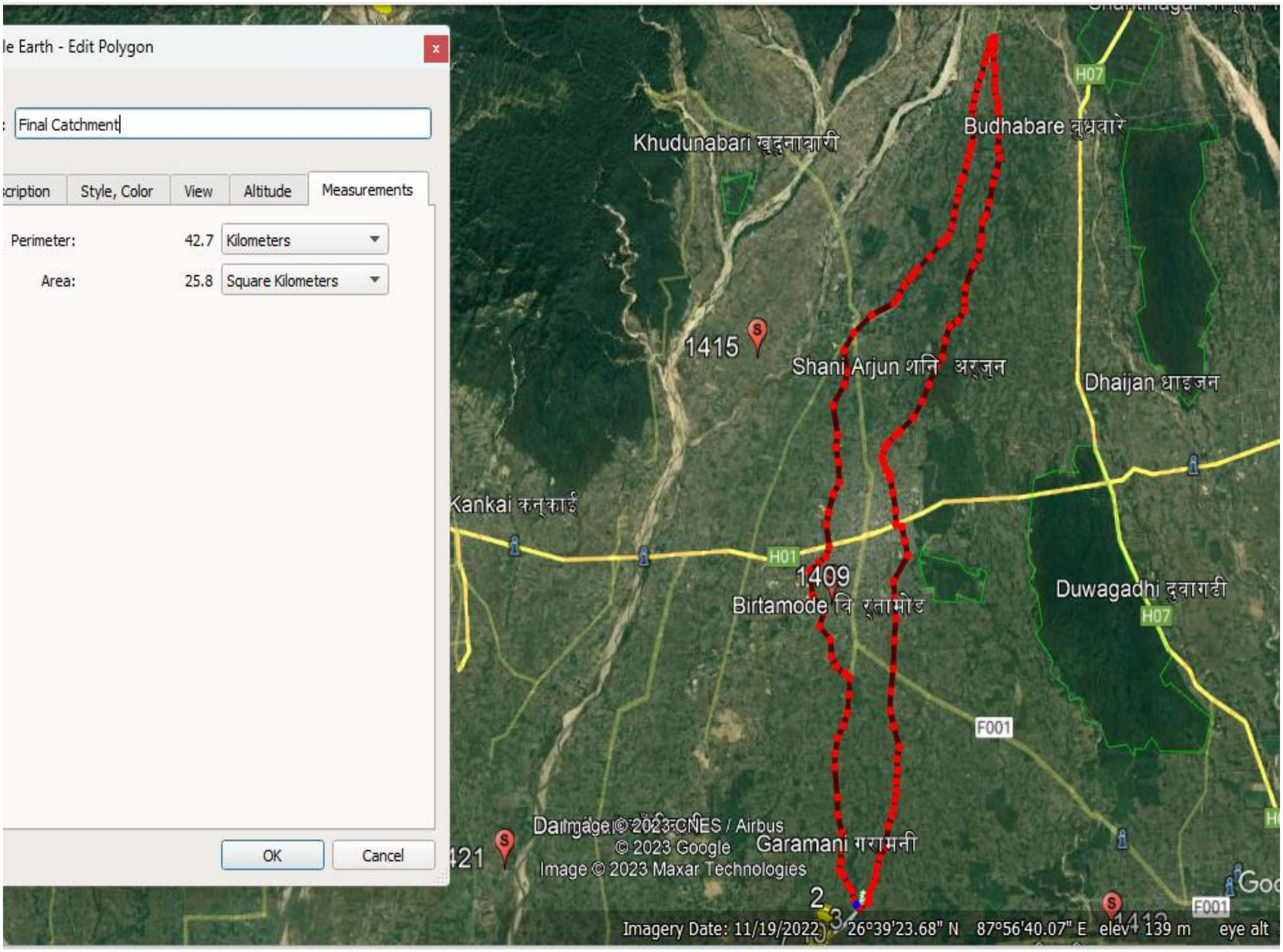


Figure 2.1: Catchment area delineated using Google earth

c. GIS

Using GIS software catchment area is calculated. GIS has resolution of 30m *30m so it could not detect our outlet point. The area obtained from GIS (as shown in Figure 2.2) was way different from that of topographic map and Google earth.

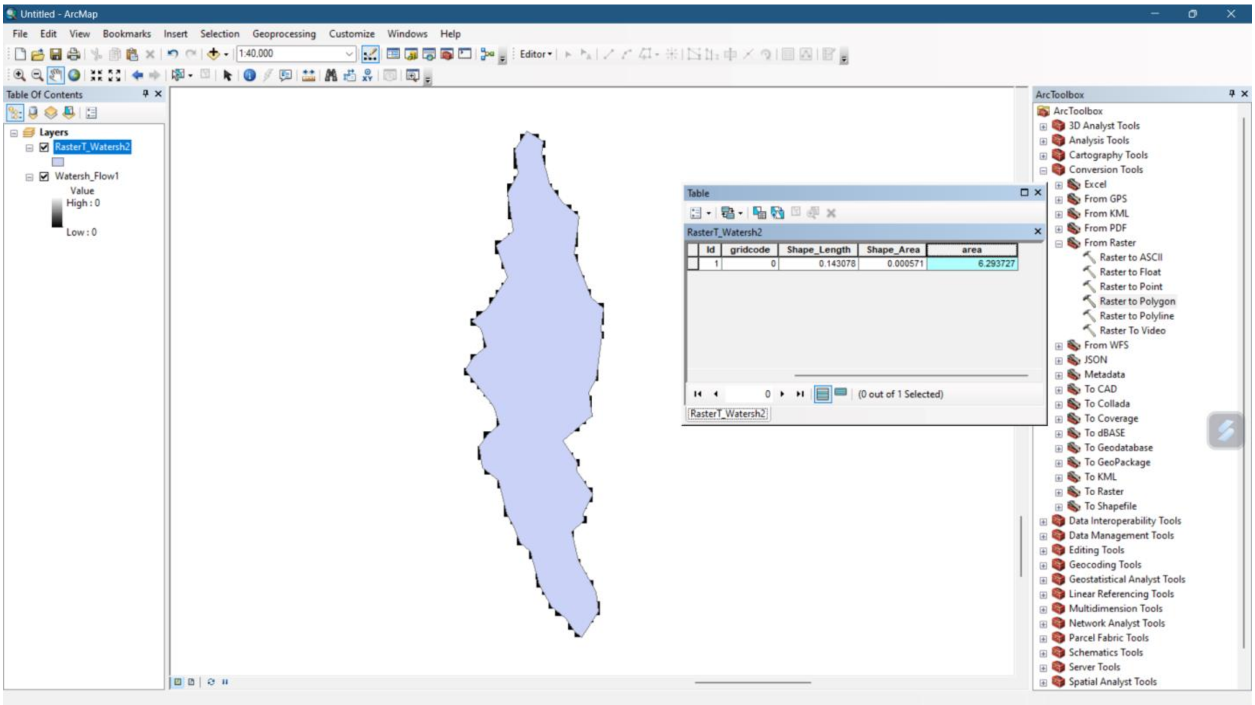


Figure 2.2 : Catchment area delineated using ArcGis

So, we go on with the area obtained through Google earth. Thus, the catchment area is **25.8 km^{2}.**

2.1.2 Command area:

For a particular irrigation system, the area of land that can be irrigated via available resource is called command area. The command area for an irrigation system is determined by factors including the water source, the crop water requirement and the topography of the area to be irrigated.

The steps followed are:

a. Field survey was done and analyzed with farmers support.

b. Primarily command area was approximated with farmer's actual need.

c. Then command area was delineated using Google earth.

d. At one side, Bhuteni khola itself restricts the command area and on other side there was a main highway lane. So, command area was chosen considering those as boundary as shown in Figure 2.3.

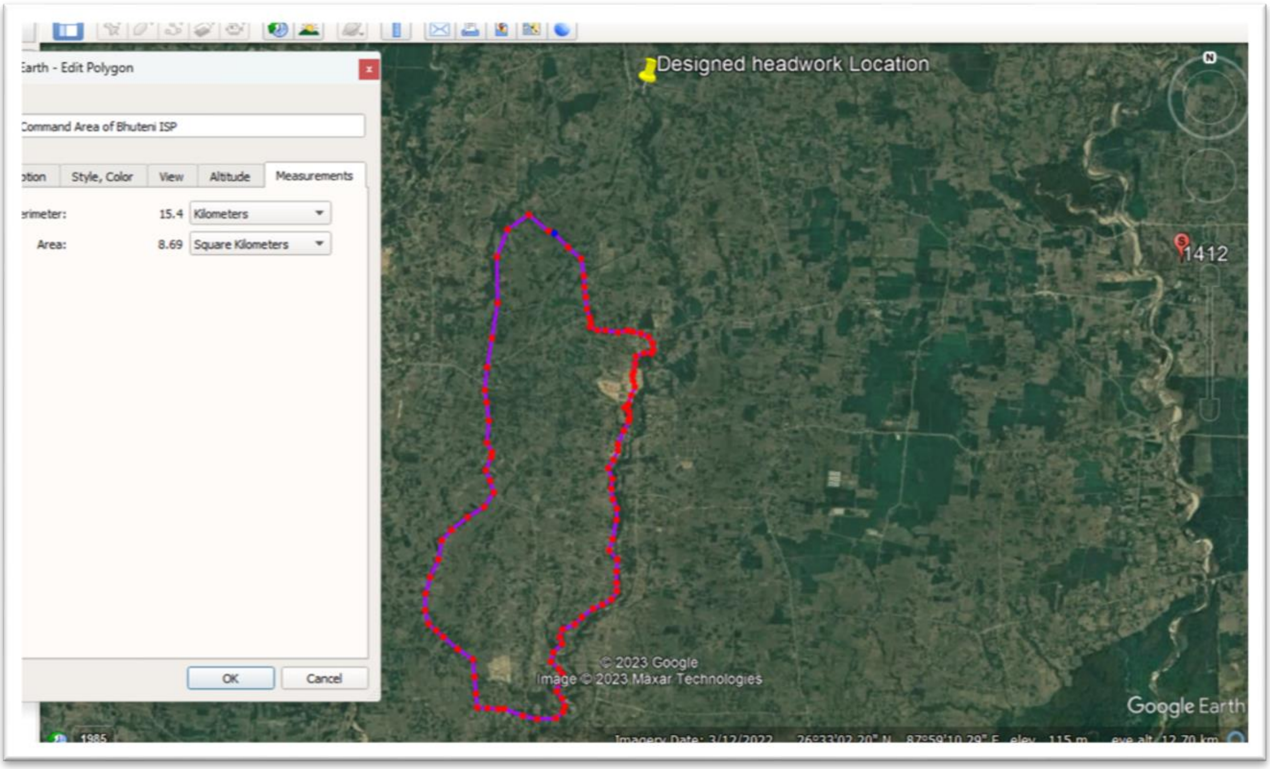


Figure 2.3: Command area delineated using google earth

The gross command area obtained using google earth is 870 ha. Studying the command area and other similar reports the net command area is taken as 85% of the gross command area. So the net command area is **750 ha.**

2.2 Hydro-meteorological database

A hydro-meteorological database is a repository of data that contains information on various hydrological and meteorological parameters, including precipitation, temperature, stream flow, and water levels. These data are utilized for analysis, modeling, and decision-making in the field of water resources management.

2.2.1 Representative rainfall stations:

The hydro-meteorological stations near our catchment area with their Index No. are in Table 2.1

Table 2.1 : Rainfall Stations and their index no.

Station name	Index No.	Station name	Index No.
Sanischare	1415	Chandra Gadhi	1412
Kechana	1422	Anarmani Birta	1409
Gaida (Kankai)	1421		

Source: http://dhm.gov.np/

Now at first, we calculate the Thiessen polygon using QGIS to find out the contributing area for our catchment as shown in Figure 2.4. and given in Table 2.2

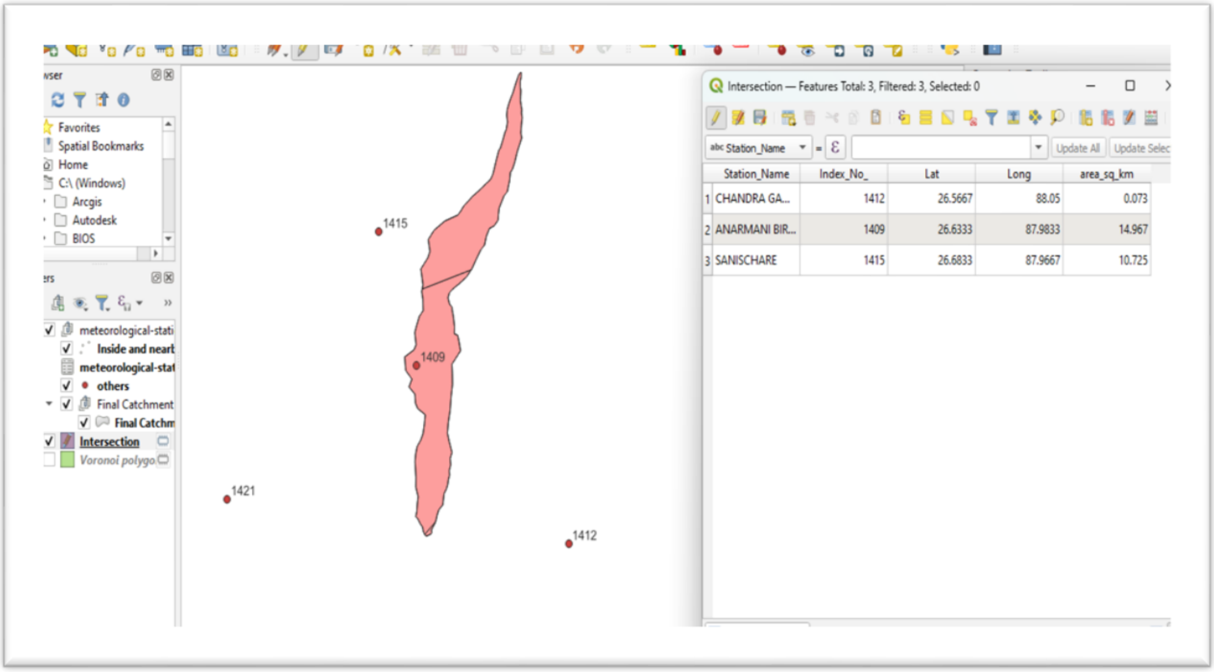


Figure 2.4: Thiessen polygon using QGis

Table 2.2:Thiessen calculation of different nearby station

Station Name	Index No.	Thiessen Area (sq. km)	Thiessen Coefficients
Chandragadhi	1412	0.07	0.0027
Anarmani Birta	1409	14.96	0.5809
Sanischare	1415	10.72	0.4163

Thus, rainfall stations namely Chandragadhi, Anarmani Birta and Sanischare are used for rainfall data computation and for flood computation and monthly flow calculation if required.

2.2.2 Maximum daily rainfall and frequency analysis:

Maximum daily rainfall is obtained through the hydro-meteorological station. Rainfall data was collected from DHM and frequency analysis was performed to calculate maximum rainfall for different return periods. It can be done by different methods like Gumbel method, log normal method, log Pearson III method. We obtain correlation between observed data and calculated data by different approaches and the one with the greatest correlation was selected.

The different frequency analysis techniques used are Gumbel distribution, Log normal type distribution and log Pearson III type distribution. These methods are used to compute the rainfall for different return year period by best fit according to correlation.

Those rainfall data obtained as in Table 2.3 are used in rational method and PCJ method due compute the flood for different return period.

Table 2.3: Maximum rainfall for different return period

Return period(T)	Anarmani	Chandragadhi	Sanischare
10	230.5	244.30	259.56
20	258.92	278.16	302.47
33	279.04	302.16	332.84
50	295.22	321.95	357.26
100	322.77	354.73	398.85
200	349.89	387.85	439.77
300	366.07	407.27	464.2
500	385.75	431.98	493.9

- For Anarmani Birta maximum correlated value was found from gumbel method i.e., $r^2 = 0.96656383$.
- For Sanischare maximum correlated value was found from gumbel method i.e., $r^2 = 0.938273946$
- For Chandra Gadhi maximum correlated value was found from Log Normal method i.e., $r^2 = 0.938273946$
- The calculation involved are shown in **Appendix – A.**

2.2.3 Potential evapotranspiration

Evaporation and transpiration are combined into one term evapotranspiration. It generally refers to the losses that occurred with evaporation and transpiration Evapotranspiration is generally estimated from climatic data. Table 2.4 consists required climatic data.

The mostly used methods are:

a. Penman's method:

The penman's equation for reference crop evapotranspiration is:

$$ETo = c\,[(w*Rn) + (1-w)*\ f(u)*(ea - ed)]$$

Where,

ETo = evapotranspiration in mm/day

c = adjustment factor depending on maximum relative humidity (RHmax), total solar radiation (Rs), daytime wind speed (Uday), and the ratio of daytime wind to night time wind (Uday /Unight). The relation is shown in **Appendix - B**

w = weighting factor depending on temperature and altitude

Rn = net radiation expressed in equivalent depth of evaporation in mm/day

f(u) = wind function,

(ea – ed) = vapor pressure deficit in millibar

A proforma is then developed sequentially to calculate the penman's evapotranspiration, the data required are:

Tmean = mean daily temperature in degree Celsius

RHmean = mean relative humidity in %

U = measured wind run in 24 hours in km/day

n = actual daily sunshine in hours

ed = actual vapor pressure in millibar

Table 2.4: DHM data for penman calculation

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
tmax	24.11	25.43	31.23	35.05	32.34	32.72	30.80	32.85	31.82	30.82	28.04	24.85
tmin	8.42	11.96	16.72	18.67	18.16	18.82	18.60	19.76	18.33	15.40	9.44	6.72
Rhmean	80.61	70.73	60.16	61.11	71.33	80.07	83.75	83.17	82.68	78.00	75.24	76.74
Wind (U)	0.97	1.27	1.78	2.11	1.88	1.50	1.08	1.00	0.95	0.73	0.52	0.53
Ra	9.31	11.13	13.12	14.94	15.90	16.26	16.03	15.30	13.82	11.86	9.84	8.74
N	10.65	11.24	12.00	12.80	13.49	13.82	13.66	13.06	12.37	11.57	10.81	10.45

Source: Department Of Hydrology and Meterology, http://dhm.gov.np/

The required data calculation is shown in **Annex – B.4**

Now by using these data penman evapotranspiration values are calculated.

The penman calculation is shown in **Annex** – B.4

Table 2.5: Monthly Eto values computed using penman's method

Month	Eto	Month	Eto
JAN	2.57	JUL	6.78
FEB	3.86	AUG	6.66
MAR	5.79	SEP	5.7
APR	7.11	OCT	4.52
MAY	7.16	NOV	2.95
JUN	7.2	DEC	2.21

b. Using cropwat

Cropwat is software developed by the Food and Agriculture Organization of the United Nations (FAO) for the calculation of crop water requirements and irrigation scheduling.

We can calculate the ETo values using cropwat in absence or presence of data. In absence of data the steps to calculate ETo are:

a. Obtaining climatic data from climwat software. For our case chandragadhi was the only near station data that climwat have.

b. Using those station data cropwat automatically calculate the ETo.

The Eto data obtained from cropwat is shown in Figure 2.5:

Monthly ETo Penman-Monteith - C:\Cropwat\CHANDRAGADHI.pen

Country: Location 7 | Station: CHANDRAGADHI

Altitude: 120 m. | Latitude: 26.56 °N | Longitude: 88.05 °E

Month	Min Temp	Max Temp	Humidity	Wind	Sun	Rad	ETo
	°C	°C	%	km/day	hours	MJ/m²/day	mm/day
January	10.5	23.4	66	86	7.9	14.6	2.26
February	11.9	26.3	63	104	8.4	17.2	3.01
March	15.9	32.0	56	121	8.8	20.3	4.38
April	20.2	34.8	37	147	8.8	22.4	5.93
May	23.1	34.0	67	147	8.1	22.1	5.38
June	25.1	33.0	77	130	5.3	18.1	4.38
July	25.3	32.2	82	121	4.2	16.3	3.87
August	24.9	32.3	84	104	4.6	16.4	3.75
September	24.0	31.7	86	95	5.7	16.6	3.62
October	21.7	31.4	74	86	7.1	16.2	3.51
November	15.4	29.8	69	78	8.1	15.2	2.93
December	10.9	24.7	76	78	7.8	13.7	2.16
Average	**19.1**	**30.5**	**70**	**108**	**7.1**	**17.4**	**3.77**

Figure 2.5: ETo calculation using cropwat

Here, the Eto data obtained using cropwat has less values than manually computed this might be because in manual calculation we have long term data. So, we would use the Eto data obtained through manual calculation rather than that obtained from cropwat.

2.3 Field investigation

a. Discharge measurement:

In field, we measured the discharge of the river using area-velocity method.

Floating method was used to measure the velocity. Floating data obtained and velocity calculation is depicted in Table 2.6. Sectional as shown in Figure 2.6 was created in AutoCAD Then by computing the cross-sectional area discharge was obtained

Discharge (Q) = Area (A) * Velocity (v)

The steps taken were:

a) Water was spilled out by opening under sluice gates.

b) At upstream a section was selected of which cross sectional area was taken through surveying.

c) Floating materials of different densities like coke bottle, small shoe, noddle cover, perfume bottle, sandal which were available at the site were used to float and calculate the time to reach the target.

d) By using the time and length to be covered, velocity was calculated as:

velocity = length/time

Length travelled by floating material = 19.6 m

Table 2.6 : Measured velocity of river

Materials	Time (sec)		Velocity (m/s)
Noodle cover	77		0.255
TT ball	85		0.232
Coke Bottle	88		0.223
Perfume Bottle	96		0.205
Sandal	99		0.198
Small shoe	105		0.187

The surface velocity = 0.223 m/s

The average velocity = 0.88*0.223 = 0.197 m/s (For river depth = 0.28m, factor = 0.88)

Then the catchment area was obtained from field measurement. At the field the depth of river flow at time of measurement was 0.28m. The area of which was approximated as

12.21 m * 0.28m = 3.42 m^2.

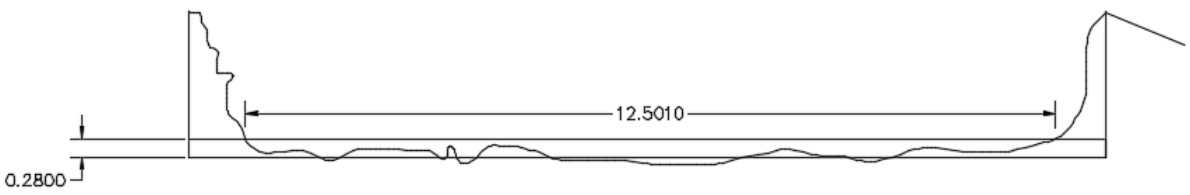


Figure 2.6: Cross section measured at Bhuteni for discharge measurement.

The discharge for river = 3.42 m^2 * 0.197 m/s = 0.67 m3/s.

b. Survey

For preparation of topographic map and for longitudinal section profile of canal we have have done survey.

The topographic map justifies the selection of headwork location at the site.

The longitudinal canal profile is used to fixed the canal bed alignment optimizing the cut and fill that occurred in the alignment. The data obtained from survey are presented in **Annex – G.**

2.4 Estimation of design flood

A design flood is a hypothetical flood (peak discharge or hydrograph) adopted as the basis in engineering design of project components. Estimation of the design flood is usually carried out by fitting observed plotting position data with a suitable probability distribution. The objective of this study is to evaluate if model selection criteria, which are seldom used in hydrological applications, can help identifying the best probability model for this purpose. Some rational, regional and empirical methods for the estimation of design flood are calculated and tabulated in Table 2.10.

2.4.1 WECS/DHM 1990

Water and Energy Commission Secretariat (WECS)/Department of Hydrology and Meteorology (DHM) developed empirical relationships for analyzing flood of different frequencies in context of Nepal. It is the modification method of WECS approach of 1982. The formula for 2-year return period is given by:

$$Q_2 = 1.8767(A_{3000}+1)^{0.8737}$$

The formula for 100-year return period is given by:

$$Q_{100}=14.63(A_{3000}+1)^{0.7342}$$

where,

Q is design flood in m3/s.

A_{3000} is basin area (in Km^2) below 3000 m elevation.

For other return period,

$$Q_T= e^{\ln(Q_2) + s\sigma}$$

where,

s= Standard Normal Variate

$\sigma = \frac{\ln(\frac{Q100}{Q2})}{2.32}$

Limitations:

- It is applicable only for catchment inside Nepal.
- It doesn't consider rainfall pattern of the specific area.
- It is an empirical method.
- It has been modified in DHM 2004.

2.4.2 DHM 2004 method

The DHM (2004) method is an update to the WESCS/DHM1990 method. The formula for 2 years returns period is given by

$Q_2=2.29(A_{3000})^{0.86}$

The formula for 100 years return period is given by:

$Q_{100}=20.7(A_{3000})^{0.72}$

For other return period,

$Q_T= e^{\ln(Q_2)+s\sigma}$

where,

s= Standard Normal Variate (which varies with return period as in Table 2.7)

$\sigma = \frac{\ln(\frac{Q100}{Q2})}{2.32}$

Table 2.7 : Standard Normal Variate for WECS/DHM1990 and DHM 2004

T (Years)	S	T (Years)	S
2	0.000	50	2.054
5	0.842	100	2.326
10	1.282	500	2.878
25	1.645	1000	3.090

Source: Garg, S. K. (2005),pg 445

Limitations:

- It is applicable only for catchment inside Nepal.
- It is an empirical method.
- It doesn't consider rainfall pattern of the specific area.

2.4.3 MHSP 1997 method

Based on the MHSP method, the flood peak is computed using the relation as below:

$$Q=kA^{b}$$

where,

Q is peak flood in m^3/s

k and b are constants which depend on the return period T as in Table 2.8.

Table 2.8 : Value of k and b for MHSP 1997

T	S	20	50	100	1000	10000
k	7.4008	13.0848	17.6058	21.5181	39.9035	69.7807
b	0.7862	0.7535	0.7380	0.7281	0.6969	0.6695

Source: Garg, S. K. (2005),pg 465

Limitation:

- It is applicable only for catchment inside Nepal.
- It is purely an empirical method
- It doesn't consider rainfall pattern of the specific area.

2.4.4 PCJ 1996 method

The PCJ method calculates design peak flood discharge based on hourly rainfall intensity. This method employs following formula:

$$Q_p = 16.67 a_p o_p \Phi F k_F + Q_s$$

where,

Q_p = Maximum rainfall design discharge for required exceedance probability (p) in m^3/sec

a_p= Maximum rainfall design intensity for required exceedance probability (p) in mm/min

$a_p = a_{hr}.k_t$,

where,

a_{hr} = Hourly rainfall intensity for required exceedance probability (p) in mm/min at selected rainfall stations

k_t = Reduction coefficient of hourly rainfall intensity (depends on the size of catchment area)

o_p = Infiltration coefficient of the basin, derived as the function of exceedance probability (p)

Φ = Areal reduction coefficient of maximum rainfall discharge (depends on the size of catchment)

F = Catchment area of drainage basin in sq. km.

k_F = Coefficient for unequal distribution of rainfall in different size of basin, captured by one rain.

Q_S = Discharge by melting of snow, can be taken as 0 to 10% of Q_P in the absence of data.

Advantage of PCJ method:

- It considers hourly-maximum whereas other methods consider daily maximum. Hence, it is more precise in this aspect.

Limitations:

- This method can be applied only for the estimation of flood discharge up to 300 yearsreturn period.
- This Method is solely developed for Nepal and values are not applicable for otherregions.

2.4.5 Rational method

A rational formula, for flood discharge considers the intensity, distribution and duration of rainfall as well as the area, slope, and permeability of the basin. This method is also based on the principle of the relationship between rainfall and runoff and hence can be considered to be similar to empirical method. It is, however, called rational method because the units of the quantities used are approximately numerically consistent. This method has become popular because of its simplicity. This method is applicable to small rural catchments. A typical rational formula is:

$$Q_T = C i_{(tc,p)} A$$

Where,

Q_T - the maximum flood discharge in m^3/s for required return period T.

C- the runoff coefficient, taken as per Table 2.9

$i_{(tc,p)}$- the maximum rainfall intensity for a given time of concentration t_c for exceedance probability p

Table 2.9: Runoff coefficient for type of basin

Type of basin	C
Rocky and permeable	0.8 – 1.0
Slightly impermeable, bare	0.6 – 0.8
Cultivated or covered with vegetation	0.4 – 0.6
Cultivated absorbent soil	0.3 – 0.4
Sandy soil	0.2 – 0.3
Heavy forest	0.1 – 0.3

(Source: https://www.researchgate.net/figure/The-runoff-coefficient-of-different-land-types)

In the absence of data on rainfall intensity, the mean intensity of rainfall (in cm/hr) can be estimated by using Sherman equation & coefficients given by Rambabu et.al.

$$I(t,p) = KT^a/(t_c+b)^n$$

where,

K, a, b and n are constants for a particular location.

For Nepal, this value can be assumed as forNorthern India (K = 5.92; a = 0.162; b = 0.5 & n = 1.013).

In the context of Nepal, if 24-hr maximum rainfall data are available, then intensity corresponding to time of concentration can be estimated from Mononobe equation.

$$i(t_c,p)= (R_p/24)*(24/t_c)^{0.667}$$

where,

R_p is the 24-hr maximum rainfall for probability P

t_c = Time of concentration:

It is the time taken by the rain water falling at the remotest point of the drainage basin to reach the point of consideration in a stream or river.

discharge measurement point. tc can be best estimated by Kirpich equation below:

t_c=0.01947 L0.77S−0.385

where,

t_c= time of concentration (minutes)

To determine in Hours above equation is modified as:

t_c = 0.000324 L0.77S-0.35

where

t_c is time of concentration in hours, In both equations,

L is length of the drainage basin in m measured along river channel upto the farthest point on

the periphery of the basin.

S is average slope of the basin from the farthest point to the discharge measuring point under consideration.

S= h_2-h_1

Where, h_2 = maximum elevation (m) & h_1 = minimum elevation (m)

2.4.6 Modified Dicken's formula:

Dicken (1885) made the first attempt in India to derive a general formula for determining the maximum flood on the basis of studies conducted for determining the relation between dischargerate to drainage area.

$$Q_T=CA^{(3/4)}$$

where,

Q_T= peak flow rate,

C=Regression constant & A=Area of Drainage (km^2)

The modified Dickens' method is an updated version of the Dickens' method. The Irrigation Research Institute, Roorkee India has done frequency studies on Himalayan Rivers & suggested the following updated relationship to compute Dickens' constant.

$$Q_T=C_TA^{3/4}$$

where,

Q_T=maximum flood discharge (m^3/s) in T yearsA=Catchment area (km^2)

C_T=modified dickens constant proposed by the Irrigation Research Institute, Roorkee, based on frequency studies on Himalayan rivers.

$$C_T= 2.342 \log(0.6T)*\log(1185/P)+4$$

$$P= 100[(a+6)/(a+A)]$$

where,

a = perpetual snow area in km^2 & T is return period in years.

Limitations:

- It was developed for India.
- It is empirical method.
- It doesn't consider any other parameter than catchment area.

Table 2.10:Flood Discharge Calculation in cumecs from Different Method

T, Years	PCJ 1996	Modified	WECS/DHM	MHSP	Rational	DHM
10	93	81	81	137	89	98

20	128	92	103	152	102	129
33	173	105	117	170	111	149
50	208	113	136	194	118	175
100	257	127	164	229	131	215

We have mentioned the values of flood discharge of different frequency by different methods in both tabular and graphical format as shown in Figure 2.7. The detailed calculation is shown in **Appendix- D**. Among those methods, PCJ 1996 gives higher value of flood discharge as it considers hourly maximum rainfall while other method uses daily maximum rainfall of the specific area. So, the flood discharge given by this method is not taken. Whereas, WECS/DHM 1990, DHM 2004 & MHSP 1997 doesn't consider rainfall pattern of that specific area. So, the values obtained from this method are also not taken. Hence, for the highest value of flood discharge of remaining method to be taken for design, we use rational method. For 100 years return period, 131 cumecs is taken as design maximum flood discharge as per the rational method.

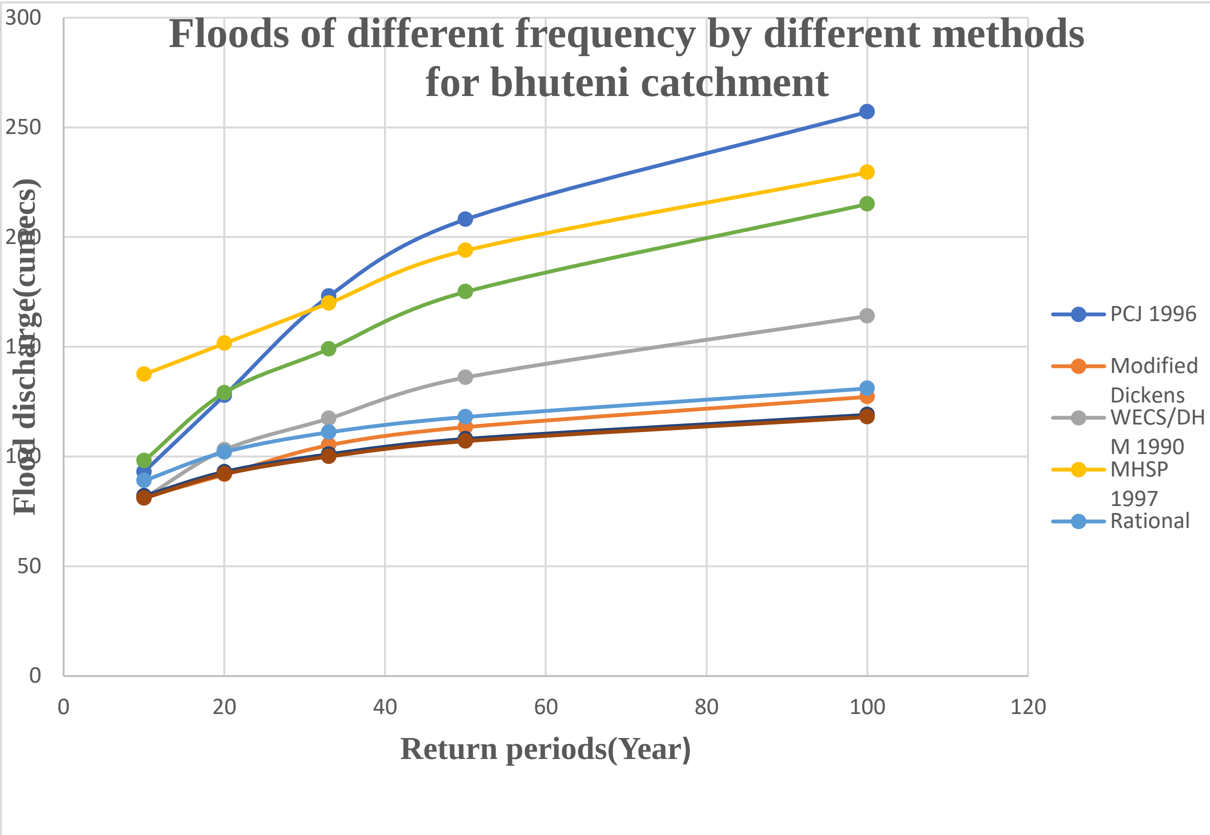


Figure 2.7: Graph for flood discharge vs time period by different methods

2.5 Calculation of high flood level

A high flood level refers to the elevation or height that floodwaters reach during a flood event. The high flood level is used as a design criterion for determining the size, shape, and location of the weir. The high flood level is used to determine the required capacity of the weir, which is a structure that allows excess water to flow over the top of the weir and safely discharge downstream. Hence, accurate estimation of the high flood level is critical in the design of weirs to ensure their safe and effective operation during flood events.

The following steps are used for the estimation of High Flood level:

- First of all, we have to choose a river cross section in which the structure is to be constructed.
- As there is already a weir structure, so, we have to measure the cross section at d/s to the structure. But, due to complications and difficulties of accessing to the d/s section, we measured the cross section of river at about 300m u/s of the existing weir.
- Now coming to the calculation part, to make easiness in calculation, the depth of river is divided into equal parts i.e., each part of 0.25m depth as in Figure 2.8.

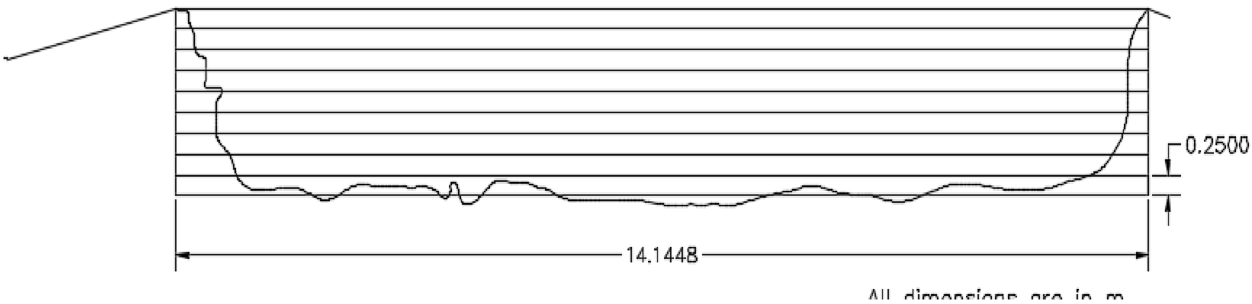


Figure 2.8: River Cross Section

- The area, perimeter and lastly discharge are calculated in different divided sections, starting from the bottom and continue it until the depth is equal to total depth of river. The value of manning coefficient is determined according to the type of soil present in the river and the approximate slope of river is obtained from the Google Earth Pro.

Taking, n = 0.035

Slope of river = 0.003

Rating curve is the curve which is plotted between water level and calculated flood discharge Calculation is shown in

The recommended design flood discharge is 131 cumecs as mentioned earlier. From Figure 2.9, the corresponding value of high flood level is 2.32m

Table 2.11: Calculation for rating curve

G-a (m)	Perimeter (m)	Area (m^2)	R (m)	V (m/s)	Q (m^3/s)
0	0	0	0	0	0
0.25	12.730	1.305	0.102	0.342	0.447
0.50	13.520	4.406	0.325	0.741	3.265
0.75	14.104	7.652	0.542	1.040	7.965
1.00	15.365	10.859	0.706	1.241	13.482
1.25	14.824	13.240	0.893	1.451	19.215
1.50	15.300	17.579	1.148	1.716	30.177
1.75	16.300	20.650	1.266	1.832	37.835
2.00	17.006	24.327	1.430	1.986	48.332
2.25	18.622	27.000	1.449	2.004	54.127
2.36	19.200	32.340	1.684	2.215	71.645

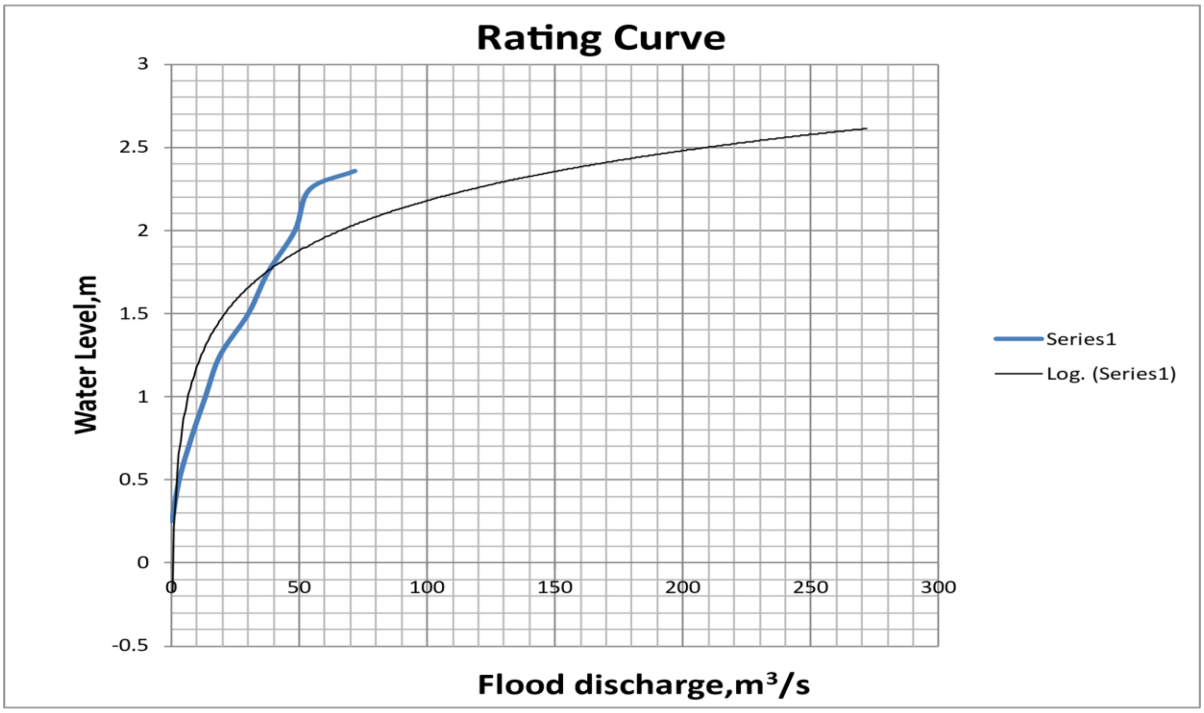


Figure 2.9: Rating curve

2.6 Availability of monthly flows

Monthly flow refers to the amount of water that flows through a river or stream in a particular month, which is measured in cubic meters per second or any other suitable unit. This information is essential for water resources management and planning, as it helps in understanding the seasonal variability of water availability and planning for water use accordingly. The term "designed monthly flow" pertains to the intended or projected rate of flow in a river or stream during each month of the year. The estimation is based on several factors like water demand, water availability, and other relevant factors involved in water resources management. The concept is generally utilized as a benchmark or objective for water allocation and management activities.

Monthly flow is calculated using different methods as presented in Table 2.12, Table 2.13, Table 2.14:

By using flood computation methods input parameters we can also calculate the monthly flows some of them are listed below:

Table 2.12 : Monthly Flow by DHM Method

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Monthly flow (m3/s)	0.54	0.45	0.72	0.5	0.48	1.4	4.01	5.98	4.43	2.07	0.94	0.64

Table 2.13 : Monthly Flow by WECS Method

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Monthly flow (m3/s)	0.35	0.3	0.26	0.25	0.3	1.22	3.77	4.66	3.63	1.59	0.71	0.47

Table 2.14 : Monthly Flow by MHSP Method

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Monthly flow (m3/s)	0.52	0.42	0.37	0.4	0.45	0.95	2.4	3.1	2.49	1.3	0.64	0.41

2.6.1 MIP method

In this method, Nepal is divided into several hydrological regions. In MIP 1990 Nepal is divided into 7 hydrological regions and in in MIP 2016 it is divided into 22 hydrological regions. MIP method gives reliable prediction only if the discharge measurement is done during the dry period (November–April). The MIP method is based upon measurement taken on an intermittent basis. The measurement of lowest discharge usually April is used to predict the mean monthly discharge of a particular location using a Unit Hydrograph (l/s per sq. Km) which was used to develop non-dimensional hydrograph for seven regions. In MIP 1990 we have first determined the April flow from most suitable method from the above-mentioned methods, then that April flow is used as the April flow in the MIP method and other values are obtained with the help of the values of monthly hydrograph.

In our case we have measured the flow by area velocity method and thus the monthly flows are reliable to use.

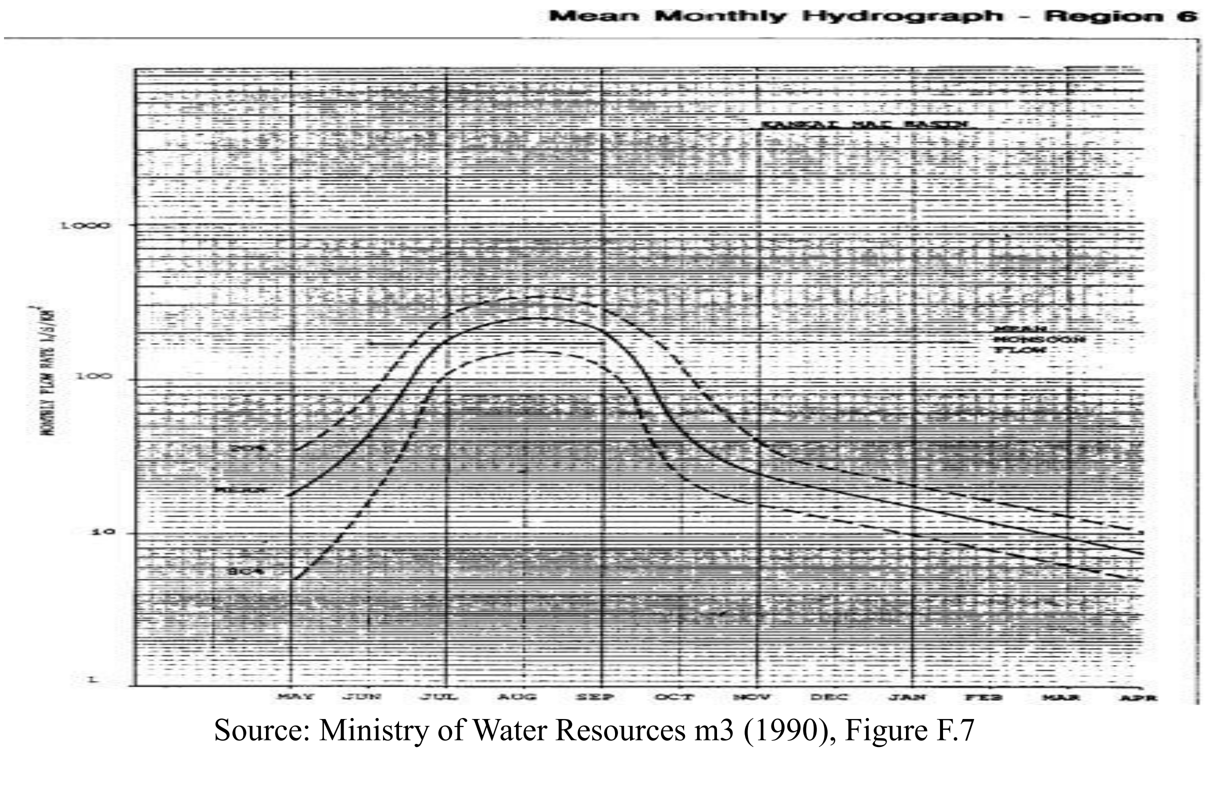
There are two methods for obtaining MIP monthly flows.

a) Using MIP graph

Our river is located at region 6. Thus, we can obtain monthly flow rate in L/S/Km^2 and by multiplying with the catchment area the monthly flows are known.

The mean monthly hydrograph is as in Figure 2.10 which gives monthly flow values as tabulated in Table 2.15

Figure 2.10: MIP graph for region 6



Source: Ministry of Water Resources m3 (1990), Figure F.7

Table 2.15 : Monthly Flow using MIP Graph

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Monthly flow (m3/s)	0.413	0.335	0.232	0.194	0.464	1.161	4.386	6.45	5.16	1.05	0.619	0.493

b) Using non-dimensional coordinates:

The MIP method has given the non-dimensional coordinates for region 6 as shown in Figure 2.11.

Knowing a monthly value all other can be computed using the following data

MIP Non-dimensional Regional Hydrographs
Mean Monthly Flow (m^3/s)

Month	Region 1	2	3	4	5	6	7*
May	2.60	1.21	1.88	2.19	0.91	2.57	3.50
June	6.00	7.27	3.13	3.75	2.73	6.08	6.00
July	14.50	18.18	13.54	6.89	11.21	24.32	14.00
August	25.00	27.27	25.00	27.27	13.94	33.78	35.00
September	16.50	20.19	20.83	20.91	10.00	27.03	24.00
October	8.00	9.09	10.42	6.89	6.52	6.08	12.00
November	4.10	3.94	5.00	5.00	4.55	3.38	7.50
December	3.10	3.03	3.75	3.44	3.33	2.57	5.00
January	2.40	2.24	2.71	2.59	2.42	2.03	3.30
February	1.80	1.70	1.88	1.88	1.82	1.62	2.20
March	1.30	1.33	1.38	1.38	1.36	1.27	1.40
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Note: * There are very few data from Region 7 - the Terai. This hydrograph must be regarded as very approximate.

Figure 2.11: MIP non dimensional values for all regions

Source: Ministry of Water Resources. m3(1990), pg. 16

For the river we have measured discharge of 0.67 m^3/s on November 15. So, the April flow is calculated and other flows are calculated using their non-dimensional values.

The computation is shown below in Table 2.16:

Table 2.16: Monthly Flow Computation by MIP

Month	Mean ND	Measured value	Mean flow (m^3/s)
Jan	2.03		0.402
Feb	1.62		0.321
Mar	1.27		0.252
Apr	1		0.198
May	2.57		0.509
Jun	6.08		1.205
Jul	24.32		4.821
Aug	33.78		6.696
Sep	27.03		5.358
Oct	6.08		1.205
Nov	3.38	0.67	0.67
Dec	2.57		0.509

Thus, from all methods the monthly flow obtained are in Table 2.17:and compared in graph using Figure 2.12

Month	DHM method	WECS method	MHSP method	MIP mean (Graph)	MIP mean (table)
Jan	0.54	0.35	0.52	0.412	0.402
Feb	0.45	0.3	0.42	0.335	0.321
Mar	0.72	0.26	0.37	0.232	0.252
Apr	0.5	0.25	0.4	0.193	0.198
May	0.48	0.3	0.45	0.464	0.509
Jun	1.4	1.22	0.95	1.161	1.205
Jul	4.01	3.77	2.4	4.386	4.821
Aug	5.98	4.66	3.1	6.450	6.696
Sep	4.43	3.63	2.49	5.160	5.358
Oct	2.07	1.59	1.3	1.057	1.205
Nov	0.94	0.71	0.64	0.619	0.670
Dec	0.64	0.47	0.41	0.493	0.509

Table 2.17: Monthly flows from different method

Here, we don't have any long-term flows and also, we don't have hydrologically similar catchment. Thus, for this case, when we have measured discharge for a particular month MIP method stood the most reliable. Thus, we proceed with the MIP monthly flows as our design flow.

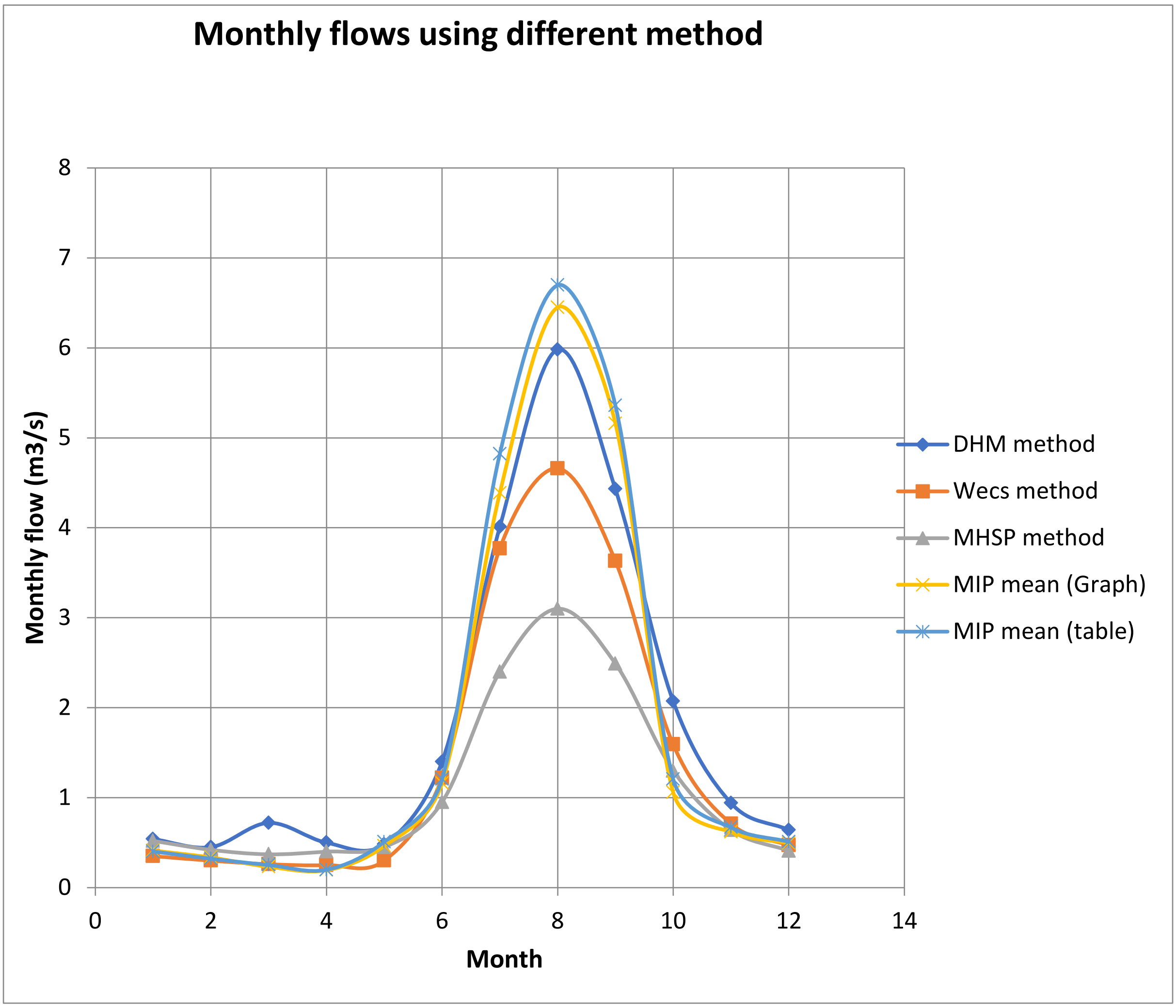


Figure 2.12: Monthly flows using different method

2.7 Design monthly and half monthly flows

The design monthly flows are obtained from MIP method.

For computation of total water requirement, we convert these values to half monthly values. Basically, for precise measurement the climatic records are recorded for half monthly flows and for which half-monthly flows are not obtained, they are calculated as below:

To calculate the values for the second half of month A (a2) and the first half of the following month B (b1) from the respective monthly values a and b, the following equations are used:

$a2 = (3a + b) / 4$

$b1 = (3b + a) / 4$

Thus, by using above concept, the half monthly flows are computed and given in Table 2.18:

Table 2.18 : Monthly and half monthly flows

Month	MIP monthly	1st half	2nd half
Jan	0.402	0.43	0.38
Feb	0.321	0.34	0.30
Mar	0.252	0.27	0.24
Apr	0.198	0.21	0.28
May	0.509	0.43	0.68
Jun	1.205	1.03	2.11
Jul	4.821	3.92	5.29
Aug	6.696	6.23	6.36
Sep	5.358	5.69	4.32
Oct	1.205	2.24	1.07
Nov	0.670	0.80	0.63
Dec	0.509	0.55	0.48

3. Crop Water and Irrigation Water Requirements

Crop water requirement refers to the amount of water that a crop needs to grow and develop satisfactorily over a specified period, considering climatic conditions, soil properties, and crop characteristics. It is an estimate of the total amount of water that a crop uses during its growth cycle, including evaporation from the soil and transpiration through the plant.

The crop water requirement is influenced by several factors such as the crop type, growth stage, planting density, temperature, humidity, wind speed, solar radiation, and rainfall. The determination of crop water requirement is critical in irrigation management and helps farmers to plan their water use efficiently, minimize water wastage and ensure optimal crop yield and quality.

Irrigation water requirement refers to the amount of additional water that must be applied to a crop during the irrigation season to supplement the amount of water available from rainfall and soil moisture, to ensure optimal plant growth and development. It is the amount of water that must be applied to the crop through irrigation to replace the water lost through evapotranspiration.

3.1 Cropping pattern

A cropping pattern refers to the planned sequence and arrangement of crops grown on a specific land area over a designated period. It typically involves intercropping or rotating different crops in a specific sequence or pattern, aimed at optimizing land productivity while reducing the risk of pests, diseases, and soil erosion.

Cropping patterns may vary, depending on several factors, such as the climate, soil type, available irrigation facilities, technology, and the goals and preferences of the farmer. The approach may involve cultivating a single crop on the same land for a whole season or alternating different crops on the same plot over a series of seasons.

3.1.1 Previous cropping pattern

The previous cropping pattern was found to be paddy at monsoon period. No other crops were cultivated in other period. There was lack of irrigation system and because of which approximately 300 ha of land was irrigated for above mentioned cropping pattern. For other seasons some crops with no water requirement like millet is grown.

3.1.2 Present cropping pattern

The present cropping pattern adopted for this season was monsoon paddy followed by winter vegetables and then pulses. They form the perfect combination of food crop and cash crop. And this cropping pattern fulfills the availability of irrigation system and farmers need so this pattern is adopted.

3.2 Computational method

The data required for calculating crop water requirement are:

a. Rainfall and its contribution:

IWR are generally calculated based on 80% reliable rainfall.

$P_{80\%} = P_{mean} – 0.8416 * \text{standard deviation}$

Where

P_{mean} = mean rainfall

0.8416 is the reduced variate

b. Effective rain:

p-eff = f* $P_{80\%}$

where

paddy crops:

f = 0 for $P_{80\%} < 5$mm

f = 0.85 for 5 mm < $P_{80\%}$ < 100 mm

f = 0.7 for 100 mm < $P_{80\%}$

for upland crops:

f = 0.7 for any amount of $P_{80\%}$

c) Crop coefficient (Kc):

Crop coefficient is a dimensionless factor used in agricultural and irrigation engineering to estimate the water requirements of a specific crop, based on its stage of growth, leaf area, and other plant characteristics. It is the ratio of the actual evapotranspiration of the crop to the reference evapotranspiration under the same conditions.

Kc values for rice is taken from Figure 3.1.

Source: Ministry of Water Resources m3 (1990), pg. 23

Crop Coefficients (kc) for Rice

	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec		Approx. Duration (days)
Crop	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	
Monsoon													1.10	1.10	1.19	1.10	0.95	0.95							90
Early							1.10	1.10	1.10	1.10	1.00	1.00													90
Late															1.10	1.10	1.10	1.10	0.95	0.95					90
Monsoon														1.10	1.10	1.10	1.10	1.05	0.95	0.95					105
Early						1.10	1.10	1.10	1.10	1.25	1.00	1.00													105
Late															1.10	1.10	1.10	1.10	1.05	0.95	0.95				105
Monsoon														1.10	1.10	1.10	1.10	1.05	1.05	0.95	0.95				120
Monsoon														1.10	1.10	1.10	1.10	1.05	1.05	1.05	0.95	0.95			135
Monsoon														1.10	1.10	1.10	1.10	1.05	1.05	1.05	1.05	0.95	0.95		150

.

Figure 3.1: Kc values for paddy

Kc values for other crops selected from Figure 3.2.

Source: Ministry of Water Resources m3 (1990), pg. 24.

Crop Coefficients (kc) for Selected Crops

	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec		Approx. Duration (days)
Crop	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	
Maize 1					0.45	0.60	0.80	1.05	1.05	1.05	0.80														105
Maize 2							0.45	0.60	0.80	1.05	1.05	1.05	0.80												105
Pulses	0.50	0.75	0.95	1.05	1.05	0.96																		0.40	105
Oilseeds	0.46	0.82	1.00	1.00	0.72																			0.40	90
Wheat 1	1.15	1.15	1.15	0.90	0.40																	0.43	0.65	1.05	120
Wheat 2	1.05	1.15	1.15	1.15	0.90	0.40																	0.43	0.65	120
Vegetable (summer)													0.34	0.54	0.93	1.05	1.05	1.04	0.91						-
Vegetable (winter)	0.86	0.95	0.95	0.89																		0.28	0.34	0.54	-
Potatoes	1.01	1.13	1.13	1.08	0.94	0.77																0.42	0.55	0.79	130
Potatoes	0.79	1.01	1.13	1.13	1.08	0.94	0.77																0.42	0.55	130

Figure 3.2: Kc values for other crops

d) Land preparation:

Land preparation refers to the process of preparing agricultural land for cultivation, including activities such as plowing, harrowing, leveling, and fertilizing. Land preparation is done for paddy crop only. Generally, 55 mm is taken for land preparation according to FAO.

e) Percolation Losses:

Percolation loss refers to the amount of water lost from a canal, reservoir, or other water storage structure due to seepage into the soil or subsurface. For calculation of Crop water requirement, percolation loss is also considered only for rice crop. Other losses like canal losses are accumulated or defined while calculating Irrigation water requirements.

Using these parameters crop water requirement is calculated.

For Irrigation water requirement, the losses typically field application losses, farm distribution channel losses and main canal losses were added as efficiency as per the guideline provided by FAO (Food and Agricultura.

The detail calculation is shown in **Appendix – C.1**

The IWR obtained for each half month are tabulated in Table 3.1:

Table 3.1 : Gross Irrigation Requirement for each half month

Month	I-gross (m^3/sec)	Month	I-gross (m^3/sec)
Jan	0.17	Jul	0.67
Jan	0.22	Jul	1.90
Feb	0.27	Aug	1.91
Feb	0.31	Aug	1.40
Mar	0.09	Sept	1.34
Mar	0.13	Sept	1.31
Apr	0.20	Oct	1.13
Apr	0.26	Oct	0.90
May	0.29	Nov	0.00
May	0.27	Nov	0.07
Jun	0.19	Dec	0.07
Jun	0.25	Dec	0.10

3.3 Duty of water

Duty of water is defined as the no. area unit of land irrigated by supplying of 1 m^3/s of water for full growth of crop. Generally, the area unit is ha. So, the unit of duty is ha/cumes.

For our irrigation system,

Net command area = 750 ha

Maximum discharge = 1.91 m^3/s

Thus, the duty of water =392.67 ha/m^3/s

4. Design of Headworks

4.1 Design of weir with gated system

4.1.1 Introduction

Weirs and barrages are constructed and designed to divert the river's entire or partial flow into a canal or conduit for diversion and usually have a limited storage capacity. They are particularly useful in regions where river flows are inconsistent and water needs to be stored for use during times of low flow. So, we construct a sloping glacis weir with gated system to raise the water level in the source channel to the required level so as to divert the required supplies into the off taking channel for the purpose of irrigation in Terai region.

4.1.2 Design consideration for weir and under sluice

- High flood discharge is taken as 131 cumecs, which was obtained from Rational method.
- High flood level was determined at weir site from the rating curve corresponding to the high flood level.
- Pond level was obtained by adding 0.5 m to the full supply level of canal.
- The weir crest level was fixed so as to pass 80% of the designed flood discharge.
- Afflux is commonly taken as 1m.
- Retrogression is taken as 0.5m.
- Concentration factor is generally taken as 20%.
- Width of crest is taken as 1m considering the stability of structure.
- U/S slope as 2:1 and D/S sloping glacis as 3:1 is taken.
- Lacey's silt factor=1.5 and Safe exit gradient = 1/6 is taken corresponding to the type of material present i.e., medium sand.

4.1.3 Design procedure

Following data were prepared/collected before a weir can be designed:

a. High flood discharge was taken as 131 cumecs, which was obtained from Rational method as mentioned earlier.

b. River cross section at the weir site.

c. Stage discharge curve i.e rating curve for the river at weir site.

d. High flood level was determined at weir site from the rating curve.

Factors to be decided while designing a weir are:

a. Pond level was obtained by adding 0.5 m to the full supply level of canal.

b. Afflux was commonly taken as 1m.

c. Waterway was measured from river cross section at weir site.

d. The weir crest level was fixed so as to pass 80% of the designed flood discharge.

The discharge formula to be used in the design of sharp crested weir is:

$$Q = 1.84(L-KnH)\, H^{3/2}$$

Where

Q = discharge in cumecs

H = Total head in meter including velocity head

n = No. of end contractions (twice the number of gated bays)

L = clear waterway length in meter

K = coefficient of end contraction: generally taken as 0.1 in ordinary calculations

e. Retrogression was taken as 0.5m (generally adopted value).

f. Concentration factor was taken as 20% (generally).

Steps to be followed during designing a weir:

a. Determine head loss H_L for different flow conditions.

b. For known values of q and H_L, read corresponding values of Ef_2 from Blench curve (Figure 4.1) and with known values of Ef_2 read corresponding values of d_2.

Cistern level =downstream TEL-Ef_2

c. Ef_1=Ef_2 +H_L, knowing Ef_1, Ef_2 and q, values of d_1 and d_2 were calculated using formula

Ef1=d1 + (q/d1)2/(2*g)

$$Ef_2=d_2 + (q/d_2)^2/(2*g)$$

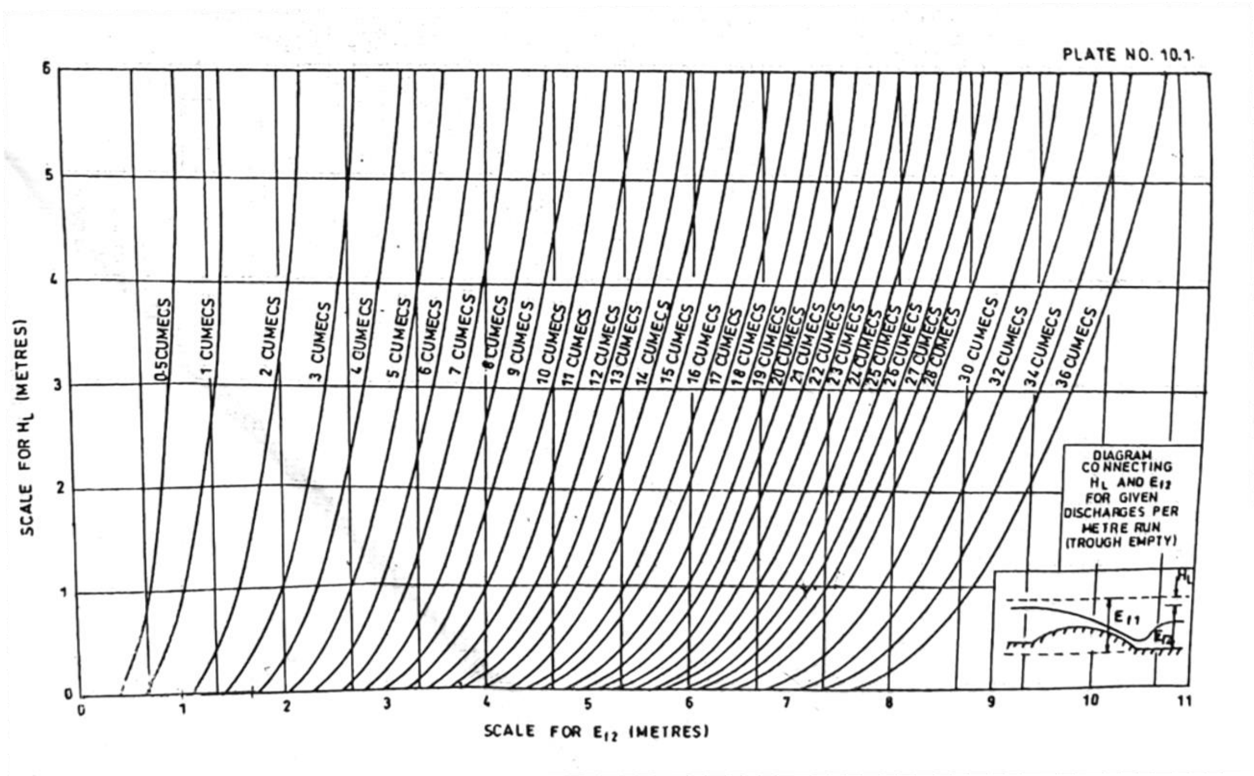


Figure 4.1: Blench Curve

(Source: Jha, P. C., & Devkota, (2074), pg. 268)

d. Scour depth was determined from the formula: $R=1.35(q^2/f)^{1/3}$

Depth of upstream sheet pile from scour consideration=1.5R

Depth of downstream sheet pile from scour consideration=2R

e. The characteristics of the hydraulic jump for the high flood condition and pond level condition were fixed with

i. No flow concentration and no retrogression

ii. flow concentration and retrogression

f. The value of $\frac{1}{\Pi\sqrt{\lambda}}$ from the equation $\frac{1}{\Pi\sqrt{\lambda}} = G_E\frac{d}{H}$ for the adopted value of G_E and the known value of d (downstream sheet pile) and H(maximum static head). Corresponding to the value of $\frac{1}{\Pi\sqrt{\lambda}}$,value of d was computed.

Total length of floor b=αd was provided

Deposition of total floor length was as follows:

Cistern length =5(d_2-d_1) t0 6(d_2-d_1)

Glacis length =3 to 5 times (crest level -cistern level) from 3:1 slope of glacis plus the cover width

Upstream floor =the balance

g. In order to determine uplift pressure acting on the floor, the %pressures at upstream and downstream sheet piles were worked out. The pressure distribution from upstream sheet pile line to downstream sheet pile is assumed to be linear.

Correction due to floor thickness

The thickness of the floor at the location of the sheet piles are tentatively assumed for correcting the values of ϕ_C in the upstream and ϕ_E in the downstream. If t_l is the floor thickness at upstream sheet pile of depth d_1, correction due to floor thickness = $t_l/d_l*(\phi_D - \phi_E)$. If t_2 is the floor thickness at downstream sheet pile of depth d_2, the correction = $t_2/d_2*(\phi_E-\phi_D)$ which is negative.

Correction due to mutual interference of sheet piles

The correction due to mutual interference of sheet pile is worked out by the formula:

$$C = 19\left(\frac{D+d}{b}\right)\sqrt{\frac{D}{b'}}$$

Correction due to slope

This is applicable only in case where an intermediate pile is provided.

h. After knowing the corrected % pressures under the key points the sub-soil pressure gradient line and hydraulic gradient line for surface with reference to the corresponding downstream water level as datum. The corresponding water profiles before and after the jump formation are plotted for the given values of discharge intensity q.

Knowing q and Ef_l at different location of the glacis, corresponding values of d_l are calculated and thus the water profile before jump formation can be plotted. For plotting water profile after jump, the Froud number Fr is determined and knowing Fr, relation between the abscissa and ordinate of the profile can be read from the curve shown in Figure 4.2.

The uplift pressure which will occur with different flow conditions was determined. The requirement of floor thickness is worked out taking the larger of those uplift pressure and divided by (G-1), G being the density of floor material and (G-1) the submerged density of floor material.

Post jump profile was plotted using following curves:

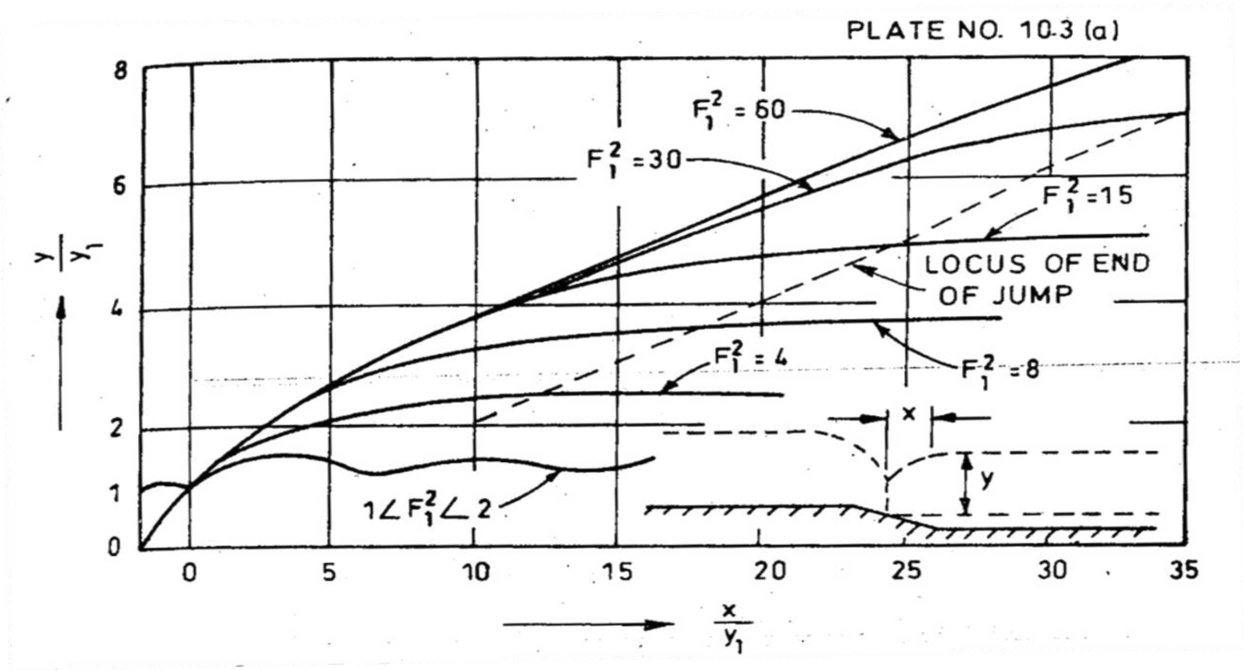


Figure 4.2 : Curve for Plotting Post Jump Profile

(Source: Jha, P. C., & Devkota, (2074), pg.270)

i. The protection works are designed for the calculated scour depth using following formulas:

1.) For u/s protection works:

The length of concrete block = 1.5*scour depth below upstream bed(D1)

Adopt certain thickness of gravel filter (taken as 0.5m).

Length of launching apron = 1.5*scour depth below upstream bed(D1)

Thickness of launching apron= 2.24*D1

2.) For d/s protection works:

The length of concrete block = 1.5*scour depth below downstream bed(D2)

Adopt certain thickness of gravel filter (taken as 0.5m).

Length of launching apron = 1.5*scour depth below downstream bed(D2)

Thickness of launching apron= 2.24*D2

4.1.4 Results

Following designed values are obtained and listed in Table 4.1:

Table 4.1 : Weir data Obtained from Design

Descriptions	Obtained value
D/S high flood level	100.517m
Crest level	98.917m
Pond level	1.4m from river bed
Height of weir from river bed	0.7m
Height of gate	0.7m
No of piers	2, each of width 1.5m
No of bay	3, each of length 4.67m
U/S slope	2:1
D/S sloping glacis	3:1
RL of u/s floor of weir (river bed)	98.217m
RL of d/s floor of weir	95.66m
Length of u/s impervious floor	2.83m of 0.4m thick
Length of d/s impervious floor	16m
Thickness at toe of glacis	2.3m
Length of concrete block at u/s of weir	4.28m
Length of launching apron at u/s of weir	4.1m
Length of concrete block at d/s of weir	5.9m
Length of launching apron at d/s of weir	6m

Here

a. Location of weir was selected in such a way that the site was a narrow, straight, well defined channel confined between banks not submerged by the highest flood.

b. The height of weir was fixed so as to pass 80% of designed flood discharge using discharge formula for sharp crested weir.

c. Generally, the waterway is calculated by Lacey's perimeter Formula:

$P = 4.75\sqrt{Q}$

But in this case rather than choosing the weir length by using above formula, the weir length was chosen as per the section of the river at weir site.

d. Sloping glacis weirs are typically used when a gradual slope is desired, such as when controlling the flow rate of water in a canal or other watercourse. Sloping glacis weirs can be used in irrigation systems to control the amount of water delivered to crops. On the other hand, vertical drop weirs are typically used in situations where a more abrupt change in water level is desired. They are often used in dams or reservoirs to regulate water flow. On the basis of above requirements, sloping glacis weir was chosen.

e. According to the type of material i.e medium sand present in the river, the value of silt factor is taken as 1.5 and corresponding value of exit gradient is 1/6.

4.2 Design of under sluice

4.2.1 Introduction

A comparatively less turbulent pocket of water is created near the canal head regulator by constructing under sluice portion of the weir. A divide wall separates the main weir portion from the under-sluice portion of the weir. Normally, the crest of the under sluice is kept equal to the deepest bed level of the river. The functions of under sluice are to control the silt entry into the canal, to pass the low floods without dropping the Sutter of the main weir and to preserve a clear and defined river channel approaching the regulator.

Design consideration for under sluice is provided in section 4.1.2.

4.2.2 Design procedure

Before starting the actual design of under sluice, the discharge over the under sluice section should be decided.

The discharge over the under sluice section should be equal to or greater than the greatest of the following:

a. It should be at least equal to twice the supply discharge of the off taking canal.

b. It should be greater than the dry weather flow of the river during winter so that weir crest gates are not required to be opened.

c. It should be a substantial portion of the total design discharge so that small floods can be passes over the under sluice section, without lifting the weir crest gates. It is the usual practice to take the discharge of the sluice section about 20% of the total discharge and the remaining 80 % discharge over the weir bay section.

Steps:

a. The crest level of the under sluice section is generally taken as river bed level.

Slope of D/S glacis was taken as 3:1.

b. Fix the water way for the under sluice section.

c. The discharge above the under sluice section was fixed using the above criteria.

The formula for discharge through under sluice section (broad crested weir) is

$$Q=1.7(L-Kn\ H)\ H^{3/2}$$

Where,

Q= Discharge in cumecs

H=Total head in meters including velocity head

N=No. of end contractions (twice the number of gated bays)

L=Clear waterway length in meters.

K=coefficient of end contraction (0.1 for blunt noses to 0.04 for thin pointed Noses)

d. The characteristics of the hydraulic jump for the high flood condition and pond level condition were fixed with

i. No flow concentration and no retrogression

ii. With flow concentration and retrogression

e. The normal scour depth was calculated and the bottom levels of the upstream and downstream piles were determined.

f. Total length of the impervious floor was determined from the exit gradient considerations. Also, the lengths and levels of the upstream and downstream floors were fixed. The D/S floor is fixed below the point at which the hydraulic jump is formed.

i. The percentage uplift pressures were calculated at the key points of all piles by Khosla's theory for the following conditions.

ii. No flow condition and High flood condition and pond level condition, without flow concentration and with retrogression.

iii. High flood condition and pond level condition, with flow concentration and retrogression. Draw the subsoil hydraulic gradient line (HGL) for each case.

g. Ef2 was computed from Figure 4.1: Blench Curve

h. The uplift pressure at various points from the subsoil HGL for no flow condition were determined. Also, the suction pressure at the jump through for high flood condition and pond level condition were determined.

i. Curve in "figure 4.2: Curve for plotting post jump Profile "was used to plot post jump profile

j. The thickness of the floor at various points were calculated at various points for the maximum of the uplift obtained for the three conditions.
k. The protection works are designed for the calculated scour depth using following formulas:

For u/s protection works:

The length of concrete block = 1.5*scour depth below upstream bed

Adopt certain thickness of gravel filter (taken as 0.5m).

Thickness of launching apron= (2.24*depth of scour below u/s bed)/adopted length

For d/s protection works:

The length of concrete block = 1.5*scour depth below downstream bed Adopt certain thickness of gravel filter (taken as 0.5m).

Length of launching apron = 1.5*scour depth below downstream bed

Thickness of launching apron= (2.24*depth of scour below d/s bed)/adopted length

4.2.3 Results

Following designed values are obtained and listed in Table 4.2:

Table 4.2 : Design Result obtained for Under Sluice

Descriptions	Obtained value
Waterway	2.5m
Crest level (bed level)	98.217 m
D/S slope	3:1
Length of u/s impervious floor	0.729m of 0.4m thick
Length of d/s impervious floor	17.5m
RL of u/s floor	98.217 m
RL of d/s floor	94.96m
Length of u/s concrete blocks	8.6m
Length of u/s launching apron	8.1m of 2.3m thick
Length of d/s concrete blocks	10.22m
Length of d/s launching apron	10m of 1.5 thick

Here

i. The crest level of under sluice was taken as same as the level of river bed.

ii. The waterway of under sluice was taken in such a way that it can pass the discharge as per criteria mentioned in procedure.

4.3 Canal head regulator

4.3.1 Introduction

A canal head regulator is to be provided at the canal entrance. It is placed at the river bank just upstream of the under sluice. In the Bhuteni irrigation project, the head regulator is designed at 2m upstream of the under sluice.

4.3.2 Design consideration

Design Consideration for Canal:

- Lacey's silt Factor = 1.5
- Invert level =1m
- D/S glacis slope =1:3
- U/S floor thickness = 0.3
- Exit Gradient = 1/6

4.3.3 Design procedure

Crest level of the head regulator is kept at 1m higher than the bed level. The d/s FSL is 99.117m and the U/S pond level is 99.617m (0.5m modular head is maintained), the crest level is kept at 99.217m i.e., there is no submerged flow from the head regulator. The crest width is calculated as:

$$\boxed{Q = \underline{2}\ *Cd*B*\sqrt{[2g*\{(h+h_a)^{3/2} - h_a^{3/2}\}]}}$$

where,

h = head causing flow = pond level- FSL

d = depth of water over crest on D/S

B = length of crest

H_a = head due to velocity of approach

Cd = coefficient of discharge for free flow = 0.577

At high flood condition i.e., water level at U/S is 101.517m, the gate opening required for the passing 1.91 cumec discharge is given by:

$Q = Cd*A*\sqrt{2gh_1}$

where,

h_1 = head causing flow (U/S HFL-D/S FSL)

A = area of flow

Cd = 0.62 is the coefficient of discharge

$Q = 0.62*x*B*\sqrt{2g}\ h_1$

The value of x can be determined from the known values of Q, B and h_1..

From the above given formulas, the value of crest width has been found to be 3.2m and opening size is 0.149m.

For the crest width of 3.2m the discharge intensity was found to be 0.597 cumec/m.

The length of downstream floor was calculated for both the pond level and high flood condition using the blench curve and energy equations.

Length d/s impervious floor was calculated using the equation $L= 5(y_2-y_1)$ and found to be 4.16m, but for higher safety we adopted d/s floor length as 7m

The total length of impervious floor was determined using the permissible exit gradient for the soil, the exit gradient of 1/6 was adopted and the total minimum length required was 23.57m (24m total length is adopted) such that:

Length of U/S floor =13.95m

Length of D/S floor =7m

Length of crest= 1m

Length of sloping glacis=3.051m

Scour depth = $1.35(q^2/f)^{1/3}$ = .8361m,

For f=1.5 and q=.597 cumec/m

Depth of U/S pile =5.355m (adopted same as pile depth for under sluice)

Depth of D/S pile = 3m > $1.5(q^2/f)$

Using Khosla's theory uplift pressure at u/s and d/s points were by applying the corrections for interference of pile and correction for thickness of floor and using the interpolated value of uplift pressure the total floor thickness was determined.

Correction for mutual interference = $19\sqrt{(D/b')} *(d+D)/b$

where,

D is the corresponding pile depth

b' is the horizontal distance between adjacent piles

b is total floor length

d is the elevation difference between the top of corresponding pile and bottom of adjacent pile

For head regulator the uplift pressure at downstream is maximum when there is no flow at high flood.

In this case, the seepage head is maximum and was found to be 3.317m.

The floor thickness is calculated as

(Uplift Pressure / Specific gravity of concrete-1)

Floor thickness at d/s floor =1.014m adopt 1.1m

Floor thickness at sloping glacis=1.1644m adopt 1.2m

Length of d/s apron= 1.5*d/s pile depth=4.5m

Length of d/s inverted filter= 1.5*d/s pile depth=4.5m so,

1.5m*1.5*m*0.7m concrete blocks are provided with 0.1m thick gap filled with bajri and 0.4 m thick graded filter

5. Design of Main Canal and Canal Structures

5.1 Design of main canal

An irrigation project a conveyance subsystem which includes open channel through the earth or rock formation, tunnel or pipelines. In context of Bhuteni irrigation project, open channel distribution system is used. For the proposed command area of 7250 ha canal system of 7.2 km is to be designed. The canal is designed to pass a peak discharge of 1.91 m^3/s. The distribution system is very long and in order to avoid the loss due to percolation losses canal lining is preferred.

5.1.1 Design consideration for canal

- Concrete lined canal with manning's coefficient 0.015
- For canal discharge of 1.91 m^3/s (< 85 m^3/s) triangular section with round bottom is considered.
- Longitudinal slope 1:900
- Canal side slope = 1:1.25
- Canal free board = 0.2m
- Lining Thickness = 0.1m

5.1.2 Design of canal cross section

Designing for a lined canal: As the canal is lined, we need to design the canal using the most economical canal design approach with least wetted perimeter. For a canal of Q<85 m^3/s a triangular section with round bottom is to be designed. The canal is designed for a longitudinal slope of 1:900 (V:H) and side slope of 1:1.25 (V:H) is assumed. For the concrete lining the value of manning's coefficient is assumed to be 0.015. Manning's equation along with the condition of most economical triangular section is used for the calculation of canal dimensions.

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

where,

R= A/P (hydraulic radius)

S= longitudinal slope of canal

Q= Canal discharge,

The designed canal parameters are:

i. Cross sectional area= 1.255m^2

ii. Wetted Perimeter=3.1442m

iii. Mean velocity =1.5225m/s

iv. Hydraulic Radius =.40363 m

v. Canal depth (y)= 0.808 m

vi. Free board= 0.2m [M8-P1 table 3.5.5]

5.1.3 Canal alignment selection:

Main canal is aligned along the road passing through the command area in the field.

Most sections were found to be aligned perpendicular to contour line i.e in north south direction

5.1.4 Canal longitudinal profile:

Canal longitudinal profile is fixed such that cut and fill gets balanced or fill is minimum. This can be achieved by excavating the canal at balancing depth but it is not always possible to excavate the canal at balancing depth. For such a long canal, guidelines suggested by S.K Garg are followed to set the canal's longitudinal profile. The canal in this project was fixed by hit and trial method, multiple profiles were made in with the aim to achieve minimum cut and fill out of those profiles, a profile with 299 m^3 of residual cut was selected which provided us a pond level of 99.117m.

5.1.5 Canal side embankment:

The canal side bank is designed as per the criteria suggested by M8 (canal design guideline).

Minimum Side slope =1.5:1(H: V) [M8-P1 Table 3.4.8]

Top width= 1.5m [M8-P1 Table 3.4.13]

5.1.6 Canal lining

As per the guidelines for concrete canal with discharge capacity less than 3 m^3/s the lining thickness of 8cm is suggested, but in our project lining of 10cm thick is adopted.

The area available for cultivation is greater than the area currently under cultivation. With an irrigation system, we could cultivate an area greater than twice the size of the currently cropped area. On one hand, the unlined canal system experiences seepage losses, which reduces the effective command area for irrigation. Additionally, during dry months, the flow in the canal is likely to seep, making irrigation difficult. Moreover, the cross-section area required for unlined canal is 5.98 times than that of lined canal. So, from economic point of view lining of canal is more economic and useful than unlined canal. So, we have chosen lined canal over unlined canal. The economic calculation is shown in **Appendix – I.3**

5.2 Settling basin

5.2.1 Introduction

Settling basin is a structure that allows large particles and a limited range of small particles to be trapped in it. The velocity in the settling zone is lowered to assist small particles to settle to the floor. Hydraulic conditions are maintained at inlet and outlet transition to prevent eddy current formation and in the scouring zone to prevent rescoring of the sediment particle. Also, at the flushing gates, hydraulic conditions are maintained to allow hydraulic flushing of sediment particles when gates are opened. To control the flow gate is provided in main canal too.

5.2.2 Design consideration

- The inlet and outlet transition are provided so as to avoid vortex formation.
- The smallest particle diameter to settle in the basin is taken as 0.5 mm.
- The particle with diameter of 0.9 mm scours during the opening of flushing gate.
- A sloping floor is provided to generate adequate high velocity to scour the sediment when flush gates are open.
- Silt flushing gates are provided for automatic flushing
- A lowered floor near the silt flushing gate to provide sufficient hydraulic head for efficient flushing of silt.
- A head regulator to control the flow of water into the main canal and to shut off flow entering the main canal during repair and emergency.
- A side spillway to automatically expel the excess water during river flood.

5.2.3 Design procedure

Using the nomograph of relation between different velocities and particle size as given in Figure 5.1, Figure 5.2 and Figure 5.3,the settling velocity, critical bottom velocity and scour velocity are determined. Knowing the discharge and velocity the depth is calculated as

Area = Q/ V

Where,

Q = discharge in m3/s

V = velocity in m/s

For plan Length to breadth ratio (L: B) is taken as 10:1

The required depth is calculated as Y = Q/ (B*v)

i. For scour bed slope(s), scour flow is assumed as 1.5*Q and is calculated using manning's eqn. We provide slope of 1/150.

ii. The height of gate (Yg is calculated using formula: $Q=1.7*B*Yg^{1.5}$

The dimension of settling basin is fixed as 34 m * 3.4m *2.8m.

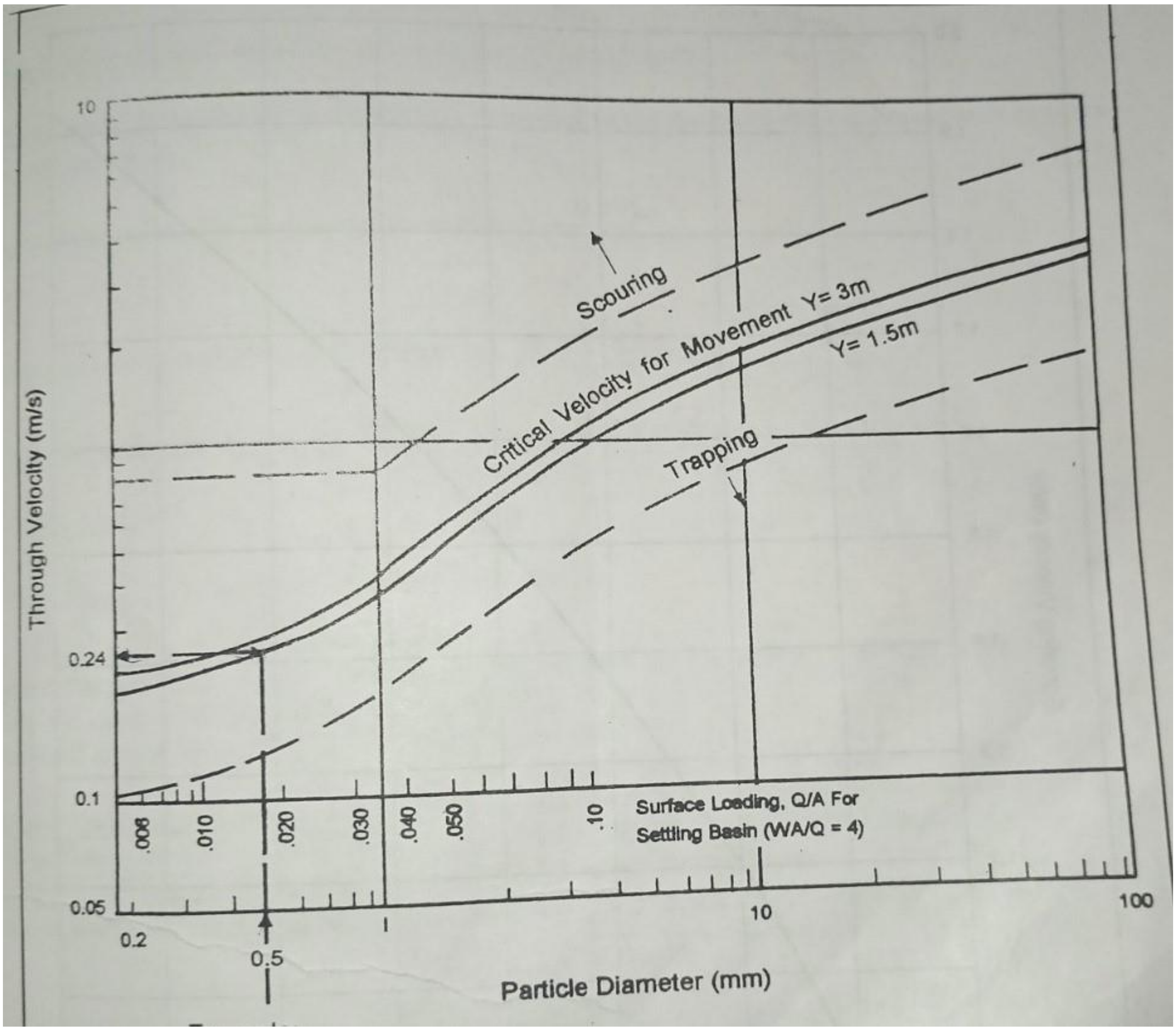


Figure 5.1:Graph of Through Velocity vs particle diameter

(source: Institute Of Engineering(1993), p.g.183)

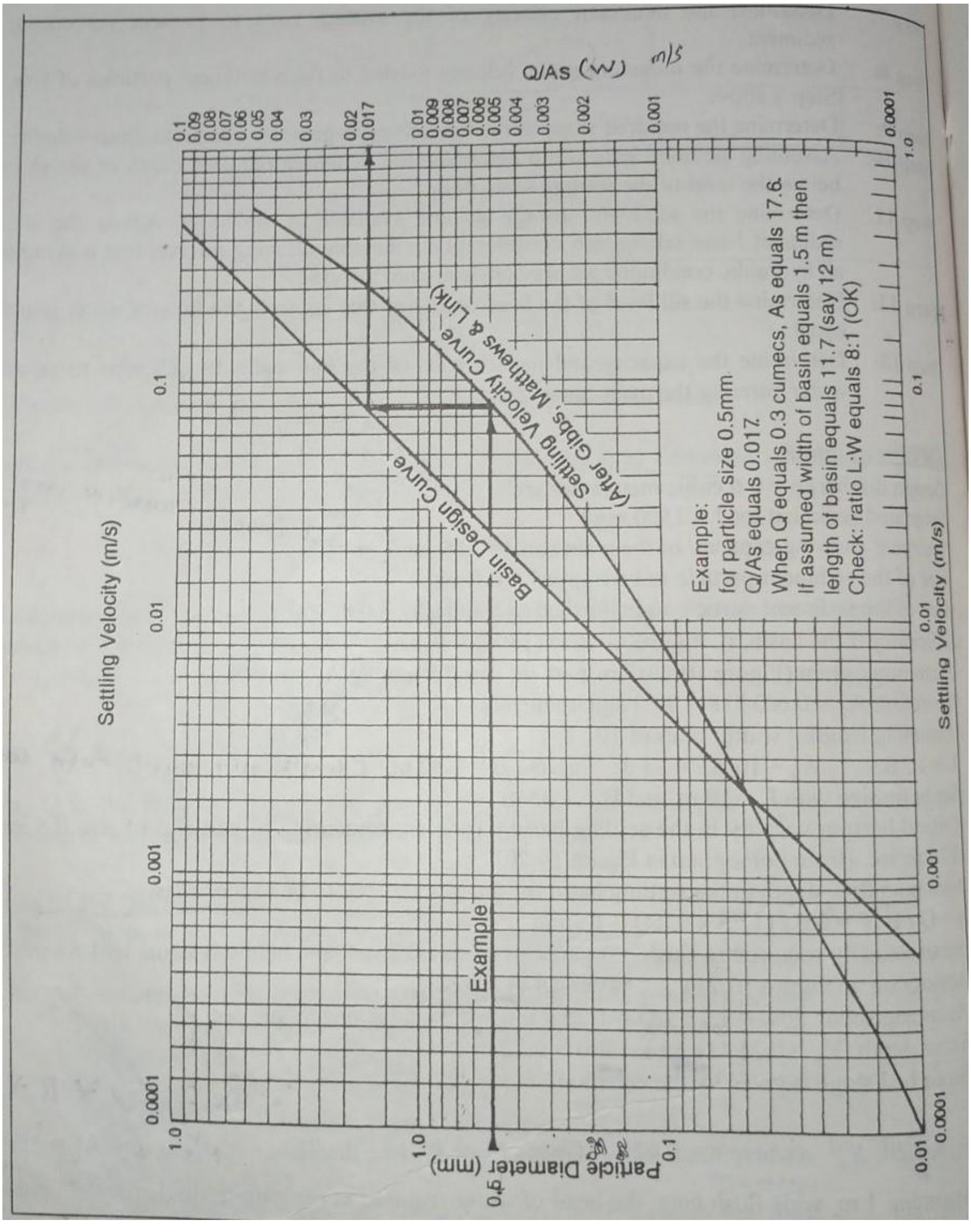


Figure 5.2 : Graph Of settling Velocity vs particle diameter

(source: Institute Of Engineering(1993), p.g.182)

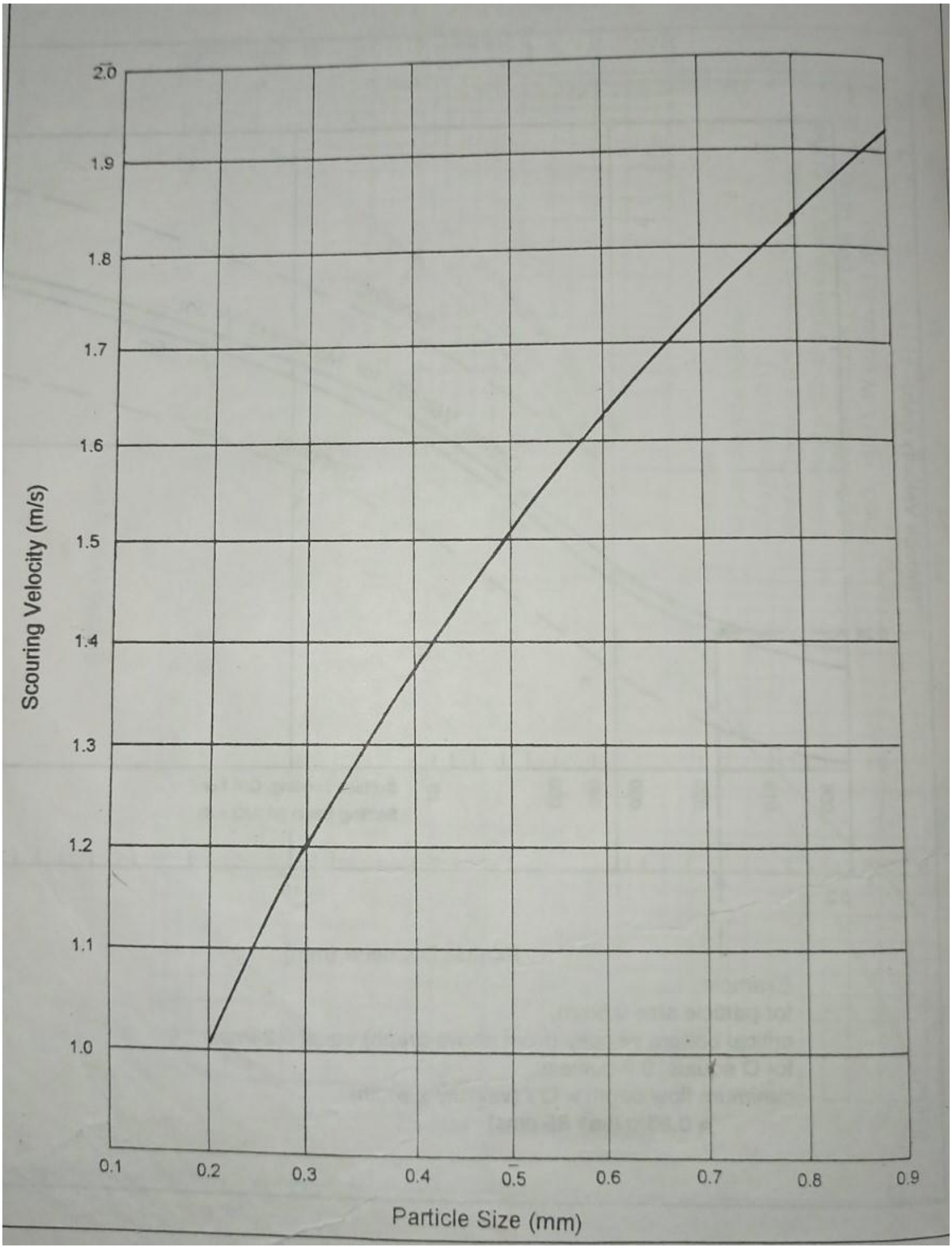


Figure 5.3 : Graph of scouring Velocity vs particle diameter

(Source :Institute Of Engineering(1993), p.g.184)

5.3 Design of drop structure (Sarda Fall)

5.3.1 Introduction

It is vertical drop fall where there is a raised crest an there is a vertical impact. It was named so because it was first evolved to replace notch valve in Sarda canal system in U.P. India. It is found economical than other because of its simple construction and design. It was found more efficient to use a greater number of small drops than to use fewer with large drops. The cistern in this type of fall acts as energy dissipation where there forms a small pondage and water strikes to the water itself.

5.3.2 Design consideration

- The approach canal for the drop is rectangular.
- For canal discharge less than 14 m^3/s, rectangular shaped crest wall is designed.
- Length of the crest width is taken equal to the bed width of canal.
- Drop is designed by using blingh's approach with coefficient of creep as 6.
- The material is concrete masonry with specific gravity 2.4.

5.3.3 Design procedure

The top width of the drop is given by:

$Q=1.84LH^{1.5}*(H/Bt)^{1/6}$ which was obtained as 0.6m

Where,

Q = discharge of canal in m^3/s

L = Length of canal or waterway in m

H = Head over crest in m

Bt = top width in m

For cistern design,

Cistern length (Lc) = $5(H*H_L)^{0.5}$

Depth of cistern (x) = $0.25*(H*H_L)^{0.67}$

where,

H_L = Head loss in m

The designed length of cistern is 4.06 m and the depth of cistern is 0.19 m.

The floor thickness and the depth of pile was designed using Bligh's theory, the total floor length adopted is 9.34 m. The design part is included in **annex -F** and the drawings are included in **annex - G**

Considering the requirement 1 m and 0.5 m drop are designed.

5.4 Design of a small road bridge and culvert

5.4.1 Introduction

Crossing of rivers to canals, roads to rivers can be solved by constructing culvert. In our case, when canal is constructed over a surface the natural streams have to cross the canal somewhere. So, for safe passing of the stream without mixing with our canal we construct Culverts. And for the safe passing of the vehicle and pedestrian we construct bridges.

5.4.2 Bridge design consideration and procedure

Bridge design is very complex and code based. The basic concept used to design a bridge, i.e. Effective width method is its self a conservative method. So, it's just an approximate design. It's a overkill method. Here the structural safety is guaranteed but cost effectiveness is low. During the design, the width selection is done so that tractors, which are frequently used can be passed easily and safely. Footpath on either side is provided for pedestrians crossing with proper handrail, to prevent fall in canals. Mainly this bridge is designed as per IRC (Indian Road Congress) Code using Indian standard.

During the design of an abutment, an RC abutment was chosen instead of a masonry abutment because the latter could not be installed due to its proximity to the canal, which could result in damage during canal construction or other work. And if it was taken with higher span, span could have been higher than 8m which needs Girder or truss bridges which design is very uneconomical as well as complex. So, we have adopted the buried type of Abutment which provides active earth pressure force that increases stability.

5.4.3 Culvert design consideration and procedure

When passing of the water is the primary purpose, construction of bridge is not economic. As bridges are complex and expensive structure, culverts solve these types of problem. Culverts are simple opening that passes through it. The passing of water is governed by downstream head, upstream head and friction during the flow through culvert.

Since in terai region, rainy season stream can have discharge up to 1m^3/s to 2m^3/s.

So, design discharge = 2 m^3/s

And culvert shape adopted= Box culvert

Thickness adopted at the top and side is 0.4m and 0.2 m at the bottom.

5.5 Design of cross regulator and distributary head regulator

5.5.1 Introduction

Distributary head regulators and cross regulators is important components of irrigation systems. They are used to control the flow of water and to ensure that it is distributed efficiently to the fields. Distributary head regulators are located at the head of a distributary and are used to regulate the flow of water into the distributary. Cross regulators are typically located at the intersection of two distributaries or at the end of a distributary. They are used to control the flow of water between the distributaries and to ensure that water is distributed evenly to the fields.

5.5.2 Design consideration

The canal has a triangular section, but the distributary cross and head regulator are of rectangular section (without altering the depth of water which is 0.808m), rectangular section provides a larger area for water to flow through, which reduces the velocity of water and prevents erosion of the structure. A slope is provided at d/s which lowers the Full Supply Level (FSL) of parent channel downstream by 0.2m. Crest: The Crest Level is the elevation of the crest or the top of the embankment, which is kept 0.3 meters higher than the bed level of the parent channel.

- Crest of cross regulator: Kept at the upstream bed level of the channel. Crest level of distributary head regulator: Positioned 0.3 m higher than the crest level of the cross regulator for silt entry control, flow division and control.
- Floor length calculation: Based on an exit gradient of 1/7, considering Khosla's criteria for fine sand 0.17 to 0.14 range because hydraulic gradient should not exceed this range to avoid erosion or instability concerns
- Upstream floor thickness: Theoretically, no thickness is required due to uplift being counterbalanced by water weight. However, a nominal thickness of 0.5 m is provided.
- Floor thickening under the crest: The floor is thickened to 1.0 m under the crest in a length equal to 1.3 m.

5.5.3 Design procedure

Discharge in Parent Channel (Q)

Discharge in distributary ($Q/5$)

Manning's coefficient (n)=0.015

Manning's equation,$Q = \frac{1}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}$ is sused, b = 1.9 is obtained

Free Board = 0.5 m is provided

Bed level of parent channel =FLS of parent Channel u/s -depth of water

Head Regulator:

Using Manning's equation d is obtained as 0.4738 m

FSL of distributary= Bed level of parent channel + d

Cross Regulator:

Using Manning's equation

b=1.6 m is provided

FSL of Parent Channel d/s = Bed level of parent channel +depth of water -0.2

A.) Design of Cross regulator and Distributary head regulator

Crest levels: Crest level of cross regulator is kept the same ass u/s bed level of parent channel as 98.043m

Waterway

$$Q = \frac{2}{3} C_{d_1} \sqrt{2g}.B\left[(h + h_v)^{3\backslash 2} - {h_v}^{3/2}\right] + C_{d_2}.B.h_1\sqrt{2g(h + h_v)}$$

where $C_{d_1} = 0.577$

$C_{d_2} = 0.8$

B = Clear water-way required

h=Difference of water level u/s and d/s of the crest, as shown in figure

h_1 = Depth of the d/s water level in the channel above the crest.

h_v = Head due to velocity of approach, which is very small and is generally ignored.

The discharge formula then becomes.

$$Q = B\sqrt{h}[1.69h + 3.54h_1]$$

In this case, h = u/s FSL -d/s FSL

$h_1 = d/s$ FSL - Crest Level

1 bay of 1.6 m is provided.

i. Downstream floor level or Cistern level

From Plate 10.1 Ef_2 is calculated

d/s floor level = d/s FSL – Ef_2

d/s Bed level = d/s FSL - W d

The Cistern or d/s floor at R.L 97.843 is provided.

ii. Length of d/s floor

a. Length Required = 5 $(y_2 - y_1)$

b. Exit gradient consideration = L = $\frac{2}{3}b$=$\frac{2}{3}$αd

Where

b= total length of impervious floor

d= downstream cut off

iii. Vertical cut-offs

U/s Cut-offs = $(y_u/3)$ + 0.6

Bottom level of u/s cutoff

D/s Cut-offs = $(y_d/2)$+0.6

Bottom level of d/s cutoff is calculated as 96.84

Total Floor Length from Exit Gradient Considerations

Total floor length b is worked out from exit gradient consideration.

iv.. Floor thickness

Floor Thickness is calculated in d/s, for that maximum unbalanced head due to static head and head due to dynamic action is considered and thickness is calculated, considering the greater head among head due to static and dynamic action.

Head Due to Dynamic action can be taken as = $50\% \ (y_2 - y_1) + \emptyset * HL$

v. Launching apron

The quantity of stone in launching apron is kept as Launching apron (2.25*D)/t m^3/m

5.5.4 Results

Design Parameters shown in Table 5.1 and Table 5.2 is obtained from the design of distributary head regulator. And cross regulator

Table 5.1 : Design Parameter for Head Regulator

Parameter	**Value**
Bays of Clear Water Way	1.6m
Crest Level	98.343 m
Length of d/s Glacis	0.3 m
Length of crest	1.0 m
Length of u/s Glacis	0.3 m
Thickness at toe of glacis	1.3 m
Thickness beyond 1.17 m from toe	1.2 m
Thickness beyond 2.33 m from toe	0.6 m
Thickness beyond 3.50 m from toe	0.3 m
Launching apron required U/S	4.3 m
Launching apron required D/S	3.4 m

Table 5.2 : Design Parameter for Cross Regulator

Parameter	**Value**
Bays of Clear Water Way	1.6 m
d/s Floor Level	97.451 m
d/s Bed Level	97.843 m
Length of d/s floor	3.0 m
Crest Level	98.043 m
Glacis length	0.4 m
U/S floor length	2.9 m
Thickness at toe of glacis	1.3 m
Thickness at 2 m from toe of glacis	1.14 m
Thickness at 5 m from toe of glacis	0.9 m
Launching apron required U/S	4.9 m
Launching apron required D/S	5.6 m

Crest of cross regulator is kept at the upstream bed level of the channel. While, the crest level of the distributary head regulator is kept 0.3 m higher than the crest level of the cross regulator. Floor length is calculated considering exit gradient as 1/7 using graph given in Figure 5.4. Theoretically no floor thickness is required under the upstream floor, since the uplift is more than counterbalanced by the weight of the water standing over it. But a nominal thickness of 0.5 m. is provided. The floor is thickened to 1.0 m under the crest in a length equal to 1.3 m.

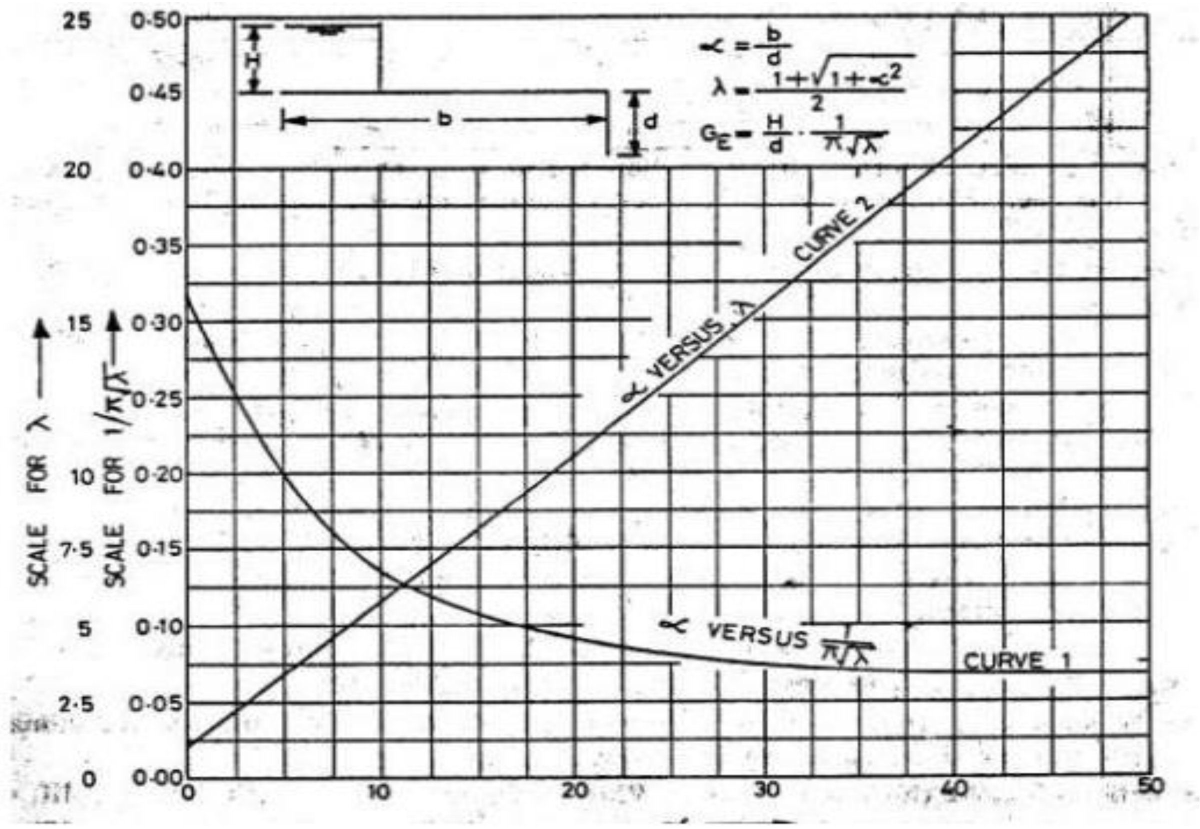


Figure 5.4 : Khosla’s safe exit gradient curve

source: P. C., & Devkota, N. (2074),pg 267

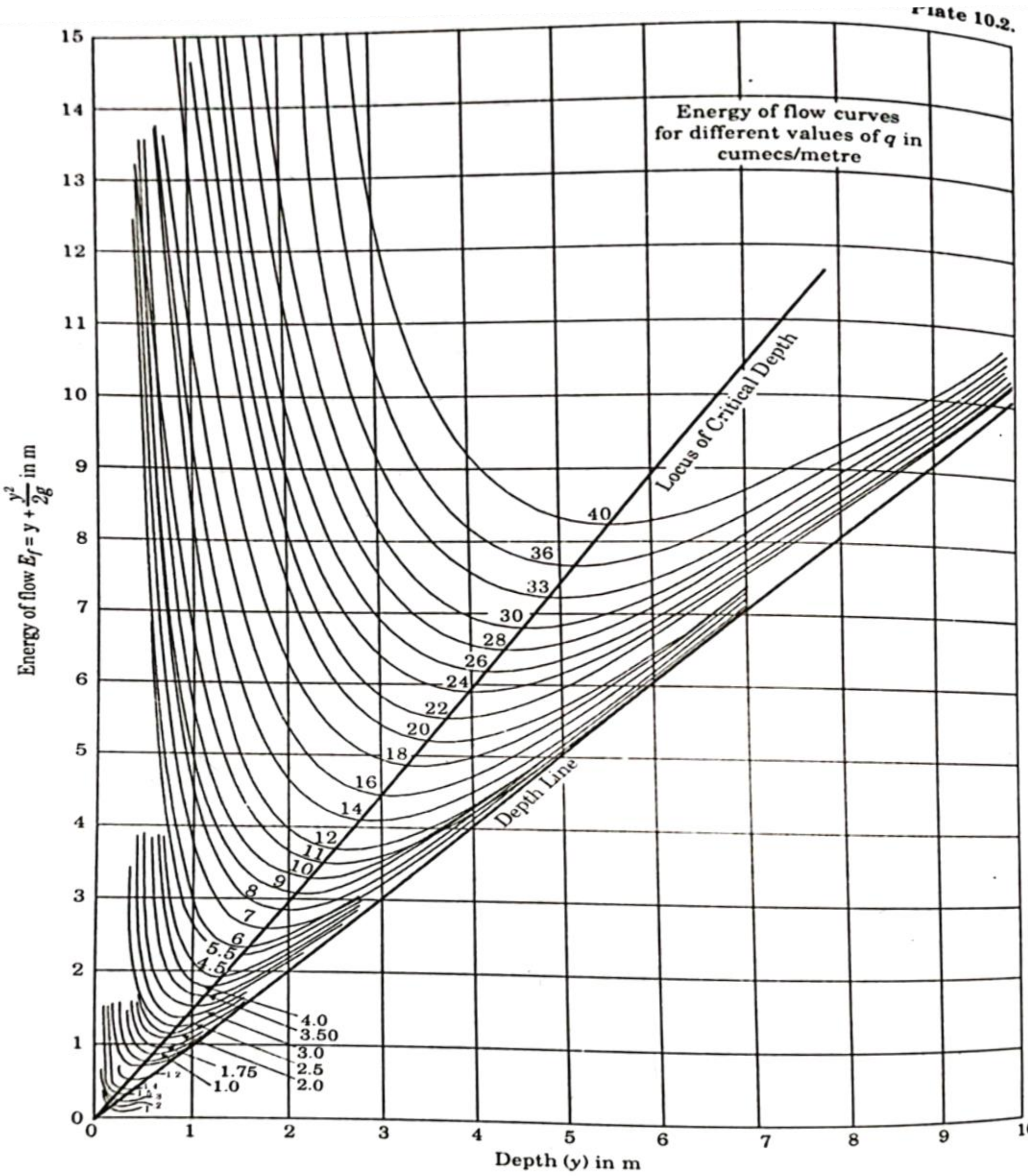


Figure 5.5 : Khosla's Energy Curve

source:Jha, P. C., & Devkota, N. (2074),pg 269

5.6 River training work

Shear wall are vertical and slim type of wall that resist lateral load. Generally, for a higher structure, the section constructed is flattened at the bottom so that it can resist more load and become more stiffened. But the problem is that more materials are required for that activity. And even if we can provide the materials, space problem may occur. So, in a restricted space and high wall requirement, shear walls are constructed. These walls have very high stiffness due to use of reinforcement.

Shear wall of thickness 0.5m is constructed in the edge of the weir so that it gives a box like channel that leads to the weir and also support the lateral load given by water (Static), water current and wind. There was a bridge at 65 m upstream of the headwork so the length of the wall designed is 65 m on each side and depth of wall designed is 4.5 m. The high flood level at upstream is 3.3 m and 1 m of wall goes below the ground level. As there is no vertical load on shear wall 1 m depth will act as the footing so the depth is provided of 4.5 m. The total volume was 221.9 m^3. For cost estimate PCC was taken of total volume and RCC 1% of the total volume which is shown in **Appendix – H.** It also guides the water to the weir so it is also used as guide bund

6. Financial Analysis

6.1 Benefit analysis

The income of the project solely depends on the production of the crops. The total gain is obtained from revenue through production deducting all the expenses. The factors which play role are production and expenses. Under production it is the yield of crop and revenue obtained from by-product. Some crops like paddy, maize has by product in huge amount which are good food for cattle. The expenses factors are:

a. Seed

b. Organic manure

c. Chemical fertilizer like Urea, DAP, Potash

d. Labor and bullock

The rate is obtained from department of irrigation which is mentioned as in Table 6.1:

Table 6.1 : Revenue through crops

Crops	**Monsoon Paddy**	**Vegetables Winter**	**Pulses**
Production (t/ha)	**2**	**8**	**3**
Price (Rs/t)	25000	35000	65000
Value (Rs/ha)	50000	280000	195000
By product (Rs/ ha)	30000		
A) Gross value of Production (Rs/ ha)	**80000**	**280000**	**195000**
Expenses			
a. Seed (kg/ha)	60	0.8	21
Price (Rs/Kg)	50	50000	70
Value (Rs/ha)	3000	40000	1470
b. Organic Manure (t/ha)	4.2	7	1.4
Price (Rs/t)	2500	2500	2500

Value (Rs/ha)	10500	17500	3500
c. Chemical Fertilizer			
i) Urea (Kg/ha)	113.28	61.95	61.95
Price (Rs/Kg)	20	20	20
Value (Rs/ha)	2265.6	1239	1239
ii) DAP (Kg/ha)	0	32.5	32.5
Price (Rs/Kg)	50	50	50
Value (Rs/ha)	0	1625	1625
iii) Potash (Kg/ha)	0	25	25
Price (Rs/ha)	40	40	40
Value (Rs/ha)	0	1000	1000
iv) Pesticides (Rs/ha)	1400	3000	200
d) Labor md/ha	104	120	64
Price (Rs/md)	400	400	400
Value (Rs/ha)	41600	48000	25600
e) Bullock ad/ha	25	25	25
Price Rs/ ad	500	500	500
Value (Rs/ha)	12500	12500	12500
B) Total Cost of Production (Rs/ha)	**71265.6**	**124864**	**47134**
C) Benefit per ha (A-B) Rs/ha	8734.4	155136	147866

(Source: Department of Irrigation, Lalitpur)

Here the benefit is the revenue generated after deducting the expenses which is in Rs/ha. The total benefit is obtained by multiplying with the command area. The benefit is calculated for both conditions with project and without the project. Thus, net benefit is obtained by subtracting benefit without project from benefit with project.

6.1.1 Crop Revenue Without Project

Without project, it was found approximately 300 ha of land cultivate the monsoon paddy and no other crop planting was possible.

Without project crop revenue is Rs. 300 * 8734.4 **= Rs. 26,20,320**

6.1.2 Crop Revenue with Project

With project, we go for 3 types of crops. For each type of crops the command area is varied according to the amount of water we have. We cropped 725 ha of land by monsoon paddy, 300 ha of land by winter vegetables and 150 ha of land by pulses

With project crop revenue is Rs. 725*8734.4 + 300*155136 + 150*147866

= Rs.**7,50,53,140**

The net benefit = Rs. 75053140 – Rs. 2620320 = **Rs. 7,24,32,820**

Thus, total benefit is **Rs. 7,24,32,820 per year**

6.2 Cost Estimate

Cost Estimate of structure is done using unit rate method. Amount for conducting key activities is estimated using estimation guidelines, materials type used in those activities, material quantity and their unit rates fixed by the district. Most of the factor's consideration that a construction work needs are added to the unit rates such as overhead cost, profit, cost for hire of toolset and equipment due to unskilled worker .so, it gives cost estimate of the project with acceptable level of reliable Even loses are considered as per the guidelines given which makes it more realistic approach of estimation. For calculation, please refer **Annex H**

The different type of key activities and their unit rate are as follows:

- PCC (1:1:2) for RCC @ Rs.18,925 per m3
- PCC (1:1:2) @ Rs.6715 per m3
- PCC (1:2:4) for RCC @ Rs. 17480 per m3
- Iron Work @ Rs. 138 per kg
- Excavation of Earthwork @ 474 per m3
- Filling of Earthwork @ 474 per m3
- Gravel Filling @ Rs.1400 per m3

6.3 Cost Benefit Analysis:

Every project must be a profit generating. Even though large sum of money is spent at the construction and initiation of the project, the revenue eventually tops the project cost and start generating profit. In this project, the evaluation period is taken as 20yrs through general adaptation. And calculation is given in Table 6.2 and break-even analysis is shown in Figure 6.1

Table 6.2 : Cost Benefit Analysis

PW of Cost (×10^6) (Rs.)	PW of Revenue (×10^6) (Rs.)	Cumulative cost (×10^6) (Rs.)	Cumulative Revenue (×10^^{6}) (Rs.)
56.38	0.00		0.00
38.44	0.00	94.83	0.00
34.95	16.33	129.77	16.33
3.34	49.47	133.12	65.80
3.04	44.98	136.16	110.77
2.76	40.89	138.92	151.66
2.51	37.17	141.44	188.83
2.28	33.79	143.72	222.62
2.08	30.72	145.80	253.34
1.89	27.93	147.68	281.26
1.72	25.39	149.40	306.65
1.56	23.08	150.96	329.73
1.42	20.98	152.38	350.71
1.29	19.07	153.67	369.79
1.17	17.34	154.84	387.13
1.07	15.76	155.91	402.89
0.97	14.33	156.88	417.22
0.88	13.03	157.76	430.25
0.80	11.84	158.56	442.09
0.73	10.77	159.28	452.86

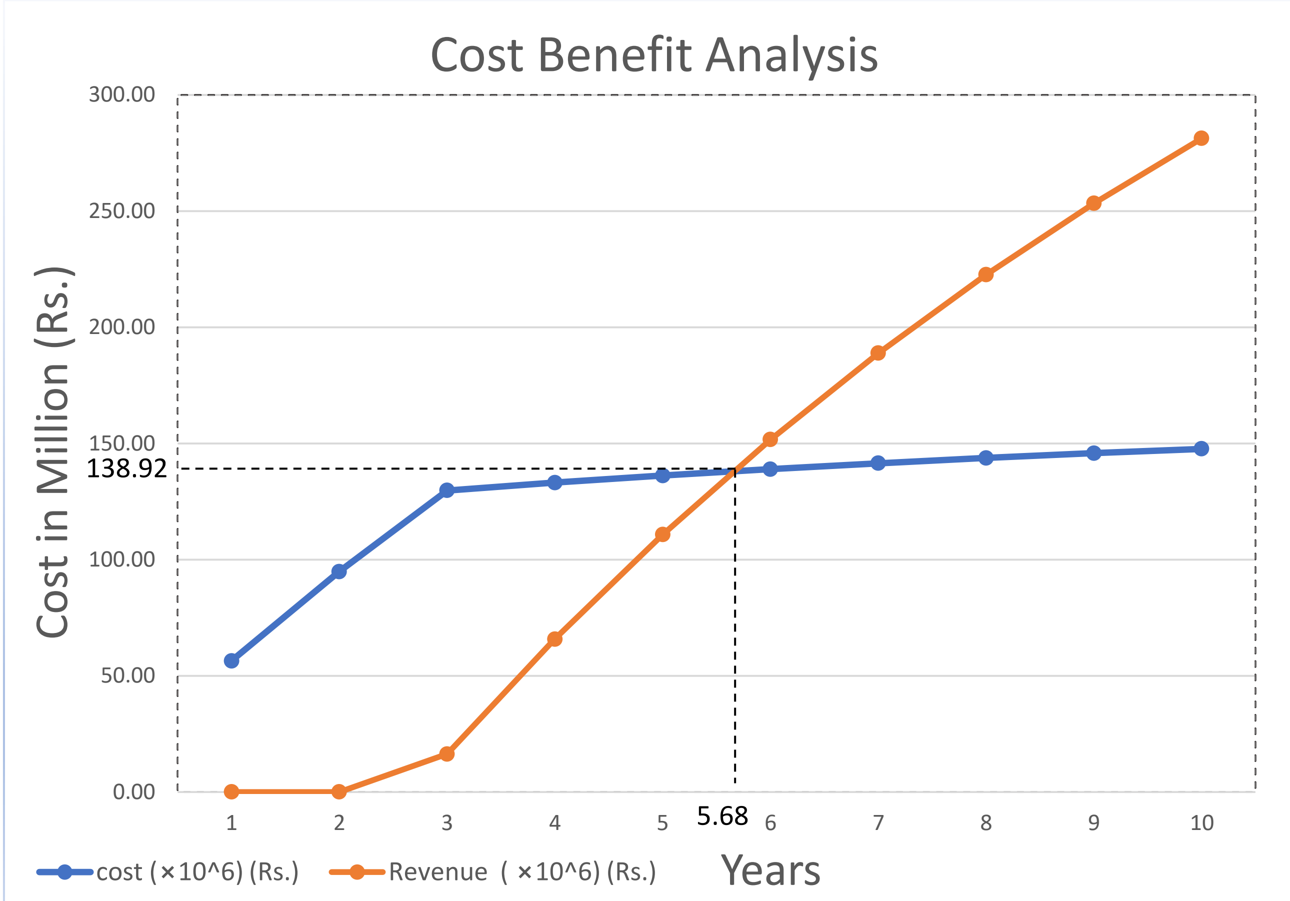


Figure 6.1: Cost Benefit Analysis

From Graph:

Breakeven Period = 5.68 years.

6.4 B/C ratio and EIRR

For any type of irrigation project, generally B/ ratio is calculated for up to 20 years because a project generally goes for rehabilitation after it and the cost estimate would be different. Its tabulated calculation is given in Table 6.3.

Table 6.3 : Calculation of B/C and EIRR

S.N.	Cash out flows	Benefit	Net cash flow	NPW at 10%	NPW at 12%
(year)	NRs	(NRs)	(NRs)	(NRs)	(NRs)
1	62021695.14	0.00	-62021695.14	-56383359.22	-55376513.52
2	46516271.35	0.00	-46516271.35	-38443199.46	-37082486.73

3	46516271.35	21729846.00	-24786425.35	-18622408.23	-17642487.98
4	4896449.62	72432820.00	67536370.38	46128249.7	42920584.33
5	4896449.62	72432820.00	67536370.38	41934772.45	38321950.29
6	4896449.62	72432820.00	67536370.38	38122520.41	34216027.05
7	4896449.62	72432820.00	67536370.38	34656836.74	30550024.15
8	4896449.62	72432820.00	67536370.38	31506215.22	27276807.28
9	4896449.62	72432820.00	67536370.38	28642013.83	24354292.21
10	4896449.62	72432820.00	67536370.38	26038194.39	21744903.76
11	4896449.62	72432820.00	67536370.38	23671085.81	19415092.64
12	4896449.62	72432820.00	67536370.38	21519168.92	17334904.15
13	4896449.62	72432820.00	67536370.38	19562880.84	15477592.99
14	4896449.62	72432820.00	67536370.38	17784437.12	13819279.45
15	4896449.62	72432820.00	67536370.38	16167670.11	12338642.37
16	4896449.62	72432820.00	67536370.38	14697881.92	11016644.97
17	4896449.62	72432820.00	67536370.38	13361710.84	9836290.15
18	4896449.62	72432820.00	67536370.38	12147009.85	8782401.92
19	4896449.62	72432820.00	67536370.38	11042736.23	7841430.29
20	4896449.62	72432820.00	67536370.38	10038851.12	7001277.04
Total			1014793904.69	293573268.6	232146656.82
EIRR =	19.56	>MARR (12%)			

EIRR = 19.5 % which is greater than MARR (Minimum Acceptable Rate of Return).

It is the ratio of benefit to cost in present value of future cost or future value of present cost or in annuity. B/C ratio of the project is 2.84 > 1.

Hence, this project is financially feasible.

7. Conclusions and Recommendations

7.1 Conclusions

Based on the pre-feasibility study of the Bhuteni Irrigation project, followings are the conclusion:

- Irrigation water requirement study was carried out for finding the canal discharge, by adopting a suitable Cropping Pattern i.e., winter paddy followed by maize and pulses in command area of 750 ha. River discharge study and calculation was conducted to meet the demand. And the maximum irrigation requirement for a particular half month gives the canal discharge which is 1.91 m^3/s.
- Hydraulic Design of Weir (with gated system) was designed for diversion with 0.7m of weir and 0.7 m gate, Under Sluice was designed of 3.2 m width for passing 20% of flood through it, Canal Head Regulator of frontal type was designed, Settling Basin of 34m length 3.4m width 2.8m high was designed for excluding silt entry to canal, Canal of 7.5 km was designed along the command area, Drop Structure was designed as per required topography of 0.5m, 0.8 and 0.9m, 3 Distributary Head Regulator and Distributary Cross Regulator were designed for secondary canals, Shear wall of 65m length, 4.5m high and 0.5m width was designed for river training work and to prevent nearby lands during floods, 7 Culverts and 3 Bridges were designed were canal crossings were found..
- Cost of Project was estimated using unit rate method which was estimated to 163.21 million Nepalese Rupees and Annual revenue generated from design system is 72.43 million Nepalese Rupees. For the span of 20years as evaluation period the B/C ratio is 2.84>1. The EIRR for the project is 19.5% > MARR (12%). The payback period for the project is 5.68 years. Thus, from all analysis criteria it is concluded that this project is financially feasible.

7.2 Recommendations

for further advancing of study in this project following are recommended:

- Detailed topography mapping of the headworks area and Study of the river morphology should be conducted. For precise hydrological analysis.

- Selection of location for distributary head regulator, canal cross regulator, secondary and Tertiary Canal. And Sediment sampling and soil type investigation shall be carried out.
- Detailed Environment impact assessment, Social and environmental feasibility should be studied.

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Appendix A:
Maximum Daily Rainfall at representative stations

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Appendix A

A.1 : Sample calculation for maximum daily rainfall at Anarmani station:

A.1.1 Gumbel Method:

The value of Y_n and S_n are interpolated from the tables shown below and are used in the frequency analysis of Gumbel method.

Table A.1.1: Reduced mean Y_n in Gumbel's extreme value distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Table A.1.2: Reduced mean S_n in Gumbel's extreme value distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Table A.1.3: Mean and S.D Calculation

SN	Xi	$(Xi-X)^2$	SN	Xi	$(Xi-X)^2$
1	58.5	10950.610	16	134.1	843.621
2	285.5	14970.707	17	239.7	5860.643
3	84	6263.957	18	191.3	792.695
4	135	792.150	19	176.5	178.352
5	178	220.666	20	146.5	277.061

6	168	23.569	21	168.4	27.613
7	168	23.569	22	160.3	8.095
8	175	140.537	23	151.7	130.992
9	210	2195.376	24	136	736.860
10	196.2	1092.622	25	138	632.279
11	166.2	9.332	26	138.5	607.384
12	137.7	647.456	27	222.6	3534.878
13	130	1098.602	28	185.2	486.416
14	178.1	223.647	29	150.2	167.577
15	152	124.215	30	104.2	3474.532
			31	192.1	838.383
	Sum=	57374.397			
n=31					
average=	X=	163.145			
	σ =	43.732			

yn =	0.5371	Sn =	1.1159

Table A.1.4: Gumbel Analysis

Observed		Gumbel				
Rainfall	Rainfall (Desc)	Rank (m)	T=(n+1)/m	Yt=- ln(ln(T/T-1))	K_T= ((YT-Yn)/Sn)	X_T=X+KT *σ
58.5	285.5	1	32	3.450	2.61	277.285
285.5	239.7	2	16	2.740	1.97	249.297
84	222.6	3	10.67	2.319	1.6	233.116
135	210	4	8	2.013	1.32	220.871
178	196.2	5	6.4	1.773	1.11	211.688
168	192.1	6	5.34	1.573	0.93	203.816
168	191.3	7	4.58	1.401	0.77	196.819
175	185.2	8	4	1.246	0.64	191.134
210	178.1	9	3.56	1.109	0.51	185.448
196.2	178	10	3.2	0.982	0.4	180.638
166.2	176.5	11	2.91	0.865	0.29	175.827
137.7	175	12	2.67	0.757	0.2	171.892
130	168.4	13	2.47	0.656	0.11	167.956
178.1	168	14	2.29	0.555	0.02	164.020
152	168	15	2.14	0.462	-0.07	160.084
134.1	166.2	16	2	0.367	-0.15	156.585
239.7	160.3	17	1.89	0.284	-0.23	153.087
191.3	152	18	1.78	0.192	-0.31	149.588

176.5	151.7	19	1.69	0.110	-0.38	146.527
146.5	150.2	20	1.6	0.019	-0.46	143.028
168.4	146.5	21	1.53	-0.058	-0.53	139.967
160.3	138.5	22	1.46	-0.144	-0.61	136.469
151.7	138	23	1.4	-0.225	-0.68	133.407
136	137.7	24	1.34	-0.316	-0.76	129.909
138	136	25	1.28	-0.419	-0.86	125.536
138.5	135	26	1.24	-0.496	-0.93	122.474
222.6	134.1	27	1.19	-0.607	-1.03	118.101
185.2	130	28	1.15	-0.711	-1.12	114.165
150.2	104.2	29	1.11	-0.838	-1.23	109.355
104.2	84	30	1.07	-1.003	-1.38	102.795
192.1	58.5	31	1.04	-1.181	-1.54	95.798

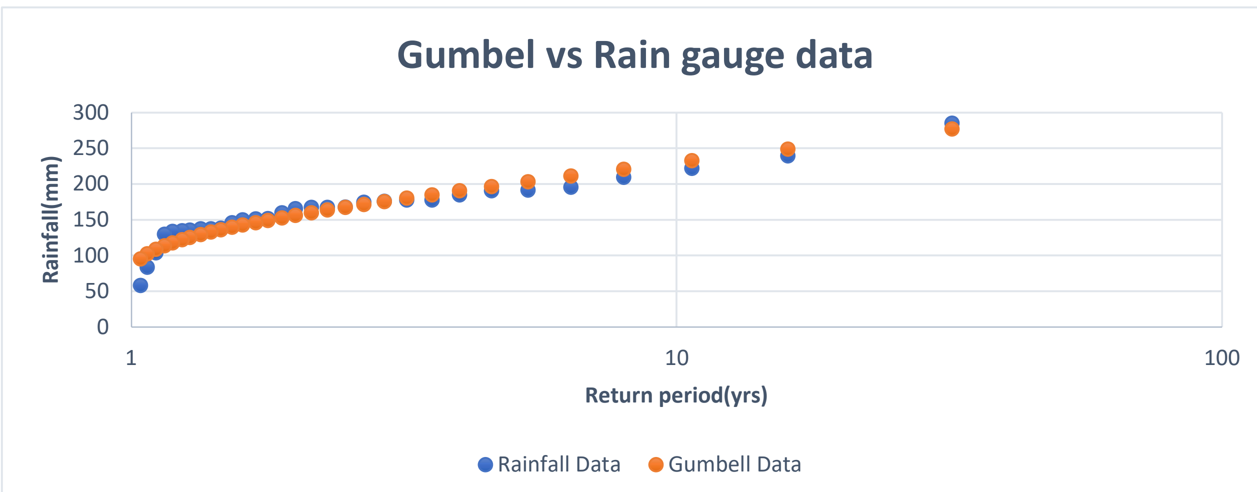

Figure A.1.1 : Graph for Gumbel Method Analysis

Table A.1.5 : Correlation Calculation for Gumbel

Return period(T)	Y_T=-ln(ln(T/T-1))	K_T=((YT-Yn)/Sn)	XT=X+KT*std	
10	2.26	1.54	230.5	
20	2.98		2.19	258.92
33	3.49		2.65	279.04
50	3.91		3.02	295.22

100	4.61	3.65	322.77
200	5.3	4.27	349.89
300	5.71	4.64	366.07
500	6.22	5.09	385.75

We get , Correlation(r2)= 0.9665

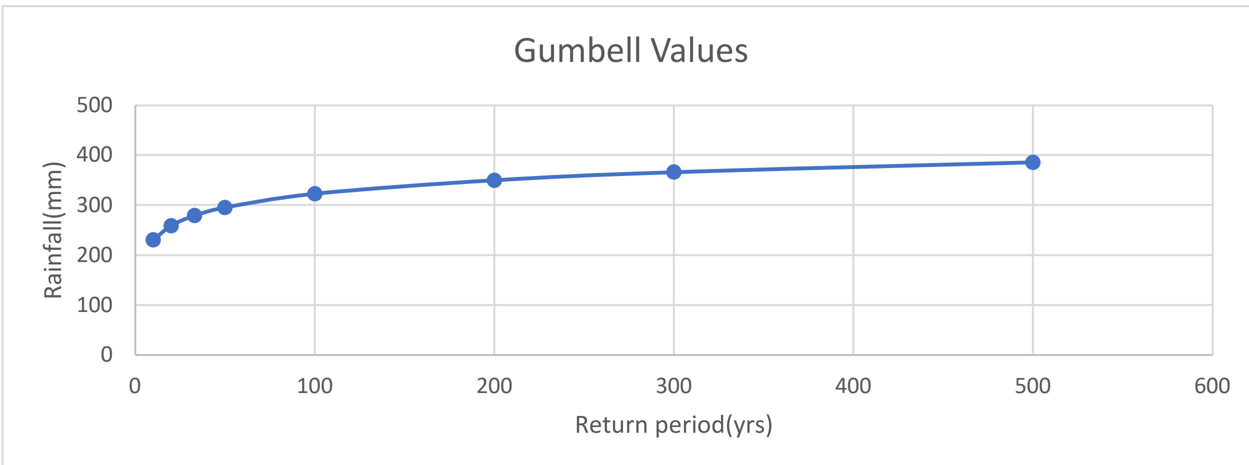

Figure A.1.2 : Gumbel Rainfall Values for different return period

A.2 Log normal method:

$$K_T = z = w - \frac{2.515517+0.802853w+0.010328w2}{1+1.432788w+0.189269w2+0.001308w3}$$

K_T=z=w-a/b

Table A.2.1 : S.D. Calculation for Log normal Method

S.N.	Y=logx	$(y-yi)^2$	S.N.	Y=logx	$(y-yi)^2$
1	1.767	0.184	17	2.380	0.034
2	2.456	0.068	18	2.282	0.007
3	1.924	0.074	19	2.247	0.003
4	2.130	0.004	20	2.166	0.001
5	2.250	0.003	21	2.226	0.001
6	2.225	0.001	22	2.205	0.000
7	2.225	0.001	23	2.181	0.000
8	2.243	0.002	24	2.134	0.004

9	2.322	0.016	25	2.140	0.003
10	2.293	0.009	26	2.141	0.003
11	2.221	0.001	27	2.348	0.023
12	2.139	0.003	28	2.268	0.005
13	2.114	0.007	29	2.177	0.000
14	2.251	0.003	30	2.018	0.032
15	2.182	0.000	31	2.284	0.008
16	2.127	0.005			

Sum= 0.50378 Avg= 2.19561742 σ = 0.12958652

Table A.2.2: Log normal Analysis I

Observed		Log Normal								
rainfall	rainfall(des)	rank(m)	T=(n+1)/m	p=1/T	$w=(\ln(1/p^2))^{0.5}$	a	b	kT=w-a/b	Yt=Y+kT*S	$Xt=10^{Yt}$
58.5	285.5	1	32	0.031	2.633	4.701	6.109	1.863	2.437	273.584
285.5	239.7	2	16	0.063	2.355	4.464	5.441	1.535	2.394	248.020
84	222.6	3	10.67	0.094	2.176	4.311	5.027	1.318	2.366	232.523
135	210	4	8	0.125	2.04	4.196	4.722	1.151	2.345	221.210
178	196.2	5	6.4	0.156	1.927	4.101	4.473	1.010	2.327	212.093
168	192.1	6	5.34	0.187	1.831	4.020	4.266	0.889	2.311	204.537
168	191.3	7	4.58	0.218	1.745	3.948	4.083	0.778	2.296	197.907
175	185.2	8	4	0.250	1.666	3.882	3.918	0.675	2.283	191.926
210	178.1	9	3.56	0.281	1.594	3.822	3.770	0.580	2.271	186.562
196.2	178	10	3.2	0.313	1.526	3.765	3.632	0.489	2.259	181.568
166.2	176.5	11	2.91	0.344	1.462	3.711	3.503	0.403	2.248	176.927
137.7	175	12	2.67	0.375	1.402	3.661	3.384	0.320	2.237	172.625
130	168.4	13	2.47	0.405	1.345	3.614	3.273	0.241	2.227	168.581
178.1	168	14	2.29	0.437	1.288	3.567	3.162	0.160	2.216	164.574
152	168	15	2.14	0.467	1.234	3.522	3.059	0.083	2.206	160.811
134.1	166.2	16	2	0.500	1.178	3.476	2.953	0.001	2.196	156.939
239.7	160.3	17	1.89	0.529	1.129	3.435	2.861	-0.072	2.186	153.574
191.3	152	18	1.78	0.562	1.074	3.390	2.759	-0.155	2.176	149.820
176.5	151.7	19	1.69	0.592	1.025	3.349	2.669	-0.230	2.166	146.494
146.5	150.2	20	1.6	0.625	0.97	3.304	2.569	-0.316	2.155	142.778
168.4	146.5	21	1.53	0.654	0.923	3.265	2.485	-0.391	2.145	139.614

160.3	138.5	22	1.46	0.685	0.87	3.222	2.391	-0.478	2.134	136.056
151.7	138	23	1.4	0.714	0.821	3.182	2.305	-0.560	2.123	132.773
136	137.7	24	1.34	0.746	0.766	3.137	2.209	-0.654	2.111	129.090
138	136	25	1.28	0.781	0.703	3.085	2.101	-0.765	2.096	124.870
138.5	135	26	1.24	0.806	0.656	3.047	2.022	-0.851	2.085	121.716
222.6	134.1	27	1.19	0.840	0.59	2.993	1.911	-0.976	2.069	117.269
185.2	130	28	1.15	0.870	0.529	2.943	1.811	-1.096	2.054	113.132
150.2	104.2	29	1.11	0.901	0.457	2.885	1.694	-1.245	2.034	108.201
104.2	84	30	1.07	0.935	0.368	2.812	1.553	-1.443	2.009	102.007
192.1	58.5	31	1.04	0.962	0.281	2.742	1.418	-1.653	1.981	95.804

Table A.2.3 : Log Normal correlation Analysis

Return period(T)	p=1/T	$w=(\ln(1/p^2))^{0.5}$	a	b	kT=z=w-a/b	Yt=Y+kT*S	$Xt=10^{Yt}$
10	0.1	2.146	4.286	4.959	1.282	2.362	229.995
20	0.05	2.448	4.543	5.661	1.646	2.409	256.362
33	0.030	2.645	4.711	6.138	1.877	2.439	274.732
50	0.02	2.798	4.843	6.519	2.055	2.462	289.694
100	0.01	3.035	5.047	7.128	2.327	2.497	314.166
200	0.005	3.256	5.239	7.717	2.577	2.530	338.511
300	0.003	3.378	5.345	8.050	2.714	2.547	352.625
500	0.002	3.526	5.475	8.462	2.879	2.569	370.428
25	0.04	2.538	4.620	5.877	1.752	2.423	264.633

Correlation(r2) = 0.9649

Appendix A

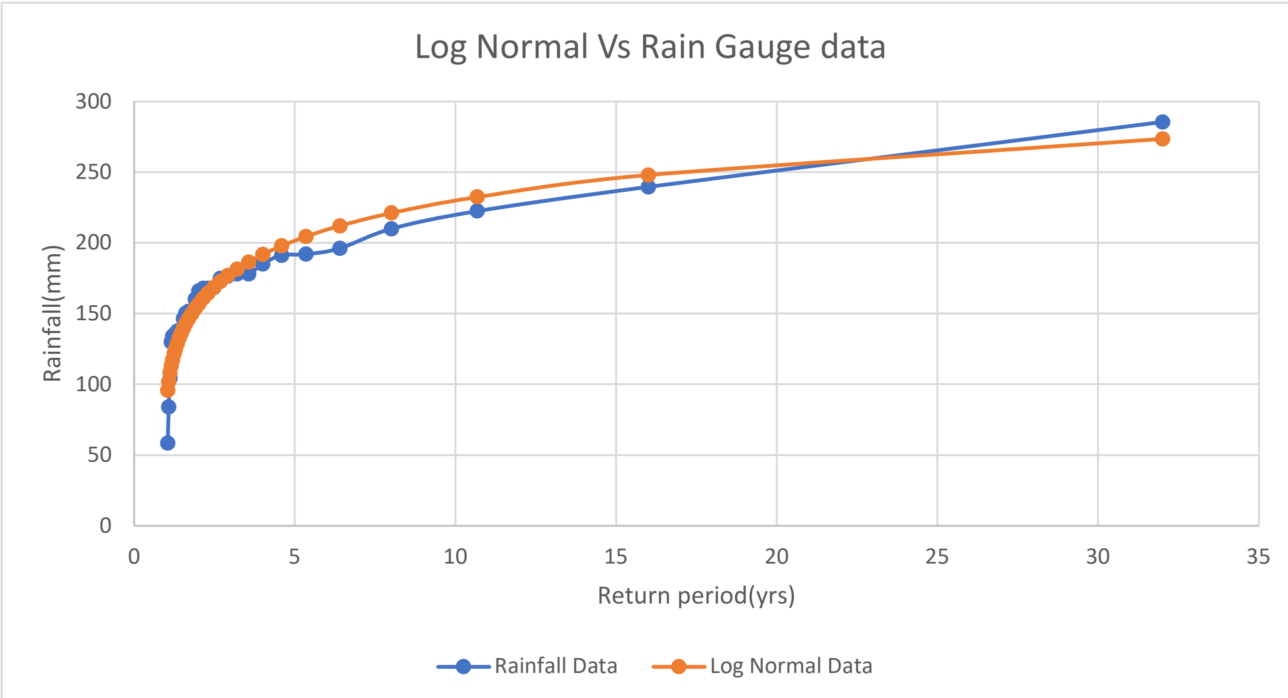

Figure A.2.2 : Graph of Log normal Analysis

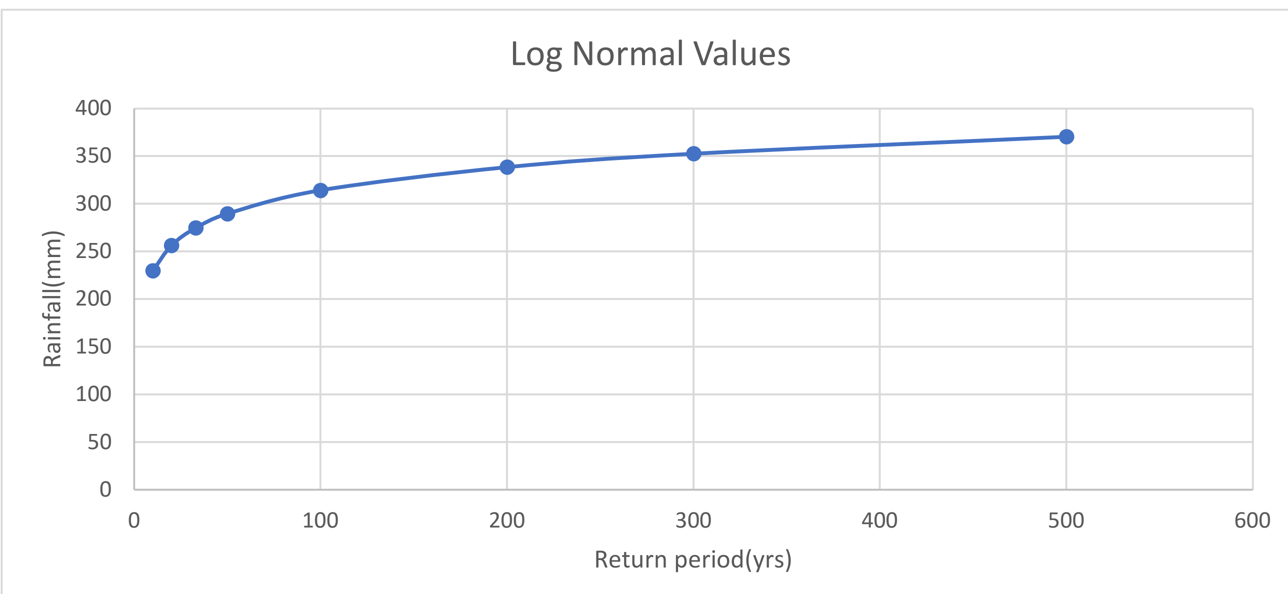

Figure A.2.1 : Log normal Rainfall for different return period

A.3 Log Pearson III method:

$$z = w - \frac{2.515517+0.802853w+0.010328w2}{1+1.432788w+0.189269w2+0.001308w3}$$

$$\boldsymbol{Cs} = \frac{N\Sigma(y-\bar{y})^3}{(N-1)(N-2)\sigma^3}$$

Table A.3.1 : Log Pearson III analysis

year	Y=logx	(y-yi)2	(y-y¯)^3		Y=logx	(y-yi)2	(y-y¯)^3
1	1.881	0.089	-0.027	17	2.302	0.015	0.002
2	2.370	0.036	0.007	18	2.240	0.004	0.000
3	2.081	0.010	-0.001	19	2.172	0.000	0.000
4	2.001	0.032	-0.006	20	2.247	0.005	0.000
5	2.029	0.023	-0.003	21	2.260	0.006	0.001
6	2.227	0.002	0.000	22	2.149	0.001	0.000
7	2.023	0.024	-0.004	23	1.997	0.033	-0.006
8	2.132	0.002	0.000	24	2.106	0.005	0.000
9	2.106	0.005	0.000	25	2.410	0.053	0.012
10	2.205	0.001	0.000	26	2.212	0.001	0.000
11	1.988	0.037	-0.007	27	2.306	0.016	0.002
12	2.090	0.008	-0.001	28	2.214	0.001	0.000
13	2.381	0.041	0.008	29	2.108	0.005	0.000
14	2.220	0.002	0.000	30	2.344	0.027	0.004
15	2.231	0.003	0.000	31	2.149	0.001	0.000

Avg=2.179

Sum of (y-yi) ^2 = 0.527

Sum of (y-yi) ^3 = -0.011

σ = 0.133

Appendix A

Table A.3.2 : Log Pearson III Calculation

rainfall	rainfall (Desc)	Rank (m)	T=(n+1) /m, n=31	p=1/T	w= $(\ln(1/p^2))^{0.5}$	a	b	z=w-a/b	K_T	Yt= Y+k T*S	$Xt=10^{Yt}$
58.5	285.5	1	32	0.031	2.633	4.7	6.11	1.863	1.796	2.417	261.363
285.5	239.7	2	16	0.063	2.355	4.46	5.44	1.535	1.497	2.378	238.568
84	222.6	3	10.67	0.094	2.176	4.31	5.03	1.318	1.297	2.351	224.463
135	210	4	8	0.125	2.04	4.2	4.72	1.151	1.141	2.330	214.025
178	196.2	5	6.4	0.156	1.927	4.1	4.47	1.010	1.008	2.313	205.525
168	192.1	6	5.34	0.187	1.831	4.02	4.27	0.889	0.893	2.298	198.419
168	191.3	7	4.58	0.218	1.745	3.95	4.08	0.778	0.788	2.284	192.139
175	185.2	8	4	0.250	1.666	3.88	3.92	0.675	0.689	2.271	186.437
210	178.1	9	3.56	0.281	1.594	3.82	3.77	0.580	0.597	2.258	181.292
196.2	178	10	3.2	0.313	1.526	3.76	3.63	0.489	0.509	2.247	176.476
166.2	176.5	11	2.91	0.344	1.462	3.71	3.5	0.403	0.425	2.235	171.979
137.7	175	12	2.67	0.375	1.402	3.66	3.38	0.320	0.344	2.225	167.791
130	168.4	13	2.47	0.405	1.345	3.61	3.27	0.241	0.266	2.214	163.837
178.1	168	14	2.29	0.437	1.288	3.57	3.16	0.160	0.186	2.204	159.903
152	168	15	2.14	0.467	1.234	3.52	3.06	0.083	0.109	2.194	156.193
134.1	166.2	16	2	0.500	1.178	3.48	2.95	0.001	0.028	2.183	152.362
239.7	160.3	17	1.89	0.529	1.129	3.44	2.86	-0.072	-0.045	2.173	149.020
191.3	152	18	1.78	0.562	1.074	3.39	2.76	-0.155	-0.128	2.162	145.278
176.5	151.7	19	1.69	0.592	1.025	3.35	2.67	-0.230	-0.204	2.152	141.951
146.5	150.2	20	1.6	0.625	0.97	3.3	2.57	-0.316	-0.291	2.141	138.221
168.4	146.5	21	1.53	0.654	0.923	3.27	2.48	-0.391	-0.368	2.130	135.035
160.3	138.5	22	1.46	0.685	0.87	3.22	2.39	-0.478	-0.456	2.119	131.440
151.7	138	23	1.4	0.714	0.821	3.18	2.3	-0.560	-0.540	2.108	128.112
136	137.7	24	1.34	0.746	0.766	3.14	2.21	-0.654	-0.638	2.095	124.367
138	136	25	1.28	0.781	0.703	3.09	2.1	-0.765	-0.753	2.079	120.059
138.5	135	26	1.24	0.806	0.656	3.05	2.02	-0.851	-0.842	2.068	116.828
222.6	134.1	27	1.19	0.840	0.59	2.99	1.91	-0.976	-0.973	2.050	112.258
185.2	130	28	1.15	0.870	0.529	2.94	1.81	-1.096	-1.100	2.033	107.992
150.2	104.2	29	1.11	0.901	0.457	2.88	1.69	-1.245	-1.259	2.012	102.888
104.2	84	30	1.07	0.935	0.368	2.81	1.55	-1.443	-1.471	1.984	96.447
192.1	58.5	31	1.04	0.962	0.281	2.74	1.42	-1.653	-1.698	1.954	89.971

Appendix A

Table A.3.3 : Log Pearson III Correlation calculation

Return period(T)	p=1/T	$w=(\ln(1/p^2))^{0.5}$	a	b	z=w-a/b	K_T	Yt=Y+kT*S	$Xt=10^{Yt}$
10	0.1	2.146	4.286	4.959	1.282	1.263	2.347	222.140
20	0.05	2.448	4.543	5.661	1.646	1.598	2.391	246.070
33	0.030	2.645	4.711	6.138	1.877	1.809	2.419	262.372
50	0.02	2.798	4.843	6.519	2.055	1.968	2.440	275.436
100	0.01	3.035	5.047	7.128	2.327	2.208	2.472	296.402
200	0.005	3.256	5.239	7.717	2.577	2.426	2.501	316.788
300	0.003	3.378	5.345	8.050	2.714	2.544	2.516	328.401
500	0.002	3.526	5.475	8.462	2.879	2.685	2.535	342.841

Correlation(r2) =0.9656

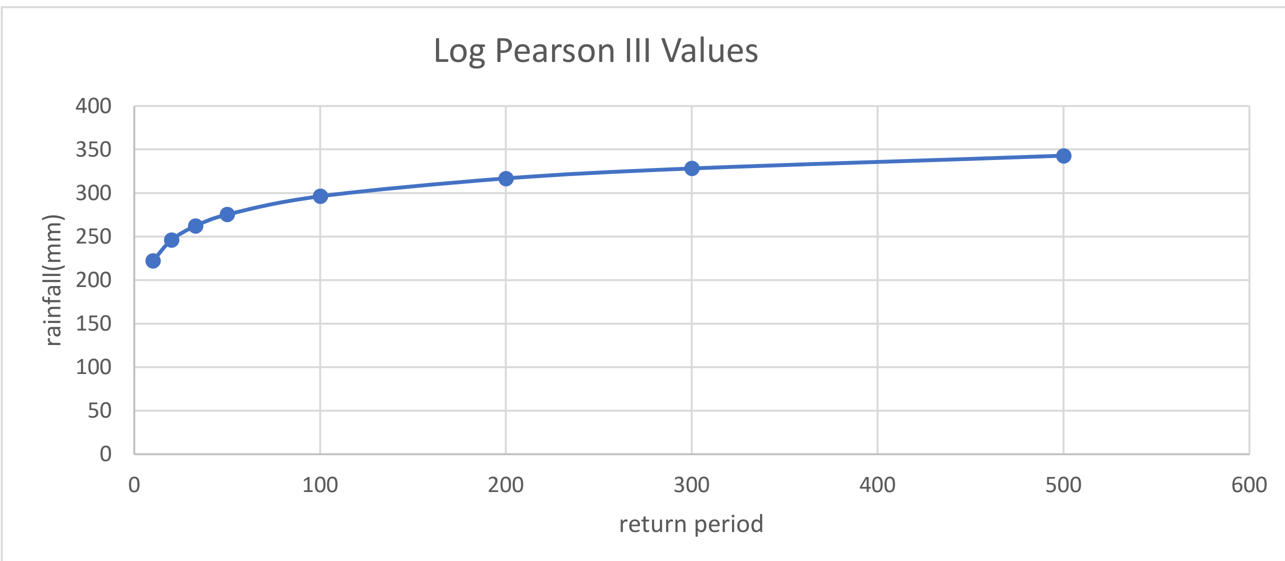

Figure A.3.1 : Log Pearson III rainfall values for different return period

By Gumbel, we found higher value of r^2, so the rainfall for different time period is:

Table A.3.4 : Rainfall from Gumbel method

Return period(T)	Xt	Return period(T)	Xt
10	230.5	100	322.77
20	258.92	200	349.89

33	279.04	300	366.07
50	295.22	500	385.75

A.4 Maximum daily rainfall at Chandra Gadhi station:

Using the calculation step as illustrated in above sample calculation, we found higher value of r^2 by Log Normal, so the rainfall for different time period is:

Table A.4.1: Maximum Daily rainfall at Chandra Gadhi Station

Return period(T)	Xt
10	244.305
20	278.165
33	302.166
50	321.950
100	354.738
200	387.857
300	407.275
500	431.986

A.5 Maximum daily rainfall at Sanischare station:

Similarly, at this station, by Gumbel, we found higher value of r^2, so the rainfall for different time period is:

Table A.5.1 :Maximum daily rainfall at Sanischare station:

Return period(T)	Xt
10	259.56
20	302.47
33	332.84
50	357.26
100	398.85
200	439.77
300	464.2
500	493.9

Appendix B:

Meteorological Data for Potential Evapotranspiration

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B. Eto CALCULATION PARAMETERS

B.1 Temperature and Wind

Table 1:Sample calculation for temperature and wind for the month of January

Dailymean tmax, tmin and wind			
Day	tmax	tmin	Wind (m/s)
1	24	9	0.75
2	24	9.5	0.625
3	24	8.5	1.025
4	24	8.5	1.425
5	24	8.5	1.375
6	24	8.5	1.575
7	23	8	1.175
8	23	8.5	1.1
9	22.5	8	0.725
10	22.5	8	1
11	23	8.5	0.9
12	22.5	8	0.725
13	23	8.5	0.85
14	22.5	8.5	0.8
15	23	8	1.175
16	22.5	7.5	1.05
17	22.5	7.5	0.975
18	23	7.6	0.75
19	25	8	0.9
20	24	8.5	0.9
21	25	8	0.85

22	21	8.5	0.975
23	25	8.2	1.125
24	25.5	8.6	1
25	26	8.5	1.15
26	26	9	0.75
27	26.8	9.4	0.875
28	26.5	9	0.875
29	26.5	8.6	1.1
30	26.5	8.5	0.675
31	26.5	9	1.025
Mean	24.10645	8.416129	0.974193548

Similarly the temperature and wind velocity was done for all months.

B.2 Humidity

Humidity data were obtained from DHM.

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Table 2: Mean monthly relative humidity calculation

year	jan	jan	feb	feb	mar	mar	apr	apr	may	may	jun	jun	july	july	aug	aug	sept	sept	oct	oct	nov	nov	dec	dec
	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000	1.000	2.000
1984.000				74.264	56.833	50.991	48.607	60.667	72.233	78.253	78.010	82.340	85.677	83.313	78.663	82.250	86.207	84.287	76.793	82.800	70.397	73.203	76.307	83.825
1985.000	81.917	74.994	70.464	69.136	60.317	59.063	50.113	58.970	67.750	72.638	78.510	80.793	86.023	84.834	80.560	86.169	83.923	83.483	81.337	79.717	77.491	79.773	81.263	82.522
1986.000	83.273	79.528	72.211	65.814	53.557	38.616	54.703	59.637	70.787	62.653		79.270	79.650	83.700	74.157	85.031	86.993	78.463	85.047	73.500	74.956	78.323	75.613	78.059
1987.000	73.323	77.072	66.068	60.857	62.927	62.306	53.913	60.993	66.033	61.900	74.497	84.193	84.833	83.609	88.307	80.656	84.410	84.363	79.310	80.093	77.378	76.680	80.140	75.347
1988.000	82.373	79.806	77.257	69.061	76.240	73.869	72.047	86.417	86.143	89.019	87.587	90.763	94.387	92.053	92.107	95.116	94.367	92.153	91.597	89.510	83.478	82.563	85.423	88.953
1989.000	88.533	89.038	82.564	76.264	76.483	70.813	63.330	63.360	68.840	85.291	87.723	88.717	92.563	84.588	81.077	84.194	86.650	87.937	80.403	75.863	78.619	79.080	75.477	83.769
1990.000	88.193	89.638	76.339	80.775	87.123	91.231	93.640	79.110	75.460	79.497	82.077	83.243	87.123	85.516	86.950	81.531	84.047	85.200	82.573	71.540	67.169	68.717	74.577	75.741
1991.000																								
1992	82.	78.	73.	60.	55.	46.	44.	52.	67.	71.	71.	76.	83.	79.	78.	78.	79.	80.	82.	75.	65.	64.	65.	70.

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.000	210	666	146	650	813	538	403	913	417	475	737	567	477	444	960	525	350	770	120	567	647	533	813	338
1993 .000	86. 817	77. 356	67. 143	64. 946	56. 170	49. 478	45. 437	61. 990	68. 783	68. 556	74. 813	78. 510	77. 980	83. 322	82. 737	81. 313	79. 153	79. 217	79. 197	70. 147	74. 675	65. 777	67. 080	62. 700
1994 .000	70. 537	72. 603	68. 371	57. 600	55. 470	58. 906	50. 940	52. 680	59. 083	68. 297	74. 830	78. 157	76. 920	76. 122	80. 863	78. 119	79. 803	78. 523	78. 127	66. 373		63. 777	69. 380	63. 684
1995 .000	70. 123	71. 403	70. 529	60. 279	57. 493	48. 531	38. 177	49. 810	58. 373	72. 628	83. 283	81. 710	86. 947	81. 406	80. 213	76. 891	79. 573	82. 447	73. 907	76. 870	84. 194	73. 530	79. 943	84. 047
1996 .000	85. 357	89. 634	76. 389	68. 807	64. 263	49. 219	36. 767	52. 767	71. 980	70. 181	72. 897	81. 863	88. 450	82. 178	84. 767	79. 875	80. 153	76. 823	74. 820	72. 253	75. 413	73. 753	72. 550	70. 653
1997 .000	74. 790	80. 053	73. 889	55. 346	53. 357	49. 216	60. 353	60. 873	62. 327	58. 797	74. 390	77. 413	82. 567	75. 684	80. 513	76. 222	81. 787	79. 173	71. 900	68. 587	67. 128	73. 357	80. 687	82. 575
1998 .000	87. 607	77. 559	65. 571	61. 457	58. 727	63. 919	59. 860	67. 637	72. 273	68. 294	77. 463	83. 657	84. 457	88. 169	86. 357	86. 703	80. 420	83. 577	76. 997	81. 503	73. 434	77. 130	77. 463	75. 178
1999 .000	76. 230	78. 009	68. 721	62. 221	45. 603	49. 947	58. 500	61. 783	72. 330	79. 456	75. 327	83. 810	87. 307	85. 709	85. 810	91. 334	90. 527	90. 593	88. 530	80. 923	82. 056	78. 010	73. 997	76. 078
2000 .000	86. 040	84. 644	80. 264	79. 600	83. 287	70. 175	70. 633	75. 263	79. 160	88. 159	83. 597	81. 963	83. 563	84. 534	87. 933	82. 875	85. 327	86. 117	79. 917	73. 877	79. 425	87. 477	77. 617	75. 119
2001 .000	81. 410	74. 469	74. 900	62. 404	47. 503	44. 097	52. 557	60. 990	71. 293	76. 422	75. 590	77. 667	76. 997	81. 706	79. 867	84. 053	85. 947	83. 400	87. 530	72. 720	79. 609	83. 533	87. 080	79. 319
2002 .000	74. 970	85. 778	82. 082	67. 500	60. 417	61. 709	68. 423	68. 423	65. 727	73. 169	76. 650	79. 020	84. 247	85. 816	81. 323	83. 797	77. 183	84. 770	74. 393	77. 953	71. 578	76. 993	73. 650	81. 313
2003 .000	83. 603	88. 991	76. 193	70. 914	62. 443	60. 594	62. 553	67. 650	57. 867	69. 128	77. 333	79. 353	84. 227	79. 059	78. 433	81. 403	80. 187	85. 537	82. 377	81. 180	81. 709	83. 880	79. 187	82. 341

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2004.000	88.513	81.950	68.950	68.046	64.997	64.916	69.063	71.780	83.223	88.091	94.823	91.757	92.787	93.541	96.927	94.978	88.027	79.500	81.313	67.887	65.141	65.697	60.520	80.013
2005.000	79.060	82.466	77.789	68.639	64.293	66.581	55.007	63.113	68.660	67.563	65.450	78.627	81.430	79.553	81.680	84.188	74.863	76.123	79.640	78.433	65.444	79.813	65.500	64.053
2006.000	75.740	71.244	66.014	74.814	62.207	44.900	57.553	62.400	66.900	72.028	78.103	80.077	81.443	80.478	78.700	80.016	84.057	80.647	76.507	72.300	74.878	76.147	80.377	74.238
2007.000	78.007	73.816	76.825	70.261	63.857	59.269	62.040	72.250	64.500	70.000	81.053	79.153	76.517	88.216	82.953	82.175	87.340	76.640	79.273	81.090	74.322	76.010	78.697	85.909
2008.000	82.313	82.384	79.271	70.107	68.133	67.222	68.747	68.380	67.253	75.294	87.577	80.570	82.627	83.566	79.947	87.003	80.577	80.210	80.750	72.523	75.038	78.313	78.800	
2009.000	82.293	84.972	78.864	73.229	54.723	53.434	56.563	61.530	65.163	70.019	72.107	80.290	78.167	81.084	82.690	81.594	75.037	78.383	77.227	75.163	73.688	78.330	85.187	81.309
RH mean	80.968	80.253	73.742	67.720	62.089	58.222	58.157	64.055	69.182	73.472	78.559	81.579	84.015	83.488	82.900	83.440	83.036	82.333	80.063	75.935	74.703	75.776	76.093	77.378
Rh mean monthly	80.611		70.731		60.155		61.106		71.327		80.069		83.751		83.170		82.685		77.999		75.239		76.736	

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B.3 Radiation and sunshine hours

Latitude = 26.56 °

Ra and N are interpolated for our latitude value

North latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0°	14.5	15.0	15.2	14.7	13.9	13.4	13.5	14.2	14.9	15.0	14.6	14.3
10°	12.8	13.9	14.8	15.2	15.0	14.8	14.8	15.0	14.9	14.1	13.1	12.4
20°	10.8	12.3	13.9	15.2	15.7	15.8	15.7	15.3	14.4	12.9	11.2	10.3
30°	8.5	10.5	12.7	14.8	16.0	16.5	16.2	15.3	13.5	11.3	9.1	7.9
40°	6.0	8.3	11.0	13.9	15.9	16.7	16.3	14.8	12.2	9.3	6.7	5.4
50°	3.6	5.9	9.1	12.7	15.4	16.7	16.1	13.9	10.5	7.1	4.3	3.0

Radiation (Ra) is calculated for latitude 26.56° using above table

Table 3: Interpolation for latitude value of 26.56°

Lat\Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
20	10.8	12.3	13.9	15.2	15.7	15.8	15.7	15.3	14.4	12.9	11.2	10.3
30	8.5	10.5	12.7	14.8	16	16.5	16.2	15.3	13.5	11.3	9.1	7.9
26.56	9.30	11.13	13.12	14.95	15.89	16.25	16.02	15.3	13.815	11.86	9.83	8.74

Mean monthly values of possible sun shine hours,N

North latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0°	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1
10°	11.6	11.8	12.1	12.4	12.6	12.7	12.6	12.4	12.9	11.9	11.7	11.5
20°	11.1	11.5	12.0	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9
30°	10.4	11.1	12.0	12.9	13.7	14.1	13.9	13.2	12.4	11.5	10.6	10.2
40°	9.6	10.7	11.9	13.2	14.4	15.0	14.7	13.8	12.5	11.2	10.0	9.4
50°	8.6	10.1	11.8	13.8	15.4	16.4	16.0	14.5	12.7	10.8	9.1	8.1

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Table 4: Interpolation for latitude of 26.56^o

Sunshine hours (N)

Lat\Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
20	11.1	11.5	12	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9
30	10.4	11.1	12	12.9	13.7	14.1	13.9	13.2	12.4	11.5	10.6	10.2
26.56	10.645	11.24	12	12.795	13.49	13.82	13.655	13.06	12.365	11.57	10.81	10.445

Table 5: Monthly values required to calculate Eto in tabulated form

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
tmax	24.11	25.43	31.23	35.05	32.34	32.72	30.80	32.85	31.82	30.82	28.04	24.85
tmin	8.42	11.96	16.72	18.67	18.16	18.82	18.60	19.76	18.33	15.40	9.44	6.72
Rhmean	80.61	70.73	60.16	61.11	71.33	80.07	83.75	83.17	82.68	78.00	75.24	76.74
Wind (U)	0.97	1.27	1.78	2.11	1.88	1.50	1.08	1.00	0.95	0.73	0.52	0.53
Ra	9.31	11.13	13.12	14.94	15.90	16.26	16.03	15.30	13.82	11.86	9.84	8.74
N	10.65	11.24	12.00	12.80	13.49	13.82	13.66	13.06	12.37	11.57	10.81	10.45

B.4 Eto Calculation

Table 6: Data we have

Altitude = 98.45 m

Latitude = 26.5 ^{o}

Table 7: Basic Climate data

Mean daily Temp=(Tmax+Tmin)/2	15.785^{o}C	
Tmax	24.85 ^{o}C	

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Tmin	6.72	
Mean relative humedity=(Rhmax+Rhmin)/2	76.74	
Mean actual vapour prerssure(ed)=	13.71	(from calculation)
Actual daily sunshine hour(n)=	10.45hr	hr
Measured 24 hr windrun(U)=	0.53m/s	m/s

Tabulated Values	
Maxmimum Sunshine hour(N):	10.45 hr
Extra Terrestial radiation(Ra):	8.74 hr

Table 8: Eto Calculation

1	Wind(already in 2m),U24=	45.792 km/day
2	Mean atm pressure (PMB),PMB=1013-0.1155*latitude	1001.63 bar
3	Mean absolute temP(Tkmean)=	288.785 kelvin
4	Saturation vapour pressure(ea)=	17.86
5	mean actual vapour pressure(ed)=	13.7084
6	f(u)=	0.3936
7	G	0.66
8	D	1.48
9	w	0.69

10	Calculation of net radiation Rn	
	R_s=	6.555
	R_{ns}=	4.916
	f(T)=	13.91
	f(ed)=	0.177
	f(n/N)=	1
	R_{n1}=	2.463
	R_n=	2.453

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11	selection of c From table with	
	RH max=	76.74
	Rs=	6.555
	U24=	45.792
	C=	1.005

Finally,

ET0= c*((w*Rn)+(1-w)*f(u)*(ea-ed))= 2.211564649 mm/day

selection of c From table		
For		
RH max=	76.74	
Rs=	6.555	
U24=	45.792	
C=	1.005	selected from table 7.4

we get:		
ET0=	2.21	mm/day

Table 9: Eto values monthly and half monthly computed as in sample calculation

Month	Eto	Half monthly values	
JAN	2.57	2.48	2.89
FEB	3.86	3.54	4.34
MAR	5.79	5.31	6.12
APR	7.11	6.78	7.12
MAY	7.16	7.15	7.17
JUN	7.2	7.19	7.1

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JUL	6.78	6.89	6.75
AUG	6.66	6.69	6.42
SEP	5.7	5.94	5.41
OCT	4.52	4.82	4.13
NOV	2.95	3.34	2.77
DEC	2.21	2.4	2.3

Appendix C:

Crop water and Irrigation Requirements

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A.1 Rainfall data of gaida kankai

For our command area gaida kankai hydro-meteorological station was the best to use. So we proceed with the rainfall available at that station.

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Table 1: Rainfall data of gaida kankai

Rainfall for Gaida

	Year	Jan	Feb	Mar	Apr	May	Jun	JUL	AUG	SEP	OCT	NOV	DEC
1	1984	0	0	17.2	32	68.4	118	126.3	35	437	98	0	15.2
2	1985	0	18.5	18	8.5	69	128	142	135	55	207	8	60
3	1986	0	0.8	0	19.5	46	95	80	150	107.8	51	4.6	3.8
4	1987	0	7.5	26.6	84	32.5	124.4	77.2	243	134	111	6.2	0.2
5	1988	2.6	27.6	26.8	49.6	43.2	40	147	111.2	58.4	81.2	31.2	3.4
6	1989	17.4	14.8	2.6	4	142.6	121.2	166.4	151.2	191	38	18	2.2
7	1990	6.2	20.2	9.5	38	82	134.5	215	148.6	126	14.8	0	0
8	1991	8.6	1.5	19.5	12.5	32	94.5	33	104.4	198	82	0	4.2
9	1992	1.2	2.8	0	8.5	42.6	36	151.5	61.4	71.2	26	0	30.6
10	1993	13.2	7.8	24.8	39.2	42.2	94.6	141.2	150	80.4	113.6	21.6	0
11	1994	24.6	31.6	13.4	19.3	50.6	87.6	106.8	188.4	82.6	7.2	7.2	0
12	1995	2.8	7.6	9.2	2.8	80.6	131.6	97.6	97.6	87.2	57.2	38.2	12.5
13	1996	32.6	6.2	2.6	17.6	65.2	77.6	238.6	116.4	67.6	76.6	0	0
14	1997	10.2	4.2	14.6	33.6	30.6	69.6	113.4	69.6	176.6	6.2	0	23.8
15	1998	0	2.2	34.6	36.5	63.6	145.6	95	258.8	123.6	58.6	7.2	0
16	1999	0	0	8.5	21.6	43.7	73	151.6	226.6	111.2	74.4	4.2	0
17	2000	0	10.6	0	40	58.5	118.5	191.3	137.7	44	80	14.4	0
18	2001	0.6	0.5	12.5	65	42.4	93.6	194	80.3	74.4	95.2	26.6	0
19	2002	34	0	7	26.2	74.4	153.5	145	54.4	49.6	108.2	0	0
20	2003	0.6	13.4	9.6	21.2	34.2	72	186.2	102.4	61.6	73.6	33.6	12
21	2004	10.6	0	10.2	32.7	63.8	64.7	145	59.2	92	91.2	0	0
22	2005	14.2	4.2	18.8	38.2	33.5	60.4	105.2	164.5	41.6	58.6	0	0
23	2006	0	2.1	3.2	26.6	38.4	45.4	88.2	55.6	74.6	40.4	2.2	2.8
24	2007	0	73.5	5.2	21.2	69.2	74.4	128.6	67.6	85.6	17.5	27.2	0
25	2008	33.6	4.4	18.4	26.4	33.6	105.6	101.6	149.6	65.4	62.6	0	0
26	2009	0	0	10.2	32.4	77.6	141.6	71.5	252.8	29.6	174.6	0	8
27	2010	0	0	0	5.6	56.6	176.6	213.4	114.2	56.6	43.4	20.5	0
28	2011	6	4.6	7.4	53.6	166.4	129.6	113.8	136.6	67.2	18.6	7.4	1.4
29	2012	4.6	4.4	2.2	38.2	30.6	84.4	77.6	88.6	98	19.8	0	0
30	2013	15.6	14.6	4.5	16.2	72.6	60.4	186.6	87.4	116.4	56.4	0	0
	total	239.2	285.6	337.1	870.7	1786.6	2951.9	4030.6	3798.1	3064.2	2042.9	278.3	180.1
	mean	7.97	9.52	11.24	29.02	59.55	98.40	134.35	126.60	102.14	68.10	9.28	6.00
	SD	10.78	14.68	9.13	18.18	30.84	35.83	49.23	60.72	76.06	46.05	11.96	12.70
80% reliablity		-1.10	-2.83	3.55	13.72	33.60	68.24	92.92	75.50	38.13	29.34	-0.79	-4.68
corrected 80% reliablity		0.00	0.00	3.55	13.72	33.60	68.24	92.92	75.50	38.13	29.34	0.00	0.00

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Table 2 : Irrigation Water requirement calculation table:

3 crops : vegetable for 300 ha ,rice for 750 ha and Pulses for 150 ha command area																								
Cropping pattern	Vegetable				Pulses							Land Prep		Monsoon Rice							Land Prep	Winter vegetable		
Month	jan	jan	feb	feb	mar	mar	apr	apr	may	may	jun	jun	july	july	aug	aug	sept	sept	oct	oct	nov	nov	dec	dec
Period	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Days	15.5	15.5	14	14	15.5	15.5	15	15	15.5	15.5	15	15	15.5	15.5	15.5	15.5	15	15	15.5	15.5	15	15	15.5	15.5
ET0(mm/day)	2.6	3.0	3.7	4.5	5.6	6.5	7.3	7.7	7.8	7.8	7.8	7.7	7.5	7.3	7.1	6.8	6.4	5.9	5.2	4.5	3.8	3.1	2.7	2.5
ET0(mm)	39.8	45.9	51.1	63.3	86.3	100.6	109.2	115.8	121.2	121.5	116.7	114.9	116.3	113.5	110.7	105.9	95.9	87.9	81.1	70.1	56.3	47.1	41.9	38.8
80% rainfall (for year)	0.0		0.0		3.6		13.7		33.6		68.2		92.9		75.5		38.1		29.3		0.0		0.0	
80% rain(mm)	0.0	0.0	0.0	0.4	1.3	3.0	5.6	9.3	14.3	21.1	29.8	37.2	43.4	44.3	39.9	33.1	23.7	18.0	15.8	11.0	3.7	0.0	0.0	0.0
Crop Coefficient (kc)	0.86	0.95	0.95	0.89	0.40	0.50	0.75	0.95	1.05	1.05	0.96	0.00	0.00	1.10	1.10	1.10	1.05	1.05	0.95	0.95		0.28	0.34	0.54
Et crop(mm)	34.26	43.59	48.55	56.32	34.53	50.30	81.90	110.01	127.27	127.60	112.03	0.00	0.00	124.81	121.74	116.45	100.64	92.30	77.01	66.56	0.00	13.19	14.23	20.93
Land Prep (mm)												55	55	50	50									
percolation(mm)													46.5	46.5	46.5	46.5	45	45	45	30				
Field Requirement(mm)	34.26	43.59	48.55	56.32	34.53	50.30	81.90	110.01	127.27	127.60	112.03	55.00	101.50	221.31	218.24	162.95	145.64	137.30	122.01	96.56	0.00	13.19	14.23	20.93
efficitive Rainfall(mm)	0.00	0.00	0.00	0.00	0.00	0.00	4.75	7.94	12.17	17.96	25.32	31.63	36.87	37.64	33.94	28.12	20.18	15.27	13.40	9.35	0.00	0.00	0.00	0.00
Net Irrigation Req (I-req)(mm)	34.26	43.59	48.55	56.32	34.53	50.30	77.15	102.07	115.10	109.64	86.71	23.37	64.63	183.67	184.30	134.83	125.47	122.02	108.61	87.21		13.19	14.23	20.93
E-field	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9		0.75	0.75	0.75
E-farm	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75		0.75	0.75	0.75
E-main	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8		0.8	0.8	0.8
I-gross(mm)	76.13	96.86	107.88	125.15	76.74	111.77	171.44	226.81	255.79	243.64	160.57	43.29	119.69	340.12	341.29	249.69	232.35	225.97	201.13	161.49		29.31	31.62	46.50
I-gross(l/s/ha)	0.57	0.72	0.89	1.03	0.57	0.83	1.32	1.75	1.91	1.82	1.24	0.33	0.89	2.54	2.55	1.86	1.79	1.74	1.50	1.21		0.23	0.24	0.35
I-gross(cubic meter/sec)	0.17	0.22	0.27	0.31	0.09	0.13	0.20	0.26	0.29	0.27	0.19	0.25	0.67	1.90	1.91	1.40	1.34	1.31	1.13	0.90	0.00	0.07	0.07	0.10
for water right	0.04	0.04	0.03	0.03	0.03	0.02	0.02	0.03	0.04	0.07	0.10	0.21	0.39	0.53	0.62	0.64	0.57	0.43	0.22	0.11	0.08	0.06	0.05	0.05
flow	0.43	0.38	0.34	0.37	0.27	0.24	0.22	0.30	0.43	0.68	1.03	2.11	3.92	5.29	6.23	6.36	5.69	4.32	2.24	1.07	0.80	0.63	0.55	0.48
surplus	0.22	0.13	0.04	0.00	0.16	0.09	0.00	0.00	0.10	0.34	0.74	1.65	2.85	2.86	3.69	4.33	3.78	2.58	0.89	0.06	0.72	0.50	0.42	0.33

Thus the canal discharge was fixed from the optimum required IWR and i.e **1.91 m^3/s.**

Appendix D:

Flood Calculation

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D. ESTIMATION OF DESIGN FLOOD

D.1 DHM 2004 METHOD

Table 1: Input Parameters for DHM 2004

River name	Bhuteni	
River location	Garamai	
Basin Area	25.8	Km^2
Basin area below 5000 m elevation level	25.8	Km^2
Basin area below 3000 m elevation level	25.8	Km^2
Average Basin Elevation	140	masl
Annual Wetness Index	1600	mm

Table 2: Instantaneous flood flow estimation BY DHM 2004

Return Period(years)	Daily flood discharge	Instantaneous flood discharge(m^3/s)
2	19	37
5	30	71
10	38	98
20	46	129
50	57	175
100	67	215
200	76	259
500	90	325
1000	101	381
5000	129	535
10000	143	612

D.2 MHSP Method

Table 3: Input Parameters for MHSP

Total drainage area	(A)	25.80	Km^2
Mean precipitation	(MMP)	280	mm
Monsoon wetness index	(MWI)	1600	

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Region	Eastern	

Table 4: Flood flows by MHSP

Return period	flow m^3/s			Remarks
	Western	Central	Eastern	
5	33.75	38.72	95.30	Flood flows are valid for the specified region only(Eastern in our case)
20	52.29	65.97	151.50	
50	66.23	87.92	193.83	
100	78.86	107.60	229.41	
1000	130.42	196.02	384.39	
10000	198.75	333.94	614.91	

D.3 WECS/DHM 1990

Table 5: Input parameters for WECS/DHM 1990

River name	Bhuteni	
River location	Garamani	
Basin Area	25.8	km
Basin area below 5000 m elevation level	25.8	
Basin area below 3000 m elevation level	25.8	
Average Basin Elevation	140	masl
Annual Wetness Index	1600	mm

Table 6: Flood flow statistics by WECS/DHM 1990

Return Periods(years)	Flood Discharge(m3/s)	
	Daily	Instantaneous
2	19	34
5	30	60
10	38	81
20	46	103
50	57	136
100	67	164

200	76	194
500	90	238
1000	101	275
5000	129	373
10000	143	421

D.4 Rational Method

Table 7: Input Parameters for Rational method

Basin Area	25.8	
L	26.04	km
h	127	m
Time of concentration(tc)	6.336009	hr
Slope	0.004877	

Table 8: IDF and Flood Estimation for Bhuteni Khola at intake

Return Period	10	20	33	50	100	200	300	500
Max. Daily	243	277	302	321	355	387	407	431
Time	RI_{10}	RI_{20}	RI_{33}	RI_{50}	RI_{100}	RI_{200}	RI_{300}	RI_{500}
0.25	211.96	242.06	263.81	280.41	310.11	338.07	355.54	376.50
0.5	133.53	152.49	166.19	176.65	195.36	212.97	223.97	237.18
0.75	101.90	116.37	126.83	134.81	149.09	162.53	170.93	181.00
1	84.12	96.06	104.70	111.28	123.07	134.16	141.10	149.42
2	52.99	60.52	65.95	70.10	77.53	84.52	88.89	94.13
4	33.38	38.12	41.55	44.16	48.84	53.24	55.99	59.30
6	25.48	29.09	31.71	33.70	37.27	40.63	42.73	45.25
8	21.03	24.02	26.17	27.82	30.77	33.54	35.27	37.35
12	16.05	18.33	19.97	21.23	23.48	25.60	26.92	28.51
16	13.25	15.13	16.49	17.53	19.38	21.13	22.22	23.53
20	11.42	13.04	14.21	15.10	16.70	18.21	19.15	20.28
24	10.11	11.55	12.58	13.38	14.79	16.13	16.96	17.96
6.34	24.56	28.04	30.56	32.49	35.93	39.17	41.19	43.62
Q, m3/s	89	102	111	118	131	143	150	159

D.5 PCJ 1996 Method

Table 9: Input parameters

Appendix D

Name of the River	Bhutani Khola
Location	Birtamod
Total Basin Area, km^2	25.8
Basin Area below 5000 m, km^2	25.8
Basin Area below 3000 m, km^2	25.8

Table 10: Average rainfall computation and flood computation

Rainfall Stations	Return Period in Years				
	10	33	50	100	300
	Hourly Intensity in mm/min				
1409	1.46	1.77	1.87	2.04	2.32
1412	1.55	1.91	2.04	2.25	2.58
1415	1.64	2.11	2.26	2.53	2.94
Average	1.54	1.91	2.03	2.25	2.58
K_t	0.99	0.99	0.99	0.99	0.99
a_p	1.52	1.89	2.01	2.22	2.55
O_p	0.5	0.75	0.85	0.95	1
Φ	0.283	0.283	0.283	0.283	0.283
K_f	1.000	1.000	1.000	1.000	1.000
F	25.8	25.8	25.8	25.8	25.8
Q, cumecs	93	173	208	257	311

Appendix E:

Design of Head Works

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Appendix E

E.1 DESIGN OF WEIR WITH GATED SYSTEM

E.1.1 Data

Design flood discharge in the river	131	m^3/s	
River bed level	98.217	m	
Hfl before construction of weir	100.517	m	river bed level+2.3m
FSL of canal	99.117	m	
Permissible afflux	1	m	
Bed retrogression	0.5	m	
Discharge concentration factor	20%		
Lacey's silt factor	1.5		
Pier contraction coefficient	0.1		
Full supply discharge of canal	2.292	m^3/s	
Safe exit gradient	0.17		

crest level of weir bay section	98.917	m
let the crest width	1	m

E.1.2 Determination of discharge intensity (q) and head loss(Hl) for different conditions

a)	High flood conditions		
i)	Without flow concentration and retrogression		
	As B<(2*H/3),the weir acts as a sharp-crested weir.He=	2.73	m
	q=1.84(He)^1.5	8.300	Cumecs/m
	u/s TEL	101.647	m
	d/s TEL	100.643	m
	head loss,HL=	1.004	m
ii)	With flow concentration and retrogression		
	q=	9.960	Cumecs/m
	head over the crest	3.100	m
	u/s TEL	102.017	m
	d/s TEL after retrogression	100.143	m
	Head loss HL	1.874	m
b)	Pond level conditions		
i)	Without flow concentration and retrogression		
	u/s TEL	99.71	m
	head over the crest	0.793	m

	q=	1.299	Cumecs/m
	d/s TEL	99.6	m
	head loss HL	0.11	m
ii)	With flow concentration and retrogression		
	q=	1.559	Cumecs/m
	head over the crest	0.895	m
	u/s TEL	99.812	m
	d/s TEL	99.155	m
	head loss HL	0.657	m

E.1.3 Depth of sheet pile calculation

Total discharge over the weir -bay section	107.13	cumecs/m
Overall wateway	17	m
Average discharge intensity ,q=	6.302	cumecs/m
Normal scour depth ,R=	4.02	m
Bottom level of d/s pile=d/s water level after retrogression -2*R	91.977	m
Bottom level of u/s pile=u/s water level -1.5*R	95.487	m
Depth of d/s pile below floor =	3.683	m
Depth of u/s pile below river bed=	2.73	m

E.1.4 Hydraulic jump calculation

SN	Item	high flood condition		pond level condition	
		No concentration and retrogression	with concentration and retrogression	No concentration and retrogression	with conc. and retrogression
1	discharge intensity ,q	8.300	9.960	1.299	0.895
2	Head loss,HL	1.004	1.874	0.11	0.657
3	D/S specific energy(Ef2) from blench curve	3.6	4.4	0.9	0.8
4	Level at which jump will form=d/s	97.04	95.74	98.7	98.355

	TEL after retrogression-Ef2				
5	u/s specific energy Ef1=Ef2 +HL	4.604	6.274	1.01	1.457
6	Pre jmp depth D1 corresponding to Ef1 from specific energy curve or calculated as Ef1=D1+((q/D1)^2)/(2*g),D1=				
	Ef1=D1+((q/D1)^2)/(2*g),D1=	0.98	0.98	0.37	0.18
7	Post jump depth D2 Corresponding to Ef2 from formula				
	Ef2=D2 +((q/D2)^2)/(2*g),D2=	3.72	4.1	0.75	0.72
8	Height of jump (D2-D1)	2.74	3.12	0.38	0.54
9	Length of concrete floor required beyond the jump=5*(D2-D1)	13.7	15.6	1.9	2.7
10	Initial froud number =	2.731	3.278	1.843	3.742

Lowest level of jump formation	95.74	m
Largest length of impervious floor	15.6	m
let us keep the level and length of d/s floor as 95.66 m and 16m.		

Maximum seepage head =pond level-d/s floor level		
H=	3.957	m
exit gradient GE=H/(d*∏*lamda^0.5)		
where GE=1/6		
lamda=	4.21	
alpha=	7.35	
b=alpha*d	27.070	
Provide the floor lengths as follows		
u/s glacis (2:1)	1.4	
crest width	1	m
d/s glacis(3:1)	9.771	
d/s horizontal floor	16	m
u/s horizontal floor	2.829	m
total length=	**31**	m

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E.1.5 Uplift pressure calculation

SN	flow condition	u/s water level(m)	d/s water level(m)	seepage head(m)	height of subsoil HGL above water level and its elevation(m)					
					u/s pile			d/s pile		
					ϕE1=100	ϕD1=81.67%	ϕC1=76.37%	ϕE2=30.03%	ϕD2=21.06%	ϕC2=0
1	no flow	99.617	95.66	3.957	3.957	3.23	3.02	1.188	0.833	0
				RL	99.617	98.89	98.68	96.848	96.493	95.96
2	High flood with flow concentration and retrogression	101.517	100.017	1.5	1.5	1.225	1.145	0.45	0.316	0
				RL	101.517	101.242	101.162	100.467	100.333	100.017
3	Flow at pond level flow condition and retrogression	99.617	99.117	0.5	0.5	0.408	0.382	0.15	0.105	0
				RL	99.617	99.525	99.499	99.267	99.222	99.117

E.1.6 Pre jump profile

Distance from d/s end of crest	level of glacis(m)	High flood condition q=9.96 u/s TEL=102.02 Ef1=u/s TEL -glacis level	D1(m)	Pond level condition q=0.89 u/s TEL=99.81 Ef1=u/s TEL -glacis level	D1(m)
0	98.917	3.103	-1.097	0.893	0.25
1.701	98.35	3.67	1.54	1.46	0.18
2	98.247	3.773	1.49		
4	97.58	4.44	1.26		
6	96.917	5.103	1.127		
8	96.25	5.77	1.03		
9.531	95.74	6.28	0.98		

E.1.7 Post jump profile

For high flood condition,with concentration and retrogression
Fr^2= 10.734 D1=0.98
For pond level condition ,with concentration and retrogression
Fr^2= 14.001 D1=0.18

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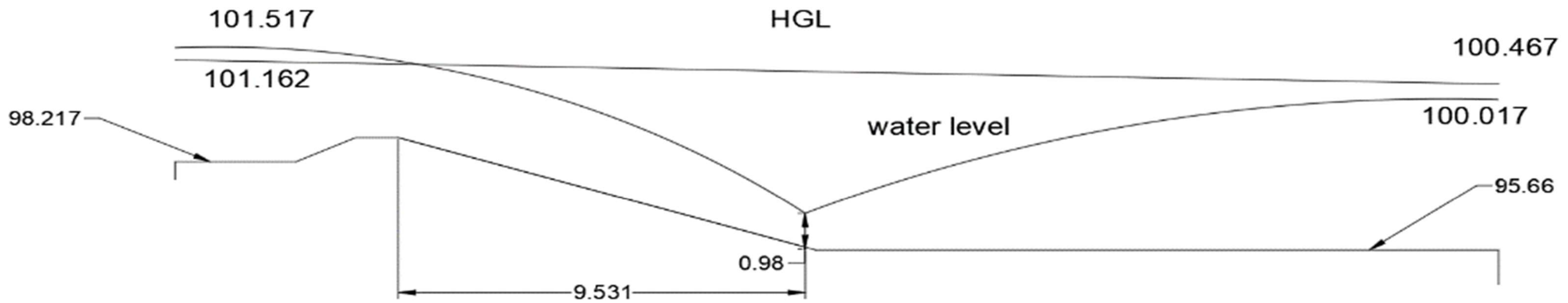
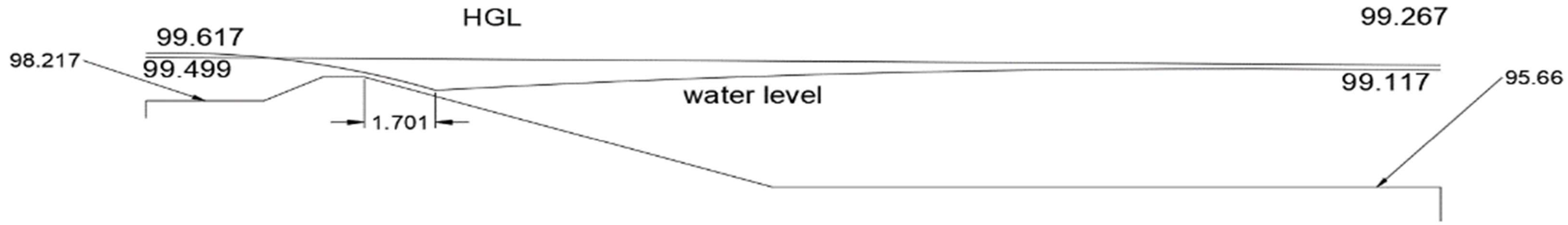
	High flood condition,y1=0.98				pond level condition ,y1=0.18			
SN	x/y1	y/y1	x(m)	y(m)	x/y1	y/y1	x(m)	y(m)
	0.245	1	0.2401	0.98	1	1.2	0.18	0.216
1	1	1.2	0.98	1.176	2.5	2	0.45	0.36
2	2.5	2	2.45	1.96	5	2.7	0.9	0.486
3	5	2.7	4.9	2.646	10	3.8	1.8	0.684
4	10	3.3	9.8	3.234	15	4.4	2.7	0.792
5	15	3.5	14.7	3.43				
6	16.57	3.8	16.2386	3.724				



High Flood condition



Pond level condition

E.1.8 Uplift Pressure and thickness of floor calculation

SN	distance from end point of crest(m)	No flow condition			High flood condition				pond level condition					
		level of glacis(m)	HGL(m)	Unbalance head(m)	water level(m)	HGL(m)	unbalanced head(m)	(2*hd)/3	water level(m)	HGL(m)	unbalanced head(m)	(2*hd)/3	controlling uplift head(m)	thickness(m)
1	0	98.917	98.371	-0.546		101.045			99.167	99.460	0.293	0.195		
2	1.701	98.35	98.270	-0.080	99.89	101.007	1.117	0.744	98.53	99.447	0.917	0.611	0.744	0.600
3	2	98.247	98.253	0.006	99.737	101.000	1.263	0.842					0.842	0.679
4	4	97.58	98.135	0.555	98.84	100.955	2.115	1.410					1.410	1.137
5	6	96.917	98.016	1.099	98.044	100.910	2.866	1.911					1.911	1.541
6	8	96.25	97.898	1.648	97.28	100.865	3.585	2.390					2.390	1.928
7	9.531	95.74	97.808	2.068	96.72	100.831	4.111	2.741					2.741	2.210
8	9.771	95.66	97.794	2.134	96.64	100.826	4.186	2.790					2.790	2.207
9	10.511	95.66	97.750	2.090	96.836	100.809	3.973	2.649					2.649	2.093
10	11.981	95.66	97.663	2.003	97.62	100.776	3.156	2.104					2.104	1.654
11	14.431	95.66	97.518	1.858	98.306	100.721	2.415	1.610					1.610	1.498
12	19.331	95.66	97.229	1.569	98.974	100.611	1.637	1.092					1.569	1.265
13	24.231	95.66	96.939	1.279	99.09	100.502	1.412	0.941					1.279	1.031
14	25.769	95.66	96.848	1.188	99.384	100.467	1.083	0.722					1.188	0.958

E.1.9 Protection Works

E.1.9.1 D/S Protection Works

Normal scour depth R, as already calculated	4.02	m
Maximum scour=2R	8.04	m
Scour level	91.977	m
Scour depth below d/s bed, D2	3.683	m
I) filter and concrete blocks		
length =1.5*D2	5.524	m
Lets provide n number of rows of cement concrete blocks,n=	11	
Dimesnion of blocks(l,b,h)	0.5	m
Gaps(filled with bajri)=	0.04	m
Total length	5.9	m
thickness of graded filter=	0.5	m
Total thickness	1	m
ii)Launching apron		
Provide a launching apron of horizontal length equal to 1.5D2=	5.524	m
adopt length =	6	m
Volume of stone in the launching apron=2.24D2	8.250	$(m^3)/m$
Thickness in the horizontal position=	1.375	m

E.1.9.2 U/S Protection Works

i)	concrete block		
	Maximum scour=1.5*R	6.03	m
	Scour level	95.487	m
	Scour depth below u/s bed D1=	2.73	m
	length of concrete blocks =1.5*D1	4.095	m
	No of rows of cement concrete block=	8	
	dimension of block(l,b,h)	0.5	m
	gaps=	0.04	m
	Total length=	4.28	m
	Thickness of gravel pack below the concrete blocks=	0.5	m
	Total thickness=	1	m
ii)	Launching apron		
	Horizontal length of launching apron=1.5*D1	4.095	m
	adopt length of	4.1	m
	Volume of stone=2.24D1=	6.115	$(m^3)/m$
	Thickness in horizontal position=	1.492	m

E.2 Design of Undersluice

E.2.1 Data:

Design flood discharge in the river	131	m^3/s	
River bed level	98.217	m	
Hfl before construction of weir	100.517	m	river bed level+2.3m
FSL of canal	99.117	m	
Permissible afflux	1	m	
Bed retrogression	0.5	m	
Discharge concentration factor	20%		
Lacey's silt factor	1.5		
Pier contraction coefficient	0.1		
Full supply discharge of canal	2.292	m^3/s	
Safe exit gradient	0.17		

E.2.2 Crest Level

crest level of undersluice	98.217	m	
crest level of weir bay	98.917	m	
pond level (FSL +modular head)	99.617	m	FSL+0.5m

E.2.3 Weir Bay

From Lacey's formula		
waterway(P)=4.75*Q^0.5	54.366	m^3/s
lets fix the water way as follows:		
a) Undersluice portion		
Number of span	1	
1 bay	2.5	m

b)Weir bay portion		
number of span	3	
3 bays of 4.67 each	14	m
2 piers of 1.5m each	3	m
width of divide wall	1	m
Overall waterway between abutments	20.5 m=	(2.5+14+3+1)m
average discharge intensity q=	6.390	m^3/m

E.2.4 Discharge

u/s HFL after construction(hfl before construction +afflux)	101.517	m
Normal scour depth from lacey's fromula R=1.35*(q^2/f)^(1/3)	4.06	m
velocity of approach v=q/R	1.574	m/s
Head due to velocity of approach (v^2/2g)	0.126	m
u/s TEL =u/s HFL + head due to velocity of approach	101.647	m
a)Undersluice		
Head over the undersluice crest	3.43	m
Since the u/s floor and the crest of the undersluice are at same level,		
width of crest is large and will behave as a broad crested weir.		
Therefore discharge is given by		
Q=1.705(L'-0.1*nHe)He^1.5		
where n is the number of end contraction .		
Q=	27.007	m^3/s

b)Weir-bay		
Hand over the weir bay crest(He)=	2.73	m
Let us assume the crest width of weir as 0.5 m. As the crest width B<(2H/3),		
The weir will act as sharp crested weir. Therefore the discharge is given by		
Q=1.84(L'-0.1*n*He)*He^1.5	107.13	m^3/s
Total discharge Q=	134.137	m^3/s

Twice the design full supply discharge of canal=	4.584	m^3/s
20 % of the total design flood discharge=	26.2	m^3/s
The discharge capacity of undersluice is more than the minimum required by above two criteria and may be adopted.		

E.2.5 Design of the undersluice section

Let us find out the dischrage intensity q and head loss HL for different conditions		
a)High flood condition		
I) Without flow concentration and without retrogression		
q=1.705(He)^1.5	10.831	cumecs/ m
D/s TEL=	100.643	m
u/s TEL =	101.647	m
head loss Hl=	1.004	m
ii)with 20 % flow concentration and retrogression of 0.5		
q=	12.997	cumecs/ m
He=	3.900	m
u/s TEL=	102.117	m
d/s TEL after retrogression=	100.143	m
Head loss Hl	1.973	m

b)Pond level condition
i)without flow concentration and without retrogression

Head over the crest of undersluice 1.4 m

At the pond level condition ,the velocity of approach can be found from the discharge which occurs at that level

Q=	21.85	m^3/s
Average discharge intensity	1.066	m
Normal Scour depth ,R	1.231	m
velocity of approach	0.866	m/s
Head due to velocity of approach	0.038	m

u/s TEL	99.71	m
Discharge intensity (q)	3.11	cumecs/m
The water level on the d/s of the weir when a discharge of 21.85 occurs in the river is obtained from the stage discharge curve as 99.617m		
d/s TEL =	99.6	m
Head loss(Hl)=	0.11	m
ii)with 20 % flow condition and 0.5 m retrogression		
Discharge intensity	3.732	cumecs/m
Head over the crest	1.496	m
u/s TEL	99.713	
d/s water level after retrogression	99.117	m
d/s TEL	99.155	m
head loss ,Hl	0.558	m

E.2.6 Hydraulic jump calculation

SN	Item	High flood condition		Pond level condition	
		No concentration and retrogression	with concentration and retrogression	No concentration and retrogression	with concentration and retrogression
1	Discharge intensity ,	10.83	12.99	3.11	3.73
2	Head loss,HL	1	1.97	0.11	0.58
3	D/S specific energy(Ef2) from blench curve	4.2	5.1	1.5	2
4	Level at which jump will form=d/s TEL after retrogression-Ef2	96.44	95.04	98.1	97.15
5	u/s specific energy Ef1=Ef2 +HL	5.2	7.07	1.61	2.58
6	Pre jmp depth D1 corresponding to Ef1 from specific energy curve or calculated as Ef1=D1+((q/D1)^2)/(2*g)	1.23	1.21	0.76	0.59
7	Post jump depth D2	3.78	4.71	1.07	2

	Corresponding to Ef2 from formula Ef2=D2 +((q/D2)^2)/(2*g)				
8	Height of jump (D2-D1)	2.55	3.5	0.31	1.41
9	Length of concrete floor required beyond the jump=5*(D2-D1)	12.75	17.5	0.85	7.05
10	Initial froud number(Fr) =	2.535	3.116	1.499	2.628

The lowest water level at which the hydraulic jump is formed corresponds to the high flood condition
with flow concentration and retrogression and is 95.04m .Let us adopt the d/s floor level is 94.96 m
Here,the greatest length of horizontal floor is 17.5m
lets adopt the total length of impervious floor as 28m.

E.2.7 Design of sheet piles

Total discharge over undersluice	27.077	m^3/s
Overall waterway	2.5	m
Average discharge intensity	10.831	cumecs/m
Normal Scour depth ,R	5.77	m
Let us take the maximum scour depth as 2R	11.54	m
Bottom level of d/s pile =d/s water level after retrogrerssion-2R	88.477	m
Let us take the maximum scour depth at u/s as 1.5R	8.655	m
Bottom level of u/s pile =u/s water level -1.5R	92.862	m
depth of u/s pile below river bed	5.355	m
depth of d/s pile below river bed	6.483	m

E.2.8 Length of impervious floor

Maximum seepage head H=Pond level -d/s floor level	4.657	m
we have,safe exit gradient GE=H/d=1/(∏*lamda^0.5) =1/6		
ƛ=	1.88	
α=	2.57	

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b=α*d=	16.661	m
let us adopt overall length of 17m		
length of u/s floor	0.729	m
length of d/s glacis	9.771	m
length of d/s floor	17.5	m
Total length of impervious floor	28	m

E.2.9 Uplift pressure calculation

SN	flow condition	u/s water level(m)	d/s water level(m)	seepage head(m)	height of subsoil HGL above water level and its elevation(m)					
					u/s pile			d/s pile		
					phi E1=100	Phi D1=68.05	phi C1=65.34%	phi E2=47.63%	phi D2=34.29	phi C2=0
1	no flow	99.617	94.96	4.657	4.657	3.444	3.172	1.846	1.314	0
				RL	99.617	98.404	98.132	96.806	96.274	94.96
2	High flood with flow concentration and retrogression	101.517	100.017	1.5	1.5	1.109	1.021	0.594	0.423	0
				RL	101.517	101.126	101.038	100.611	100.44	100.017
3	Flow at pond level flow condition and retrogression	99.617	99.117	0.5	0.5	0.369	0.34	0.198	0.141	0
				RL	99.617	99.486	99.457	99.315	99.258	99.117

E.2.10 Hydraulic jump profile

For high flood condition and flow at pond level,the suction pressure is computed from the hydraulic jump profile
a) Prejump profile
b)High flow with 20% concentration and 0.5m retrogression
c)Pond level flow with 20 % concentration and 0.5 m retrogression
The hydraulic jump forms at RL 95.04m in the first case and 97.15m in the second case as already calculated.

E.2.11 Prejump Profile

		q=12.99		q=3.73	
		u/s TEL=102.116		u/s TEL=99.71	
Distance from the end of crest(m)	Level of glacis(m)	Ef1=u/s TEL - glacis level	D1(m)	Ef1=u/s TEL - glacis level	D1(m)
0	98.217	3.899	2.42	1.493	-0.58
2	97.547	4.569	1.75	2.163	0.69
3	97.217	4.899	1.61	2.493	0.61
3.201	97.15	4.966	1.59	2.56	0.6
4	96.88	5.236	1.52		
6	96.217	5.899	1.38		
8	95.55	6.566	1.27		
9	95.217	6.899	1.23		
9.531	95.04	7.076	1.21		

E.2.12 Post jump profile

For high flood condition, with concentration and retrogression,
Fr^2= 9.71 and D1=1.21 m
for pond levl condition,with concentration and retrogression
Fr^2=6.9 and D1=0.59 m

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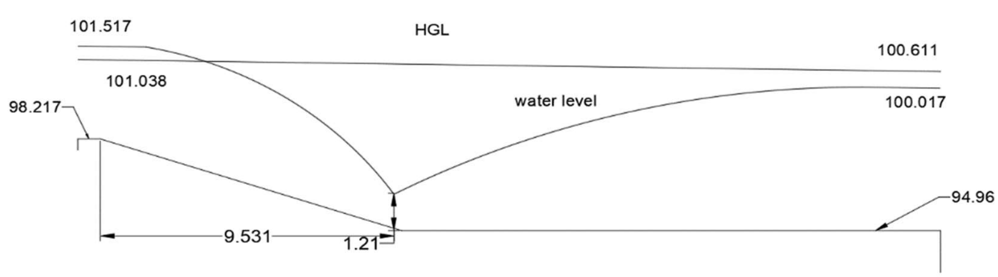
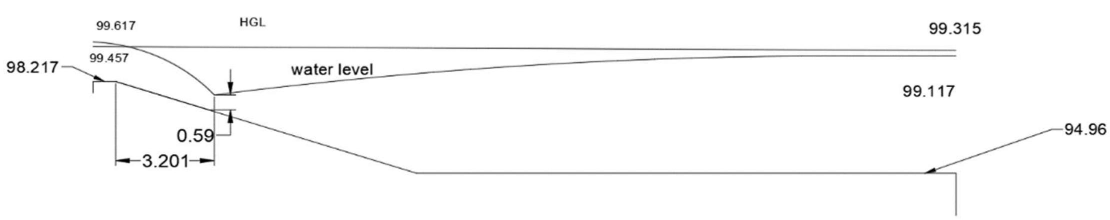
SN	High flood condition,y1=1.21				Pond level condition,y1=0.59 m			
	x/y1	y/y1	x(m)	y(m)	x/y1	y/y1	x(m)	y(m)
1	0.198	1	0.2396	1.21	1	1.2	0.59	0.708
2	1	1.2	1.21	1.452	2.5	1.9	1.475	1.121
3	2.5	2	3.025	2.42	5	2.5	2.95	1.475
4	5	2.6	6.05	3.146	10	3	5.9	1.77
5	10	3.5	12.1	4.235	11.13	3.1	6.5667	1.829
6	14.66	3.8	17.739	4.598	24.07	3.6	14.2013	2.124

x is the distance n m towards d/s from the point of jump formation



High flood condition



Pond level condition

E.2.13 Uplift and thickness of impervious floor calculation

		No flow condition			High flood condition				pond level condition					
SN	distance from last point of crest	level of glacis(m)	HGL(m)	Unbalance head(hd in m)	water level(m)	HGL(m)	unbalanced head(hd in m)	(2*hd)/3	water level(m)	HGL(m)	unbalanced head (hd in m)	(2*hd)/3	controlling uplift head(m)	thickness(m)
1	0	98.217	98.097	-0.120	100.637	101.027	0.390	0.260		99.457			0.260	0.210
2	3	97.217	97.955	0.738	98.827	100.981	2.154	1.436	97.827	99.446	1.619	1.079	1.436	1.158
3	3.201	97.15	97.946	0.796	98.74	100.978	2.238	1.492	97.75	99.445	1.695	1.130	1.492	1.203
4	6	96.217	97.813	1.596	97.597	100.935	3.338	2.226					2.226	1.795
5	9	95.217	97.671	2.454	96.447	100.890	4.443	2.962					2.962	2.389
6	9.531	95.04	97.646	2.606	96.25	100.882	4.632	3.088					3.088	2.490
7	9.771	94.96	97.635	2.675	96.17	100.878	4.708	3.139					3.139	2.531
8	27.271	94.96	96.806	1.846	99.558	100.611	1.053	0.702					1.846	1.489

E.2.14 Protection works

E.2.14.1 D/S protection works

Normal scour depth R, as already calculated	5.77	m
Maximum scour=2R	11.54	m
Scour level	88.477	m
Scour depth below d/s bed, D2	6.483	m
I) filter and concrete blocks		
length =1.5*D2	9.724	m
Lets provide n number of rows of cement concrete blocks,n=	19	
Dimension of blocks(length,breadth,height)	0.5	m
Gaps(filled with bajri)=	0.04	m
Total length	10.22	m
thickness of gravel filter=	0.5	m
Total thickness	1	m
ii)Launching apron		
Provide a launching apron of horizontal length equal to 1.5D2=	9.724	m
adopt length =	10	m
Thickness in a launched position (assumed)=	0.5	m
Volume of stone in the launching apron=2.24D2	14.522	$(m^3)/m$
Thickness in the horizontal position=	1.493	m
adopt thickness =	1.5	m

E.2.14.2 u/s protection works

i)concrete block		
Maximum scour=1.5*R	8.655	m
Scour level	92.862	m
Scour depth below u/s bed D1=	5.355	m
length of concrete blocks =1.5*D1	8.033	m
No of rows of cement concrete block=	16	
dimension of block(length,breadth,height)	0.5	m
gaps=	0.04	m
Total length=	8.6	m
Thickness of gravel pack below the concrete blocks=	0.5	m
Total thickness=	1	m
ii)Launching apron		
Horizontal length of launching apron=1.5*D1	8.033	m
adopt length of	8.1	m
Volume of stone=2.24D1=	17.993	$(m^3)/m$
Thickness in horizontal position=	2.221	m

E.3 DESIGN OF CANAL HEAD REGULATOR

E.3.1 Input parameters

Crest level of undersluice	98.217	m
Bed level of canal	98.309	m
FSL of canal	99.117	m
Pond level	99.617	m
High flood level at U/S	101.517	cumecs
Full supply discharge (Q)	1.91	m
Velocity of approach (V)	1.534	m/s
Head due to velocity of approach	0.12	m
Crest level of head regulator	99.217	m
Width of crest	3.2	m
Discharge per unit width	0.597	cumecs/m
Opening required for passing design discharge at HFL	0.149	m
Velocity through opening	4.0058	m/s
Head loss	0.4089	m
Total energy line at upstream at high flood	101.637	m
Total energy line d/s of gate at high flood	101.367	m
Total head loss	2.111	m

E.3.2 Hydraulic jump calculation

S.NO	Item	HFL	Pond Level
1	q(in cumecs/m)	0.597	0.597
2	U/S water level(m)	101.517	99.617
3	D/S water level(m)	99.117	99.117
4	U/S TEL just D/S of gate	101.2281	99.617
5	D/S TEL (m)	99.236	99.236
6	Head loss (m)	1.99	0.381
7	D/S specific energy (Ef2)	0.9	0.4
8	U/S specific energy (Ef1=Ef2+Head loss)	2.89	0.781
9	Pre jump depth (y1 in m)	0.080408	0.173
10	Froude No. (Fr)	8.3597	2.649
11	Post jump depth(y2 in m)	0.91129	0.5673
12	Length of impervious floor[5(y2-y1)] in m	4.155	1.9715
13	Level at which jump is formed(m)	98.236	98.836

Provide d/s floor at 98.2m Length of impervious floor =7m

E.3.3 Floor design using khosla theory

1	Depth of sheet piles:	5.355	m
2	RL of bottom of U/s Sheet pile (River bed-Pile depth)	92.862	m
D/S pile depth calculation			
1	q(Q/B)=	0.597	cumec/m
2	Scour depth(1.5R)=	1.26	m
3	Bottom of d/s pile (FSL of canal -Max scour depth)	97.857	m
4	Adopt Pile depth	3	m
5	D/S pile level	95.2	m
Length of impervious floor			
1	Seepage Head (H) (HFL-D/S floor level)	3.317	m
2	Exit gradient $\dfrac{H}{D*\sqrt{\lambda}*3.14}$	(1/6)	
3	λ	4.46	
4	α(b/d)	7.86	
5	b=	23.57 (adopt 24)	m
6	Length of U/S floor	13.95	m
7	Length of D/S floor	7	m
8	weir length	1	m
9	sloping glacis	3.051	m

Appendix E

E.3.4 Percentage Pressure calculation

For U/S Pile

d=	5.355	m
b=	24	m
α(b/d)	4.482	
λ	2.8	
ΦE1	100	%
ΦD1	72.22	%
ΦC1	68.682	%

For d/s Pile

d	3	m
b	24	m
α(b/d)	8	m
λ	4.53	
ΦC2	0	%
ΦD2	21.56%	%
ΦE2	31.14%	%

	Correctiot for floor thickness	Correction for mutual interference	Corrected uplift	
ΦE1			100	%
ΦD1			72.22	%
ΦC1	0.19282	2.07	70.945	%
ΦC2			0	%
ΦD2			0.216	%
ΦE2	1.59628	2.6082	26.93%	%

E.3.5 Uplift pressure calculation for different condition

S.NO	FLOW CONDITTION	U/S water level	D/S water level	Seepage head(m)	ΦE	ΦD1	ΦC1	ΦC2	ΦD2	ΦE2
	No flow				100	72.22	70.95	26.93	21.56	0
1		101.517	98.2	3.317	3.31	2.39	2.35	0.89	0.71	0
2	High flood	101.517	99.117	2.4	2.4	1.73	1.70	0.64	0.51	0
3	Pond level	99.617	99.117	0.5	0.5	0.36	0.35	0.13	0.10	0

Max uplift head is always found at no flow high flood condition.

So calculation of floor thick is done assuming this condition.

Uplift at end of crest	1.444	m
thickness	1.16664	m
adopt	1.2	m

Uplift at toe of d/s slope	1.2583	m
thickness	1.014	m
adopt	1.2	m

E.3.6 D/S Protection Works

D/S apron length=	1.5*depth of pile	4.5m
length of concrete block=	1.5*depth of pile	4.5m
adopt		1.5*1.5*.0.7block
Bajri gap=		10cm
Overall length of concrete block=		4.8m
gravel filter thickness		40 cm

E.4 Design of Settling Basin

E.4.1 Input data

canal discharge (Q)	1.91	m^3/s
Diameter of particle (d)	0.5	mm

E.4.2 Design

i)	From nomogram, for particle size 0.72 mm, settling velocity (Vs)	0.017	m/s
	surface area (As) = Q/Vs	112.35	m^2
	Take L:B = 10:1		
	B	3.352	m
	Adopt B =	3.4	m
	L =	34	m
ii)	Critical Bottom velocity in setling basin to prevent rescouring for particle size 0.72mm from nomogram (v) =	0.24	m/s
	Minimum flow depth in settling basin for this velocity is		
	y = Q/(B*v)	2.34	m
	Adopt, y =	2.4	m
iii)	Using nomogram, Scour velocity required to flush out particle of 0.9 mm is (v') =	1.92	m/s
	Assuming scour flow =1.25*Q	2.39	m^3/s
	Scour depth, Ys = Q/(B*v')	0.37	m
iv)	For Scour bed slope		
	V	1.92	m/s
	R	0.174	m
	n	0.015	
	Using mannings eqn		
	Slope (s) =	1/120	
v)	Assuming 1m wide flush gate, the head of water required at gate is given by		
	Q = 1.7 bYg^1.5		
	So, depth of gate (Yg)	1.25	m

The depth of scour gate sill below the floor of settling basin is calculated as = Yg +0.05 -Ys	0.93	m
Thus, Total depth required = 2.4 +0.4 Thus the dimension of settling basin required is (34*3.4*2.8)	2.8	m m^3

Appendix F:

Design of Main Canal and Canal Structures

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Appendix F

F.1 Canal Design

F.1.1 Input Parameters

Design discharge	1.91(m^3/s)
Lining type	Concrete lining
Manning's coefficeint	0.015
longitudnal slope	0.0011
Side slope	1.25:1(H:V)

F.1.2 Calculation

For most economical channel

A=BD+D^2(θ+COTθ)

P=B+2D(θ+COTθ)

Formula	Parameters	Values	Units
Applying Manning's formula	Mean Velocity	1.522564734	(m/s)
For Most Economic Channel A=BD+D^2(θ+COTθ)	Crossectional Area (A)(Q/V)	4.462541484	square meter
P=B+2D(θ+COTθ)	Wetted Perimeter(P)	3.1442777	m
	Hydraulic Radius(A/P)	0.761282367	m
	θ	51.34	degree
	θ(radian)	0.895597778	radian
	Water Depth (y)	0.80726	m

F.1.3 Input parameters

Design discharge	1.91(m^3/s)
Lining type	Concrete lining
Manning's coefficeint	0.015
longitudnal slope	0.001111111
Side slope	1.25:1(H:V)

Appendix F

F.1.4 Designed Canal Parameters

Θ	51.34	degree
θ(radian)	0.895597778	radian
Water Depth (y)	0.80726	m
Cross sectional Area (A)	1.254462262	square meter
Mean Velocity	1.522564734	(m/s)
Wetted Perimeter	3.1442777	m
Hydraulic Radius	0.40363	m

F.1.5 CUT-FILL VOLUME REPORT

Station	Cut Area (m^2)	Cut Volume (m^3)	Reusable Volume (m^3)	Fill Area (m^2)	Fill Volume (m^3)	Cum. Cut Vol. (m^3)	Cum. Reusable Vol. (m^3)	Cum. Fill Vol. (m^3)	Cum. Net Vol. (m^3)
0+000.000	3.85	0	0	0	0	0	0	0	0
0+050.000	7.27	278.15	278.15	0	0	278.15	278.15	0	278.15
0+100.000	0.51	194.44	194.44	3.64	91.12	472.59	472.59	91.12	381.47
0+150.000	1.54	51.13	51.13	1.07	117.93	523.72	523.72	209.05	314.67
0+200.000	4.07	140.32	140.32	0	26.8	664.04	664.04	235.86	428.18
0+250.000	5.14	230.42	230.42	0	0	894.45	894.45	235.86	658.6
0+300.000	2.29	185.91	185.91	0	0	1080.37	1080.37	235.86	844.51
0+350.000	1.33	90.56	90.56	1.47	36.78	1170.93	1170.93	272.63	898.3
0+400.000	0.86	54.69	54.69	2.56	100.65	1225.62	1225.62	373.29	852.33
0+450.000	0.76	40.56	40.56	2.82	134.26	1266.18	1266.18	507.54	758.64
0+500.000	1.92	67.18	67.18	0.44	81.32	1333.36	1333.36	588.87	744.49
0+550.000	0.85	69.33	69.33	2.61	76.28	1402.69	1402.69	665.15	737.54

0+600.000	0.52	34.38	34.38	3.58	154.76	1437.07	1437.07	819.9	617.17
0+650.000	0.27	19.82	19.82	5.35	223.17	1456.89	1456.89	1043.07	413.82
0+700.000	1.49	44.02	44.02	1.16	162.66	1500.91	1500.91	1205.73	295.19
0+750.000	0.71	55.17	55.17	3.01	104.13	1556.08	1556.08	1309.86	246.22
0+800.000	0.64	33.83	33.83	3.19	155	1589.91	1589.91	1464.86	125.05
0+850.000	0.57	30.23	30.23	3.44	165.83	1620.14	1620.14	1630.69	-10.55
0+900.000	0.75	32.87	32.87	2.87	157.81	1653.01	1653.01	1788.5	-135.5
0+950.000	0.94	42.19	42.19	2.32	129.76	1695.19	1695.19	1918.26	-223.06
1+000.000	0.88	45.52	45.52	2.5	120.53	1740.72	1740.72	2038.79	-298.07
1+050.000	0.82	42.41	42.41	2.67	129.38	1783.13	1783.13	2168.17	-385.04
1+100.000	0.88	42.51	42.51	2.49	129.13	1825.64	1825.64	2297.3	-471.67
1+150.000	0.95	45.81	45.81	2.32	120.16	1871.45	1871.45	2417.46	-546.02
1+200.000	0.97	48.05	48.05	2.26	114.45	1919.5	1919.5	2531.91	-612.41
1+250.000	1	49.19	49.19	2.21	111.77	1968.7	1968.7	2643.68	-674.98
1+300.000	1.05	51.21	51.21	2.07	106.93	2019.9	2019.9	2750.6	-730.7
1+350.000	1.11	54.13	54.13	1.94	100.17	2074.03	2074.03	2850.77	-776.74
1+400.000	1.26	59.39	59.39	1.6	88.57	2133.43	2133.43	2939.34	-805.91
1+450.000	1.42	67.2	67.2	1.27	71.97	2200.63	2200.63	3011.31	-810.68
1+500.000	1.39	70.35	70.35	1.35	65.66	2270.98	2270.98	3076.97	-805.99
1+550.000	1.36	68.62	68.62	1.43	69.53	2339.61	2339.61	3146.49	-806.89
1+600.000	1.49	71.2	71.2	1.16	64.65	2410.81	2410.81	3211.14	-800.33
1+650.000	1.64	78.23	78.23	0.9	51.43	2489.04	2489.04	3262.57	-773.53
1+700.000	1.77	85.18	85.18	0.68	39.42	2574.22	2574.22	3301.99	-727.77
1+750.000	1.91	92.03	92.03	0.46	28.36	2666.25	2666.25	3330.35	-664.1
1+800.000	2.09	99.96	99.96	0.19	16.32	2766.22	2766.22	3346.67	-580.45

1+850.000	2.33	110.42	110.42	0.01	5.02	2876.64	2876.64	3351.68	-475.05
1+900.000	3.27	140.04	140.04	0	0.15	3016.68	3016.68	3351.83	-335.16
1+950.000	4.24	187.7	187.7	0	0	3204.38	3204.38	3351.83	-147.46
2+000.000	4.02	206.39	206.39	0	0	3410.76	3410.76	3351.83	58.93
2+050.000	3.83	196.14	196.14	0	0	3606.91	3606.91	3351.83	255.07
2+100.000	4.09	197.92	197.92	0	0	3804.82	3804.82	3351.83	452.99
2+150.000	4.49	214.57	214.57	0	0	4019.4	4019.4	3351.84	667.56
2+200.000	5.34	245.89	245.89	0	0	4265.29	4265.29	3351.84	913.45
2+250.000	6.2	288.7	288.7	0	0	4553.99	4553.99	3351.84	1202.15
2+300.000	6.48	317.19	317.19	0	0	4871.18	4871.18	3351.84	1519.34
2+350.000	6.8	332.04	332.04	0	0	5203.22	5203.22	3351.84	1851.39
2+400.000	5.94	318.48	318.48	0	0	5521.7	5521.7	3351.84	2169.86
2+450.000	5.07	275.38	275.38	0	0.05	5797.08	5797.08	3351.89	2445.19
2+500.000	6.08	278.94	278.94	0	0.05	6076.02	6076.02	3351.94	2724.07
2+550.000	5.8	297.06	297.06	0	0	6373.08	6373.08	3351.94	3021.14
2+600.000	5.37	279.29	279.29	0	0	6652.37	6652.37	3351.94	3300.43
2+650.000	4.44	245.2	245.2	0	0	6897.57	6897.57	3351.94	3545.63
2+700.000	3.5	198.31	198.31	0	0	7095.88	7095.88	3351.94	3743.94
2+750.000	1.89	134.76	134.76	0.48	12.07	7230.64	7230.64	3364.02	3866.62
2+800.000	1.16	76.24	76.24	1.83	57.91	7306.89	7306.89	3421.93	3884.96
2+850.000	0.67	45.56	45.56	3.11	123.56	7352.44	7352.44	3545.49	3806.96
2+900.000	0.3	24.06	24.06	4.51	190.39	7376.5	7376.5	3735.88	3640.62
2+950.000	0.26	13.84	13.84	4.7	230.17	7390.33	7390.33	3966.05	3424.28
3+000.000	0.22	11.91	11.91	4.9	239.9	7402.24	7402.24	4205.95	3196.29
3+050.000	0.08	7.5	7.5	5.79	267.07	7409.74	7409.74	4473.03	2936.71

3+100.000	0	2.04	2.04	6.83	315.33	7411.78	7411.78	4788.36	2623.42
3+150.000	1.65	41.23	41.23	0.88	192.68	7453	7453	4981.03	2471.97
3+200.000	1.43	76.93	76.93	1.28	53.93	7529.93	7529.93	5034.96	2494.97
3+250.000	1.33	68.93	68.93	1.47	68.73	7598.86	7598.86	5103.7	2495.17
3+300.000	1.23	63.97	63.97	1.67	78.67	7662.83	7662.83	5182.37	2480.47
3+350.000	0.25	37.1	37.1	4.72	159.79	7699.93	7699.93	5342.15	2357.78
3+400.000	0	6.33	6.33	9.03	343.71	7706.27	7706.27	5685.87	2020.4
3+450.000	0	0	0	9.12	453.67	7706.27	7706.27	6139.54	1566.73
3+500.000	0	0	0	9.2	457.96	7706.27	7706.27	6597.5	1108.77
3+550.000	0	0	0	9.87	476.8	7706.27	7706.27	7074.31	631.96
3+600.000	0	0	0	10.55	510.5	7706.27	7706.27	7584.8	121.47
3+650.000	0.09	2.37	2.37	5.76	407.83	7708.64	7708.64	7992.63	-283.99
3+700.000	0.9	24.92	24.92	2.44	205.07	7733.56	7733.56	8197.7	-464.14
3+750.000	0.81	42.76	42.76	2.69	128.28	7776.32	7776.32	8325.98	-549.66
3+800.000	0.72	38.17	38.17	2.95	140.99	7814.49	7814.49	8466.97	-652.48
3+850.000	0.7	35.52	35.52	3	148.64	7850.01	7850.01	8615.61	-765.6
3+900.000	0.69	34.7	34.7	3.05	151.11	7884.72	7884.72	8766.72	-882
3+950.000	0.66	33.77	33.77	3.11	153.96	7918.49	7918.49	8920.68	-1002.2
4+000.000	0.64	32.73	32.73	3.18	157.21	7951.21	7951.21	9077.88	-1126.7
4+050.000	0.62	31.7	31.7	3.24	160.46	7982.91	7982.91	9238.35	-1255.4
4+100.000	2.8	85.57	85.57	0	81.05	8068.49	8068.49	9319.4	-1250.9
4+150.000	1.77	114.2	114.2	0.68	16.99	8182.69	8182.69	9336.39	-1153.7
4+200.000	1.17	73.57	73.57	1.79	61.86	8256.26	8256.26	9398.25	-1142
4+250.000	1.04	55.27	55.27	2.11	97.54	8311.54	8311.54	9495.79	-1184.3
4+300.000	0.91	48.61	48.61	2.43	113.31	8360.15	8360.15	9609.1	-1249

4+350.000	0.9	45.18	45.18	2.45	121.81	8405.32	8405.32	9730.91	-1325.6
4+400.000	0.89	44.78	44.78	2.47	122.84	8450.11	8450.11	9853.75	-1403.6
4+450.000	1.01	47.52	47.52	2.17	115.99	8497.63	8497.63	9969.74	-1472.1
4+500.000	1.13	53.57	53.57	1.88	101.43	8551.2	8551.2	10071.2	-1520
4+550.000	0.94	51.71	51.71	2.36	106	8602.91	8602.91	10177.2	-1574.3
4+600.000	0.75	42.24	42.24	2.84	129.98	8645.15	8645.15	10307.1	-1662
4+650.000	0.81	39.07	39.07	2.69	138.34	8684.22	8684.22	10445.5	-1761.3
4+700.000	0.86	41.83	41.83	2.54	130.71	8726.06	8726.06	10576.2	-1850.1
4+750.000	0.86	43.04	43.04	2.56	127.43	8769.09	8769.09	10703.6	-1934.5
4+800.000	0.85	42.65	42.65	2.58	128.46	8811.74	8811.74	10832.1	-2020.4
4+850.000	0.84	42.24	42.24	2.6	129.57	8853.98	8853.98	10961.7	-2107.7
4+900.000	0.83	41.8	41.8	2.63	130.75	8895.78	8895.78	11092.4	-2196.6
4+950.000	0.82	41.26	41.26	2.66	132.23	8937.04	8937.04	11224.7	-2287.6
5+000.000	0.81	40.61	40.61	2.7	134.01	8977.66	8977.66	11358.7	-2381
5+050.000	0.8	40.24	40.24	2.7	135.05	9017.89	9017.89	11493.7	-2475.8
5+100.000	0.8	40.12	40.12	2.71	135.36	9058.02	9058.02	11629.1	-2571.1
5+150.000	0.92	43.15	43.15	2.38	127.3	9101.17	9101.17	11756.4	-2655.2
5+200.000	1.06	49.53	49.53	2.06	111.07	9150.7	9150.7	11867.4	-2716.7
5+250.000	1.02	51.97	51.97	2.14	105.03	9202.67	9202.67	11972.5	-2769.8
5+300.000	0.99	50.32	50.32	2.22	108.99	9252.99	9252.99	12081.5	-2828.5
5+350.000	1.13	52.88	52.88	1.9	103.06	9305.86	9305.86	12184.5	-2878.7
5+400.000	1.27	59.85	59.85	1.59	87.43	9365.71	9365.71	12272	-2906.3
5+450.000	1.24	62.81	62.81	1.65	81.01	9428.52	9428.52	12353	-2924.4
5+500.000	1.22	61.58	61.58	1.7	83.59	9490.09	9490.09	12436.6	-2946.5
5+550.000	1.37	64.68	64.68	1.39	77.31	9554.77	9554.77	12513.9	-2959.1

5+600.000	1.52	72.31	72.31	1.1	62.33	9627.08	9627.08	12576.2	-2949.1
5+650.000	1.52	76.15	76.15	1.1	55.08	9703.23	9703.23	12631.3	-2928.1
5+700.000	1.52	76	76	1.11	55.35	9779.23	9779.23	12686.6	-2907.4
5+750.000	1.68	80.03	80.03	0.82	48.3	9859.26	9859.26	12734.9	-2875.7
5+800.000	1.86	88.45	88.45	0.54	34.11	9947.71	9947.71	12769	-2821.3
5+850.000	2.04	97.26	97.26	0.27	20.32	10045	10045	12789.4	-2744.4
5+900.000	2.22	106.47	106.47	0.01	6.92	10151.4	10151.4	12796.3	-2644.8
5+950.000	2.08	107.57	107.57	0.21	5.34	10259	10259	12801.6	-2542.6
6+000.000	1.94	100.47	100.47	0.41	15.48	10359.5	10359.5	12817.1	-2457.6
6+050.000	2	98.38	98.38	0.33	18.51	10457.9	10457.9	12835.6	-2377.7
6+100.000	2.05	101.2	101.2	0.25	14.35	10559.1	10559.1	12850	-2290.9
6+150.000	2.11	104.1	104.1	0.16	10.16	10663.2	10663.2	12860.1	-2196.9
6+200.000	2.17	107.09	107.09	0.08	5.95	10770.3	10770.3	12866.1	-2095.8
6+250.000	2.15	107.95	107.95	0.11	4.74	10878.2	10878.2	12870.8	-1992.6
6+300.000	2.12	106.69	106.69	0.15	6.5	10984.9	10984.9	12877.3	-1892.4
6+350.000	2.11	105.72	105.72	0.17	7.85	11090.6	11090.6	12885.1	-1794.5
6+400.000	2.09	105.05	105.05	0.19	8.8	11195.7	11195.7	12893.9	-1698.3
6+450.000	1.97	101.66	101.66	0.36	13.73	11297.3	11297.3	12907.7	-1610.3
6+500.000	1.85	95.62	95.62	0.55	22.73	11393	11393	12930.4	-1537.4
6+550.000	2.03	97.15	97.15	0.27	20.48	11490.1	11490.1	12950.9	-1460.8
6+600.000	2.22	106.36	106.36	0.01	7.08	11596.5	11596.5	12958	-1361.5
6+650.000	2.19	110.38	110.38	0.05	1.41	11706.9	11706.9	12959.4	-1252.5
6+700.000	2.17	109.02	109.02	0.08	3.27	11815.9	11815.9	12962.6	-1146.8
6+750.000	2.53	117.54	117.54	0	2.1	11933.4	11933.4	12964.7	-1031.3
6+800.000	2.99	138.04	138.04	0	0	12071.4	12071.4	12964.7	-893.29

6+850.000	3.44	160.64	160.64	0	0	12232.1	12232.1	12964.7	-732.65
6+900.000	3.89	183.25	183.25	0	0	12415.3	12415.3	12964.7	-549.4
6+950.000	4.34	205.85	205.85	0	0	12621.2	12621.2	12964.7	-343.55
7+000.000	4.8	228.45	228.45	0	0	12849.6	12849.6	12964.7	-115.1
7+050.000	2.82	190.5	190.5	0	0	13040.1	13040.1	12964.7	75.4
7+100.000	1.68	112.63	112.63	0.83	20.65	13152.8	13152.8	12985.4	167.38
7+150.000	1.86	88.44	88.44	0.54	34.12	13241.2	13241.2	13019.5	221.71
7+200.000	2.04	97.5	97.5	0.26	19.96	13338.7	13338.7	13039.5	299.24

F.2 Canal Design for Drop

F.2.1 Design Procedure

Discharge in Parent Channel (Q) = 1.91 Cumecs

Discharge in distributary ($Q/5$) = 0.382 Cumecs

FLS of parent Channel u/s (Near regulating structure) = 98.851

$$1.91 = \frac{1}{0.015} * (0.808 * \mathrm{b}) * \left(\frac{\mathrm{b} + 0.808}{\mathrm{b} + 2 * 0.808}\right)^{\frac{2}{3}} * \left(\frac{1}{900}\right)^{\frac{1}{2}}$$

Solving we get;

b = 1.860 m

Provide b = 1.9 m

Free Board = 0.5 m

Bed level of parent channel = 98.851 – 0.808 = 98.043 m

Head Regulator

$$0.382 = \frac{1}{0.015}(2\mathrm{d}^2) * \left(\frac{\mathrm{d}}{2}\right)^{2/3} * \left(\frac{1}{900}\right)^{1/2}$$

Solving, we get;

d= 0.4738 m

FSL of distributary = 98.043+ 0.4738

=98.517 m

Cross Regulator:

$$1.528 = \frac{1}{0.015} * (0.808 * \mathrm{b}) * \left(\frac{\mathrm{b} + 0.808}{\mathrm{b} + 2 * 0.808}\right)^{\frac{2}{3}} = \left(\frac{1}{900}\right)^{\frac{1}{2}}$$

On solving, we get;

B= 1.57 m

Provide b=1.6 m

FSL of Parent Channel d/s = 98.043 + 0.808-0.2

= 98.651 m

F.3 Drop Structure Design

F.3.1 Design of 1m Drop

F.3.1.1 Data

Fully supply discharge of canal	1.91	m^3/s
Drop height	1	m
U/S FSL	97	m
D/S FSL	96	m
U/S and D/S fully supply depth	0.8	m
U/S bed level	96.2	m
D/S bed level	95.2	m
Bed Width	1.9	m

F.3.1.2 Design of crest

Trial-1		
Assume, Top width (Bt)	0.5	m
Using Q=1.84LH^1.5*(H/Bt)^(1/6)		
H=	0.69	m
Approach velocity (Va)	1.26	m/s
Approach velocity Head (Ha)	0.08	m
U/S TEL= U/S FSL + Ha	97.08	m
RL of Crest=U/S TEL - H	96.39	m
Ht of crest above U/S floor=	0.19	m
Ht of crest above D/S bed (d) =	1.19	m
Tod width=0.55*d^0.5	0.60	m
Trial-2		
Assume , Top width = 0.6 m		
Using Q=1.84LH^1.5*(H/Bt)		
H=	0.66	m
Approach velocity (Va)	1.26	m/s
Approach velocity Head (Ha)	0.08	m
U/S TEL= U/S FSL + Ha	97.08	m
RL of Crest=U/S TEL - H	96.42	m
Ht of crest above U/S floor=	0.22	m
Ht of crest above D/S bed (d) =	1.22	m
Tod width=0.55*d^0.5	0.61	m
Thus adopt top width = 0.6 m		

Thickness at base B1 = (H + d)/G	0.84	m

F.3.1.3 Design of cistern

Length of cistern (Lc) =5(H*HL)^0.5	4.06	m
Depth of cistern (x) = 0.25*(H*HL)^0.67	0.19	m
RL of bed cistern = RL of D/S bed - x	95.01	m

F.3.1.4 Design of impervious floor

Seepage head, Hs =d=	1.22	m
Bligh's Coef =	6	
Required creep length=	7.32	m
U/S cut off pile depth (d1) = D/3	0.27	m
D/S cut off pile depth (d2) = D/2	0.4	m
provide		
d1	0.4	m
d2	0.5	m
Vertical length of creep =2(d1 +d2)	1.8	m
Length of horizontal impervious floor	5.52	m
Min. horizontal length required=2(D+L)+HL	6.4	m
Provide		
Min D/s impervious floor	6.5	m
Min U/s floor length	2	m

F.3.1.5 Calculation of uplift pressure and thickness

Total creep length	11.14	m
Provide min. thickness of 0.3 m for U/S floor		
Max unbalanced head for D/S toe of crest wall (h) =	0.88	m
Required thickness (t) = h/(G-1)	0.71	m
Hence, provide 0.75 m thick cement concrete floor over laid by 0.2 m brick pitching		

F.3.2 Design of 0.5 m drop

F.3.2.1 Data

Fully supply discharge of canal	1.91	m^3/s
Drop height	0.5	m
U/S FSL	97	m
D/S FSL	96	m
U/S and D/S fully supply depth	0.8	m
U/S bed level	96.2	m
D/S bed level	95.2	m
Bed Width	1.9	m

F.3.2.2 Design of crest

Trial-1		
Assume, Top width (Bt)	0.5	m
Using Q=1.84LH^1.5*(H/Bt)^(1/6)		
H=	0.65	m
Approach velocity (Va)	1.26	m/s
Approach velocity Head (Ha)	0.08	m
U/S TEL= U/S FSL + Ha	97.08	m
RL of Crest=U/S TEL - H	96.43	m
Ht of crest above U/S floor=	0.23	m
Ht of crest above D/S bed (d) =	1.23	m
Tod width=0.55*d^0.5	0.61	m
Trial-2		
Assume , Top width = 0.6 m		
Using Q=1.84LH^1.5*(H/Bt)		
H=	0.66	m
Approach velocity (Va)	1.26	m/s
Approach velocity Head (Ha)	0.08	m
U/S TEL= U/S FSL + Ha	97.08	m
RL of Crest=U/S TEL - H	96.42	m
Ht of crest above U/S floor=	0.22	m
Ht of crest above D/S bed (d) =	1.22	m
Tod width=0.55*d^0.5	0.61	m
Thus adopt top width = 0.6 m		

Thickness at base B1 = (H + d)/G	0.85	m

F.3.2.3 Design of cistern

Length of cistern (Lc) =5(H*HL)^0.5	2.87	m
Depth of cistern (x) = 0.25*(H*HL)^0.67	0.12	m
RL of bed cistern = RL of D/S bed - x	95.08	m

F.3.2.4 Design of impervious floor

Seepage head, Hs =d=	1.22	m
Bligh's Coef =	6	
Required creep length=	7.32	m
U/S cut off pile depth (d1) = D/3	0.27	m
D/S cut off pile depth (d2) = D/2	0.4	m
provide		
d1	0.4	m
d2	0.5	m
Vertical length of creep =2(d1 +d2)	1.8	m
Length of horizontal impervious floor	5.52	m
Min. horizontal length required=2(D+L)+HL	5.9	m
Provide		
Min D/s impervious floor	6	m
Min U/s floor length	2	m

F.3.2.5 Calculation of uplift pressure and thickness

Total creep length	10.65	m
Provide min. thickness of 0.3 m for U/S floor		
Max unbalanced head for D/S toe of crest wall (h) =	0.81	m
Required thickness (t) = h/(G-1)	0.65	m
Hence, provide 0.65 m thick cement concrete floor over laid by 0.2 m brick pitching		

F.4 Design of Small Bridges

F.4.1.1 Design value adopted:

Width : 6m

Carriage width =4m

Foothpath = 1m (both side)

Length = 7m

F.4.2 Deck Slab Design:

Dead Load calculation:

Assume thickness: 450mm with 40mm clear cover

Effective depth (d) = 410mm

Wearing coarse= 75mm assumed.

Then,

Dead Load of slab= 0.6*25 =15kN/m2

Dead load of w/c =0.075*22kN/m2 = 1.650KN/m2

Then,

Effective Length= 7m (span)

Then,

Dead Load moment = $W*L^2/8$ = 141.71KNm

F.4.3 Live Load Calculation:

We consider Irc class A vehicle to cover 1 lane and other remaining is loaded with $5kN/m^2$ loading.

From Effective width method: (Cl. 305.16 IRC 21)

$be = \alpha a(1-a/L)+b_\omega$

case 1:

when only one wheel is considered:

be= 4.708m

but only 4m is available

so total width=4m

total load = 114kN

Then, impact factor = 0.336

Partial safety factor = 1.5

Max live load moment for design = 165.63 KNm

Total design moment = 165.63+141.71 = 307.34KNm

F.4.3.1 Reinforcement Design:

Slab thickness:

d_{bal} >= $(Mu/0.138*fck*b)^{1/2}$ = 298mm

but adopted = 540mm so Ok.

Shear:

Designed By effective width so safe in shear. (Cl. IRC 24)

Moment in longitudinal direction = 472.71KNm

Ast required= 1662.62mm^2

Provide 16mm bar @110mm c/c.

Transverse Reinforcement:

Ast min = 720mm2

Provide 12mm bar @ 130mm c/c

Temperature Reinforcement

Provide 250mm2 per 1000mm (Cl. Irc 5)

So provide 12 mm bar @ 300 mm c/c

Railing details:

Provide 225mm * 225mm * 1.1 m RC post

3- 48.3mm dia rod @4.37 kg/m

Provide 6 railing post on both side.

F.4.4 Abutment Design

Preliminary Assumptions:

Span = 7m

And Height of abutment = 7/1.5 = 4.66m

So, providing 4.5m high abutment.

Preliminary Design of Abutment:

a) Height of abutment
4.5m

b) Material M25 concrete
Fe415 steel

c) Geometry

i) Seating width
0.5 m

ii) Ht of dirt wall,
m
0.3m

iii) Thickness of dirt wall
0.25m

iv) Width of stem of abutment
0.6m

v)Thickness of footing
0.6m

vi) Length of Abutment
6m

vii) Width of footing
2m

viii) Thickness of abutment cap
0.15m

ix) Size of Approach Slab

Length 3.5m Clause 214.2,

Appendix F

Thickness	0.3m	IRC 6
Width =	6m	

Active pressure coefficient

Angle of earth face of wall with vertical, \alpha =	0	deg	0	radians
Angle of internal friction, \phi	35	deg	0.610865	radians
Angle of friction between wall & earth fill, \delta =	22.5	deg	0.392699	radians
Slope of earth fills, \beta =	0	deg	0	radians

1. Static Earth Pressure Coefficient

C1 = cos^2.(phi - alpha) / { cos^2.(alpha) * cos.(delta + alpha)=	0.7263
C2 = sin(phi + delta) sin(phi - beta) / { cos(alpha - beta) * cos(delta + alpha) }=	0.52361
K_a=	0.24448

a) Dead Load from Superstructure

(Unfactored DL without wearing coat) 59.791 KN/m

b.1) Wt. of Wearing Coat 3.85 KN/m

b.2) Wt. of appraoch slab 26.25 KN/m

c) Live Load from supersture

Live load 228.99 KN/m

d) Load from breaking effort

F_{br}^H 3.717 KN/m

F_{br}^V 0.955 KN/m

e) Wind Load

Transverse wind load per unit length(F_W^T)

0.505 KN/m

Longitidunal wind load per unit length(F_W^L)

0.12625 KN/m

Vertical wind load per unit length(F_W^V)

2.434 KN/m

g) Load due to temperautre variation, creep and shrinkage effect:

G	0.9	Clause 4.2.1,
long strain	0.0005	Table 1,

			IRC 83	
delta		1.5	mm	
h_0		64	mm	
Approax size	300*400		mm2	
	F_CST		2.3571	KN
	F_cst		1.17855	KN/m

h) Self wt. of abutment

			58.5	KN/m

j) load due to static earth pressure

delta	22.5	radians	0.3926991	deg
gamma_soil		18	KN/m3	
H		3.9		
K_a		0.24447663		
P_a		33.4664059	KN/m	
P_{EP}^{H(S)}		30.91892743	KN/m	
P_{EP}^{V(S)}		12.80703908	KN/m	

k) load due to dynamic earth pressure

Adopte K_a	0.365024238	
P_a	49.968168	KN/m
P_{EP}^{H(D)}	46.16456769	KN/m
P_{EP}^{V(D)}	19.12199004	KN/m

l) Surcharge Load

1.2 m earth fill surcharge		Clause 710.4.4, IRC 78, 2014
P_{sur}	20.59471132	KN/m
P_{sur}^{H(S)}	19.02703227	KN/m
P_{sur}^{V(S)}	7.881254817	KN/m

m) Backfill weight on heel slab of footing:

W_{BF}	56.16	KN/m

n) Weight of footing:

W_{ft}	30	KN/m

Appendix F

Stability check for basic combinations of loads: (per meter)									
Load Description	Symbol	Value, KN	\gamma f Overturnir	Restoring	Lever Arm(m)	Overturning Moment	Restoring Moment	Shear Force	Vertical Load
a) Dead Load from Superstructure	DL {ss}	59.791		0.9	0.7		37.6683		53.8119
b.1) Wt. of Wearing Coat	DL {wc}	3.85		1	0.7		2.695		3.85
b.2) Wt. of appraoch slab	DL {as}	26.25		0.9	1.2		28.35		23.625
c) Live Load from supersture	LL	228.99		0	0.7		0		0
d) Load from breaking effort	F {br}^{H}	3.717	1.15		4.2	17.95311		4.27455	
	F {br}^{V}	0.955		0	0.7		0		0
e) Wind Load	F {w}^{L}	0.12625	1.5		4.2	0.795375		0.18938	
f)temp, creep and shrinkage load	F {cst}	1.17855	1.5		4.2	7.424865		1.76783	
g) Dead load of abutment	DL {ab}	58.5		0.9	0.9		47.385		52.65
h) load due to static earth pressure	P {EP}^{H(S)}	30.9189	1.5		2.238	103.7948		46.3784	
	P {EP}^{V(S)}	12.807		1	0.9		11.5263		12.807
i) Surcharge Load	P {sur}^{H(S)}	19.027	1.2		2.55	58.22272		22.8324	
	P {sur}^{V(S)}	7.88125		0	0.9		0		0
j)weight of Backfill	W {bf}	56.16		0.9	1.6		80.8704		50.544
k)weight of footing	W {ft}	30		0.9	1		27		27
Total(KN)						188.1909	235.495	75.4426	224.288

Check for factor of safety for: Clause 706.3.4, IRC 78 - 2014

a) Sliding

1.85771 >1.5 ok (phi = 35deg = 0.55851 radians)

b) Overturning

So Buried Type is constructed

1.25136 < 2 .

then , (per meter length)

height of filling	2.6m	
kp	3.3311	for phi =35 degree
pressure force	202.66KN	
horizontal pressure force	177.46KN	fs=1
veritcal pressure force	76.998KN	fs=1.5

Appendix F

additional

factored restoring moment produced	218.639KNm	
factored overturning moment produced	20.097	
new FOS against overturning	2.17	>2 ok

F.5 Culvert Design

F.5.1 Data Adopted

Since in terai region, rainy season stream can have discharge up to $1m^3/s$ to $2m^3/s$.

So, design discharge = $2m^3/s$

And culvert shape adopted = Box culvert

Width/depth = 1.5

F.5.2 Design

$V=(2g*H_L/(1+K_e+K_L))^{1/2}$

Where ,

Ke=0.505 (for box culvert Entry loss)

$K_L=n^2*L*2g/R^{4/3}$

Length = 11m

n=0.012 (concrete lining in culvert)

then,

Q=A.V

Gives, H_L=1.304m

And V=1.304m/s

F.6 Design of Cross Regulator

F.6.1 Data :

Discharge of Parent Channel	1.91	Cumecs
Discharge of Distributary	0.382	Cumecs
FSL of parent channel, u/s	98.851	m
FSL of parent channel, d/s	98.651	m
Bed width of parent channel, u/s	1.9	m
Bed width of parent channel, d/s	1.6	m
Depth of water in the parent channel, d/s and u/s	0.808	
	0.808	m
FSL of Distributary	98.517	m
Silt factor	1.5	
Assume safe exit-gradient	0.14	
Glacis slope	2	:1

F.6.2 Design of Cross Regulator

F.6.2.1 Crest Level = 98.043 m

Q= B.√h(1.69h + 3.54h1)

h = u/s FSL - d/s FSL = 0.2 m

h_1 = d/s FSL - Crest Level = 0.608 m

Clear waterway required, B= Q / h1/2*(1.69h + 3.54h_1) =1.71 m

One bays of m is provided

F.6.2.2 Downstream Floor Level or Cistern Level

Discharge intensity q = 1.19375 cumecs/m

H_L = 0.2m

From Plate 10.1 Ef_2 = 1.2 m

d/s floor level = d/s FSL - Ef_2 = 97.451 m

d/s Bed level = d/s FSL - W d = 97.843 m

Cistern or d/s floor is provided at R.L 97.843

F.6.2.3 Length of d/s floor

From Plate 10.2	For Ef_2 = 1.2 m		y_2 = 0.9 m
For Ef_1	=	Ef_1 + HL= 1.4m	y_1 = 0.3 m
Length Required =	$5\,(y_2 - y_1)$ = 3m		

F.6.3 Vertical cut-offs

U/S Cut-offs =(y_u/3) + 0.6 = 0.87 m

Bottom level of u/s cutoff= 97.17 m

D/s Cut-offs = $(y_d/2) + 0.6$ = 1 m

Bottom level of d/s cutoff= 96.84 m

F.6.4 Total Floor Length from Exit Gradient Considerations

GE = (H/d)*(1/π*√λ)

H =u/s FSL -d/s bed level = 1.008 m

d= Depth of d/s cutoff 1.00 m

(1/π*√λ) = 0.142

From Plate 11.2

For (1/π*√λ) = 0.142, α = 10

Floor Length, b = α*d= 10.04 m say 10 m

Min d/s floor length required = (2/3)*b = 6.7 m

Glacis length = 0.4 m

U/s floor length = 2.9 m

F.6.5 Calculations for Uplift Pressures

F.6.5.1 Upstream Cut-off

d= 0.87 m

b= 10 m

(1/α) = (d/b) = 0.087

From khosla

Curve

λ	=	6.27	
ϕD	=	18%	
ϕE	=	26%	
ϕE_1 =	100%		
ϕD_1 =	100 - ϕD	82%	
ϕC_1 =	100 - ϕ E	74%	

Assuming minimum floor thickness (Ft) at U/S end = 0.3 m

Correction for floor thickness at u/s =($(D_1 - C_1)$xFt)/d= 2.74 %

ϕC_1 (corrected)=76.59 %

F.6.5.2 Downstream Cut-off

d=1 m, b=10 m

(1/α) = (d/b) = 0.1

From khosla

Curve

λ	=	5.51	
$\phi E_2 = \phi E$	=	28	%
$\phi D_2 = \phi D$	=	19	%
ϕC_2	=	0	%

Assuming minimum floor thickness (Ft) at D/s end = 0.4m

Correction for floor thickness at d/s =($(E_2 - D_2)$xFt)/d = 3.40 %

ϕE_2 (corrected)= 26.63 %

F.6.6 Floor Thicknesses : D/S Floor

At Toe of Glacis

% pressure at toe of the Glacis = 57.44%

Max unbalanced head at toe of glacis due to max static head = 1.55 m

Head Due to Dynamic action can be taken as = 50% $(y_2 - y_1)$+ϕ*HL = 0.41m

Thickness required at toe of glacis due to dynamic action = 0.15

Thickness required at toe of glacis = 1.2508m

Thickness for a distance of 2 m from toe of Glacis

% Pressure	=	28.1	%
Max unbalanced head	=	0.76	m
Thickness required	=	1.1	m

Thickness at beyond 5m from toe of Glacis

% Pressure	=	2.9 %
Max unbalanced head	=	0.62 m
Thickness required	=	0.9 m

F.6.7 Upstream protection

Scour depth D		=	0.87	m
Launching apron thickness	t	=	0.4	m
Launching apron Required	L	=	(2.25*D)/t	
			4.9m	

Downstream protection

Scour depth	D	=	1.004 m
Launching apron Required	L	=	5.6 m

F.7 Design of Distributary Head Regulator

F.7.1 Data:

Discharge of Parent Channel	1.91	Cumecs
Discharge of Distributary	0.382	Cumecs
FSL of parent channel, u/s	98.851	m
FSL of parent channel, d/s	98.651	m
Bed width of parent channel, u/s	1.9	m
Bed width of parent channel, d/s	1.6	m
Depth of water in the parent channel, d/s and u/s	0.808	
	0.808	m
FSL of Distributary	98.517	m
Silt factor	1.5	m
Assume safe exit-gradient	1/7	
Glacis slope	2:1	

F.7.2 Design

F.7.2.1 Crest Level

Crest Level is kept 0.3 m higher then bed level parent channel

Crest Level = 98.343 m

F.7.2.2 Water Way

$$Q = B.\sqrt{h}(1.69h + 3.54h_1)$$

h=u/s FSL - d/s FSL =0.334 m

h_1 = d/s FSL - Crest Level = 0.174 m

Clear waterway required .

B = Q / $h^{\frac{1}{2}}$ *(1.69h + 3.54h_1) = 0.56 m

which is very less than bed width of distributary =0.948

1 bays of 1.6m is provided so, clear water way is 1.6

F.7.2.3 Downstream Floor Level or Cistern Level

q= 0.23875 cumecs/m

HL = 0.334 m

From Plate 10.1 Ef_2= 0.3 m

$Ef_1 = Ef_2 + H_l$= 0.634 m

From Plate 10.2

For Ef_1 = 0.634 m y1 = 0.2 m

For Ef2 = 0.3 m y2 = 0.4 m

R.L of Cistern = d/s FSL - Ef_2 = 98.217 m

Length Required = 5 (y2- y1) = 1 m

F.7.2.4 Vertical cut-offs

U/S Cut-offs =(y_u/3) + 0.6 = 0.87 m

Bottom level of u/s cutoff= 97.17 m

D/s Cut-offs = $(y_d/2) + 0.6$ = 0.8 m

Bottom level of d/s cutoff= 97.38 m

F.7.2.5 Total Floor Length from Exit Gradient Considerations

H = u/s FSL -d/s bed level = 0.634 m

d = Depth of d/s cutoff = 0.8 m

GE = (H/d)*(1/π*√λ)

(1/π*√λ) = 0.189

From Plate 10.2

For (1/π*√λ) 0.189 , α = 8

Floor Length, b = α*d = 6.6952 m , say 7 m

Min d/s floor length required = (2/3)*b = 4.7 m

Length of d/s Glacis =0.3 m

Length of crest = 1.0 m

Length of u/s Glacis =0.3 m

F.7.2.6 Calculations for Uplift Pressures

a. Upstream Cut-off

d= 0.87 m

b= 7 m

(1/α)= (d/b) = 0.124

From khosla

Curve

λ	=	4.56
ϕD	=	21%
ϕE	=	31%
ϕE_1 =	100%	
ϕD_1 =	100 - ϕD	79%
ϕC_1 =	100 - ϕ E	69%

Assuming minimum floor thickness (Ft) at U/S end = 0.4 m

Correction for floor thickness at u/s =(($D_1 - C_1$)xFt)/d= 4.39 %

ϕC_1 (corrected)=73.35 %

Downstream Cut-off

d=0.8 m, b=7 m

(1/α) = (d/b) = 0.120

From khosla

Curve

λ	=	4.71	
ϕ E_2 = ϕE	=	30	%
ϕD_2 = ϕD	=	21	%
ϕC_2	=	0	%

Assuming minimum floor thickness (Ft) at D/s end = 0.42m

Correction for floor thickness at d/s =(($E_2 - D_2$)xFt)/d = 4.70 %

ϕE_2 (corrected)= 25.78 %

F.7.2.7 Floor Thicknesses: D/S Floor

At Toe of Glacis

% pressure at toe of the Glacis = 57.50 %

Max unbalanced head at toe of glacis due to max static head = 1.55 m

Head Due to Dynamic action can be taken as = 50% ($y_2 - y_1$)+ϕ*HL = 0.29 m

Thickness required at toe of glacis = 1.3 m

Thickness at beyond 1.17 from toe of Glacis

% Pressure	=	37.1	%
Max unbalanced head	=	1.00	m
Thickness required	=	1.2	m

Thickness at beyond 2.33 from toe of Glacis

% Pressure	=	16.7	%
Max unbalanced head	=	0.45	m
Thickness required	=	0.5506	m

Thickness at beyond 3.50 from toe of Glacis

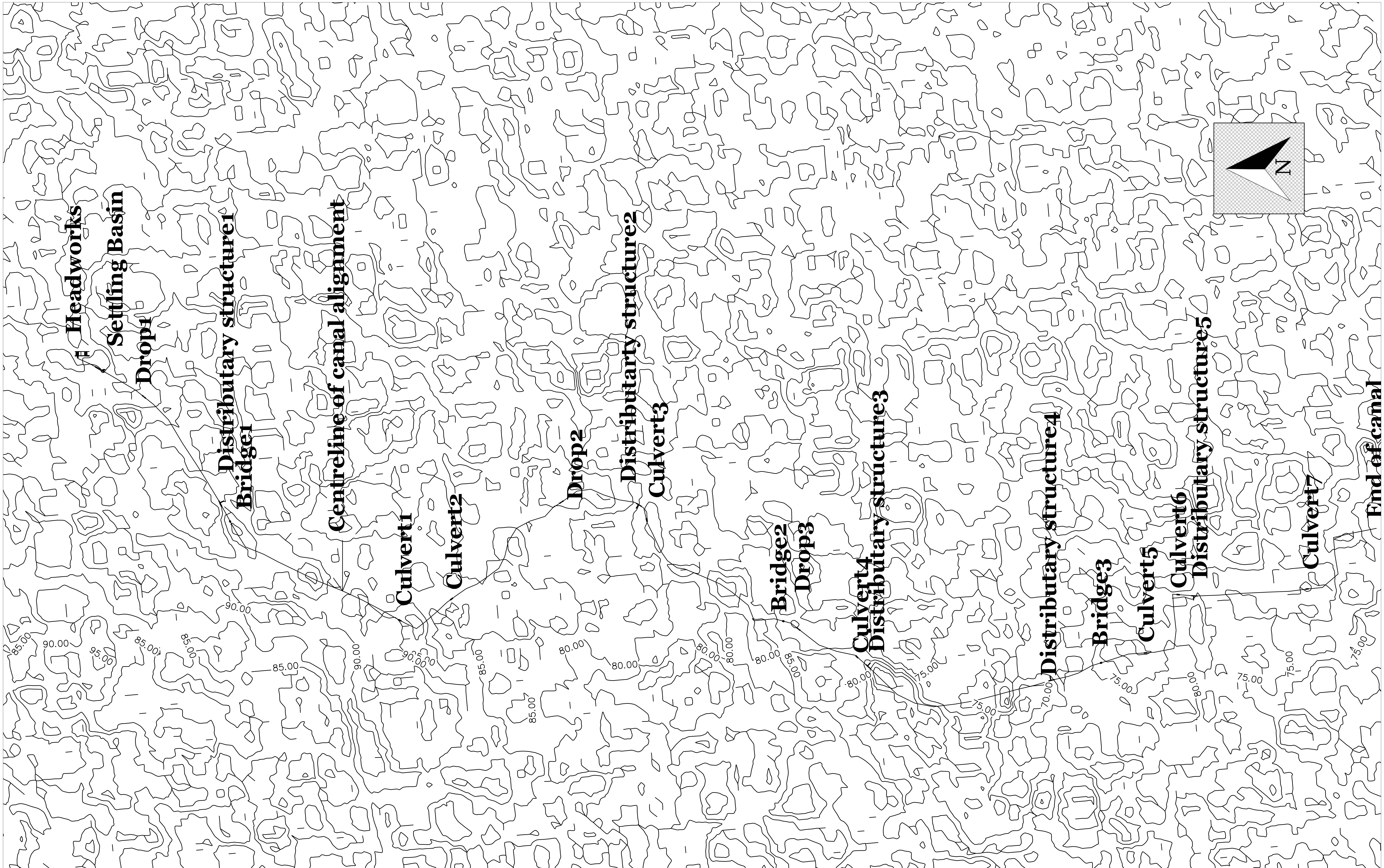
% Pressure	=	9.9	%
Max unbalanced head	=	0.27	m
Thickness required	=	0.3	m

F.7.2.8 Upstream protections

Scour depth D		=	0.76	m
Launching apron thickness	t	=	0.4	m
Launching apron Required	L	=	(2.25*D)/t	
			4.3m	
Downstream protection				
Scour depth	D	=	0.6 m	
Launching apron Required	L	=	3.4 m	

APPENDIX G:

DRAWINGS



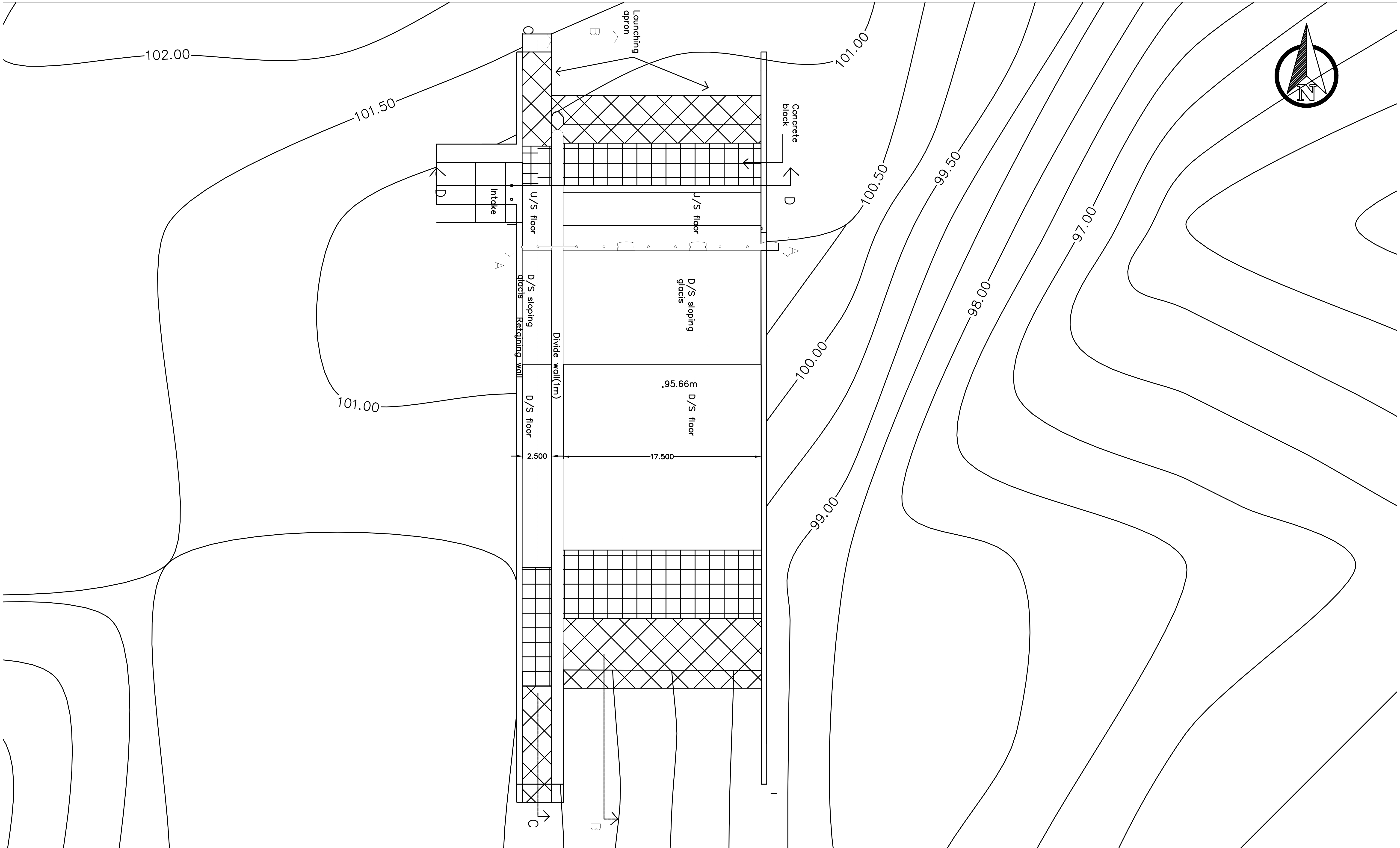
TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

LAYOUT OF PROJECT

PREPARED BY:
Saugat Paudel
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:11000
DrawingNo.1

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

HEADWORKS PLAN

PREPARED BY:
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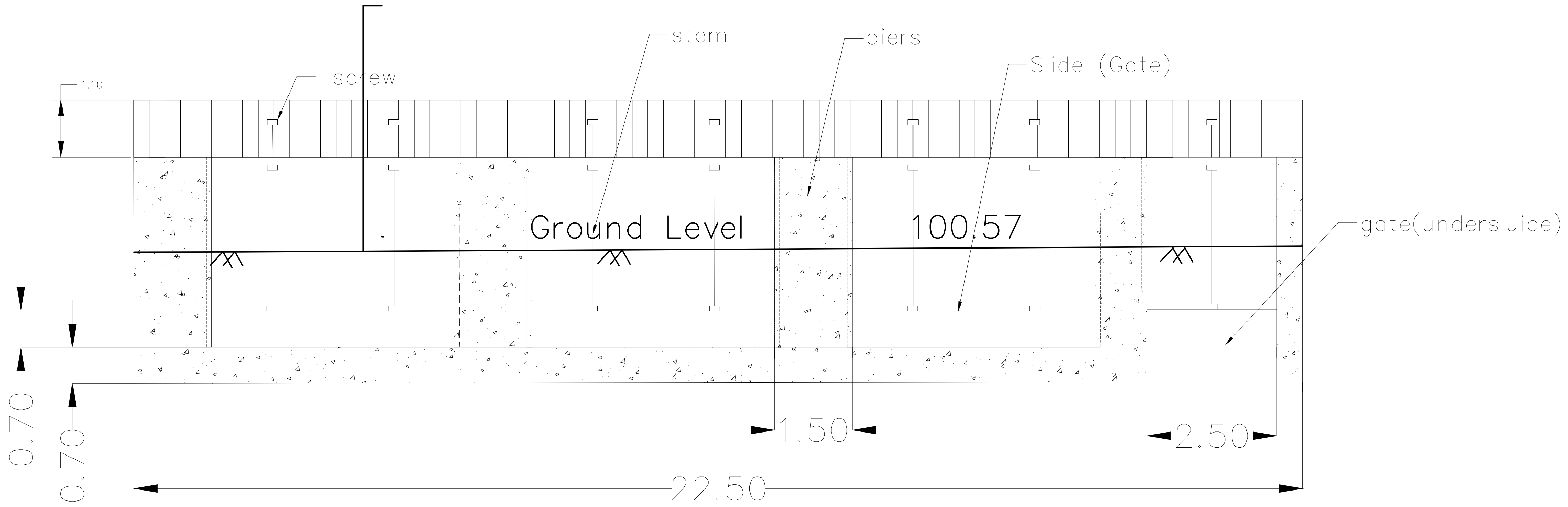
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1: 300

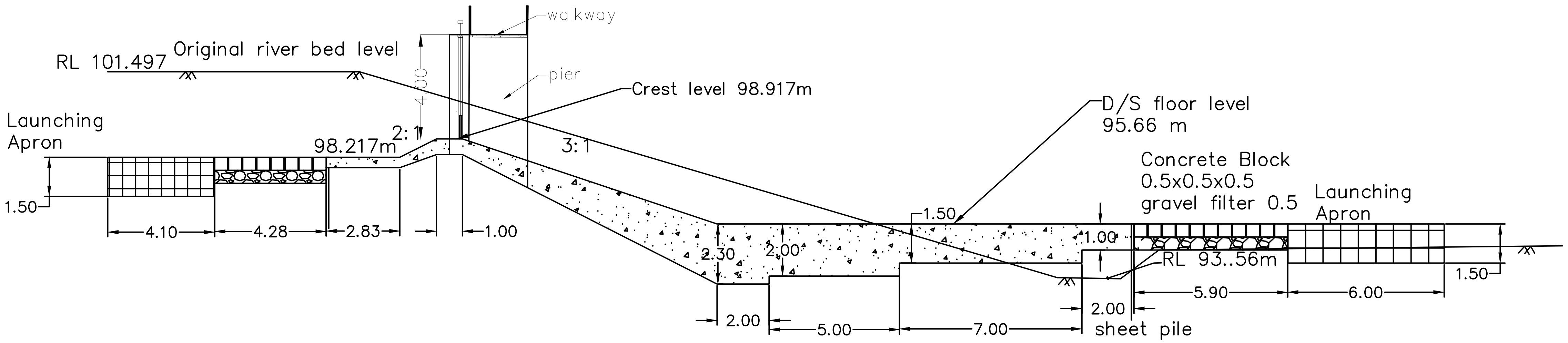
Drawing No.2

PRE–FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

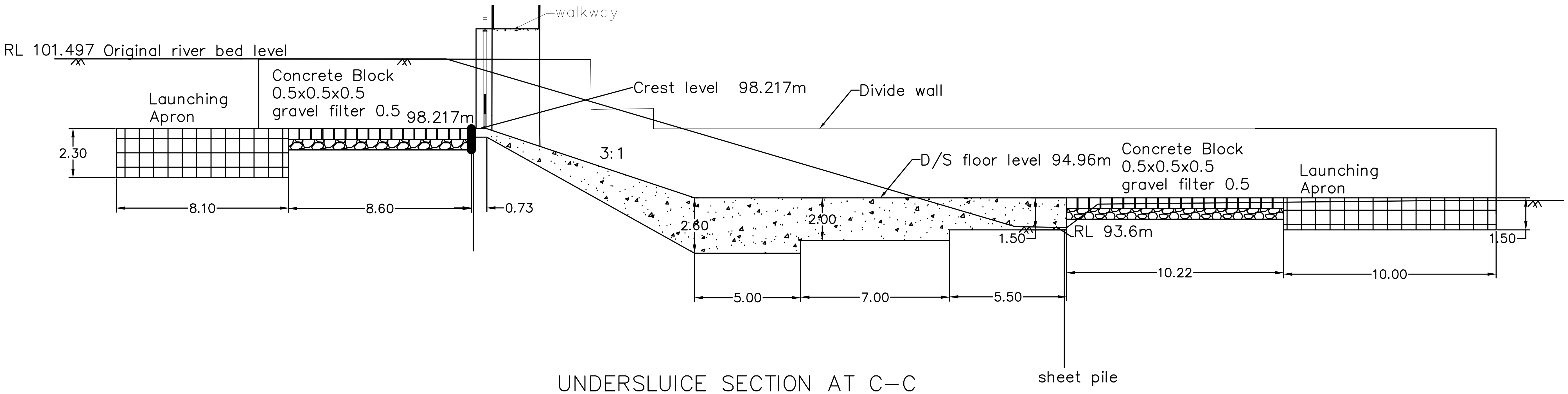


SECTION AT A-A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	WEIR AND UNDERSLUICE SECTION	PREPARED BY: Saugat Paudel — Shital Upadhea Siddhartha Kandel — Sudip Adhikari Sundar Mani Bhattrai — Suwaj Aryal	Scale 1:100 DrawingNo.3	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

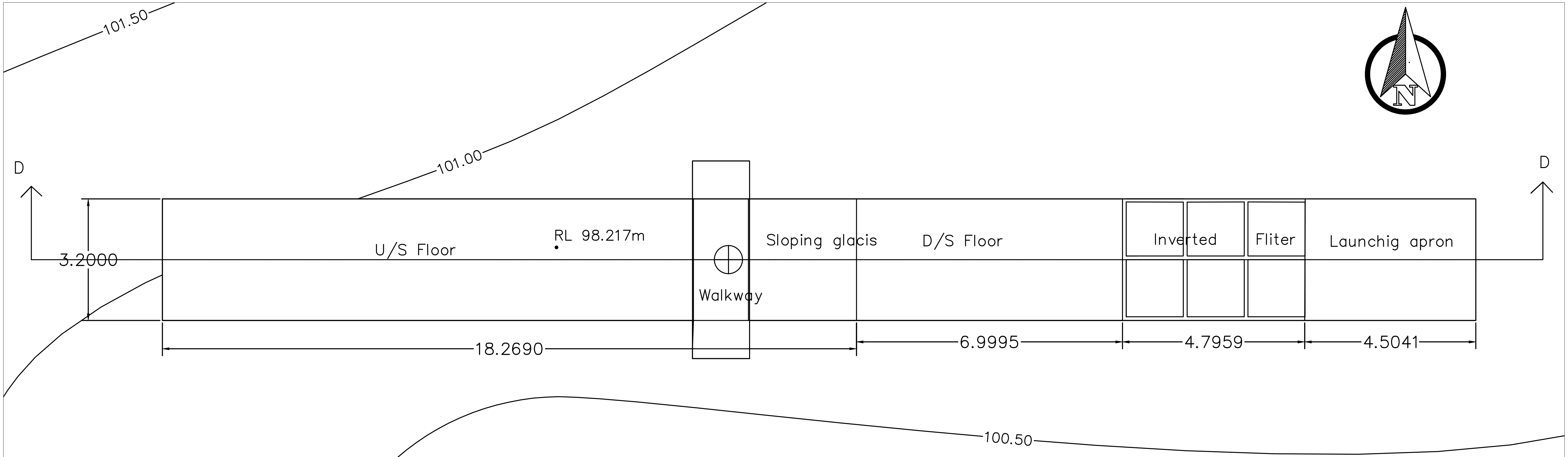


WEIR SECTION AT B–B

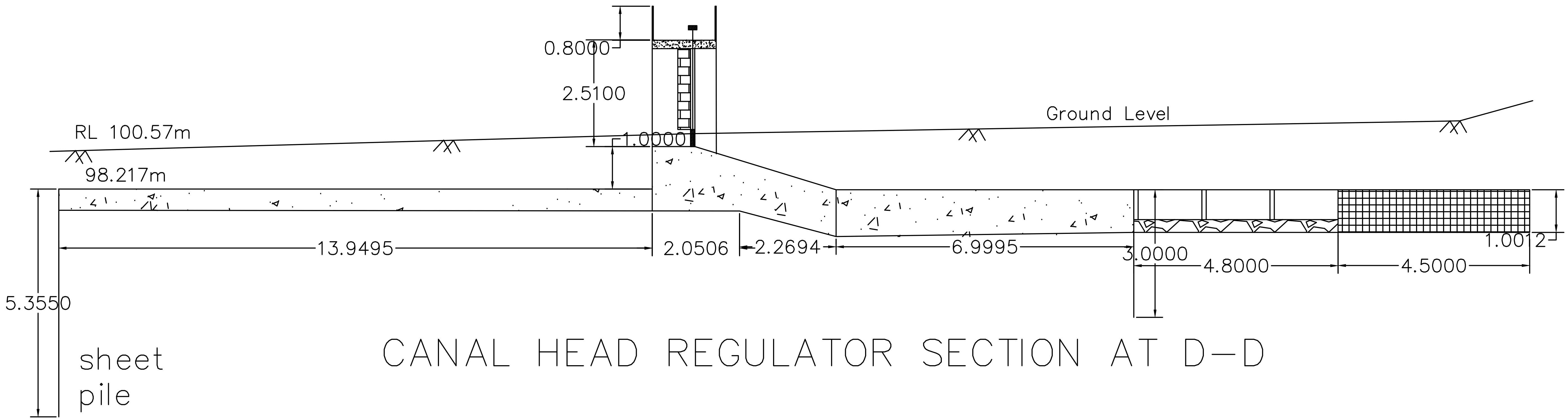


UNDERSLUICE SECTION AT C–C

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	WEIR AND UNDERSLUICE SECTION	PREPARED BY: Saugat Paudel — Shital Upadhea Siddhartha Kandel — Sudip Adhikari Sundar Mani Bhattrai — Suwaj Aryal	Scale 1:200 DrawingNo.4	PRE–FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



CANAL HEAD REGULATOR PLAN



CANAL HEAD REGULATOR SECTION AT D-D

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

CANAL HEAD REGULATOR

PREPARED BY:
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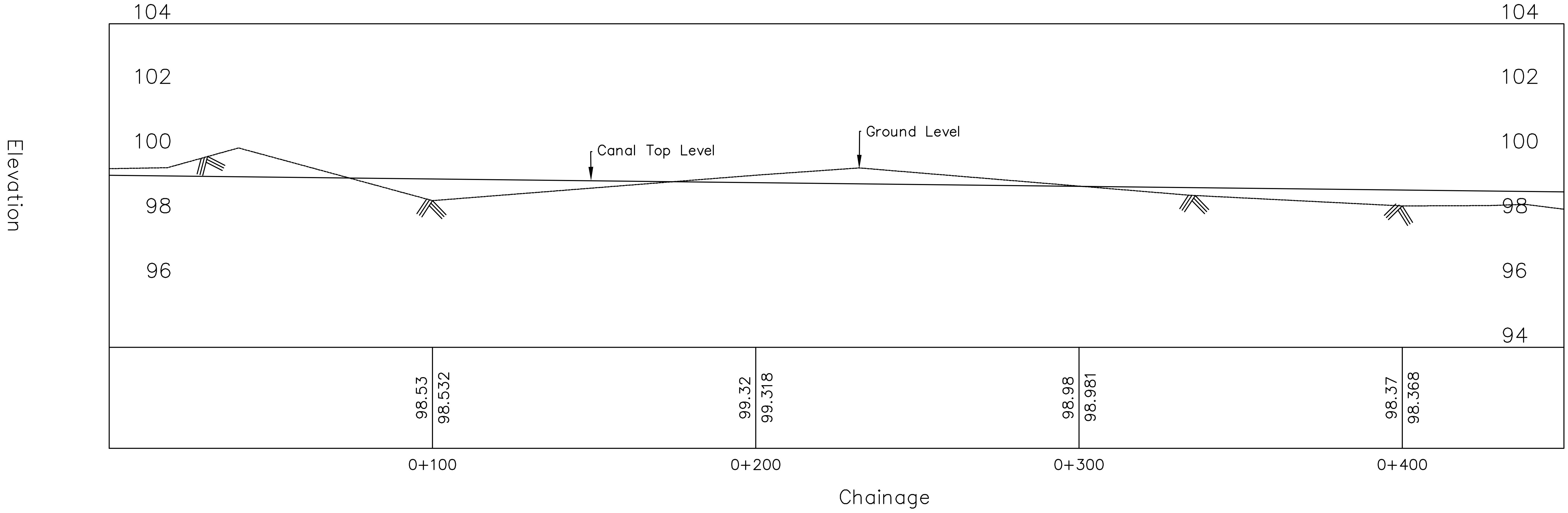
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:100

Drawing No.5

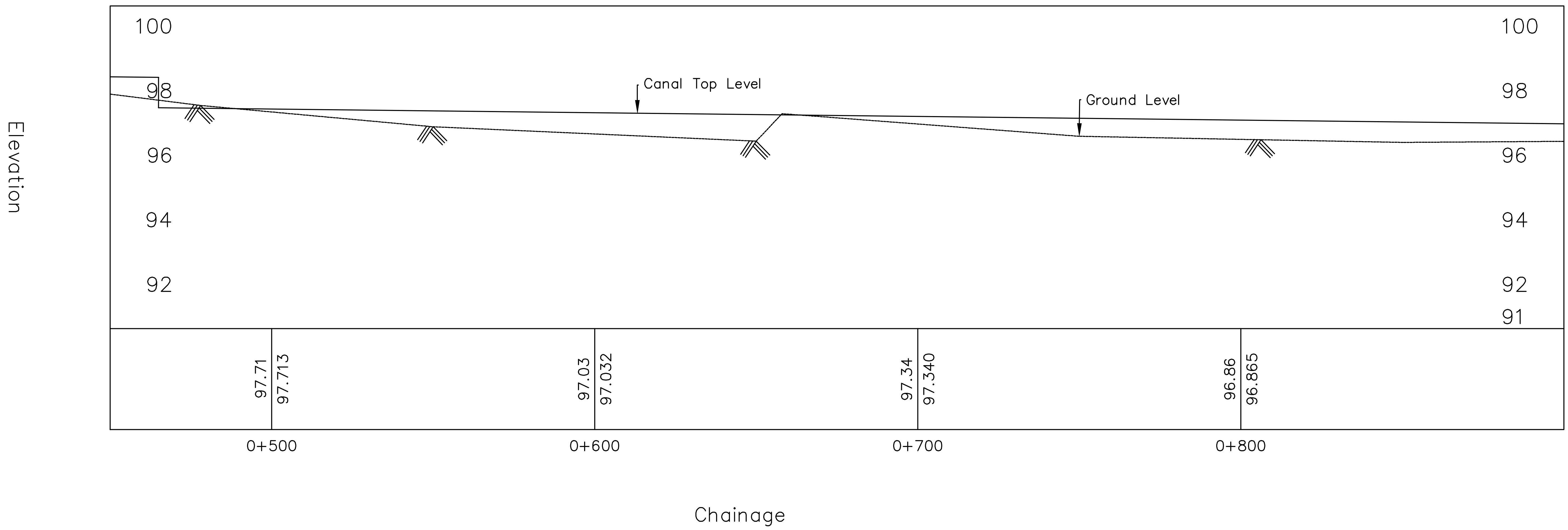
PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



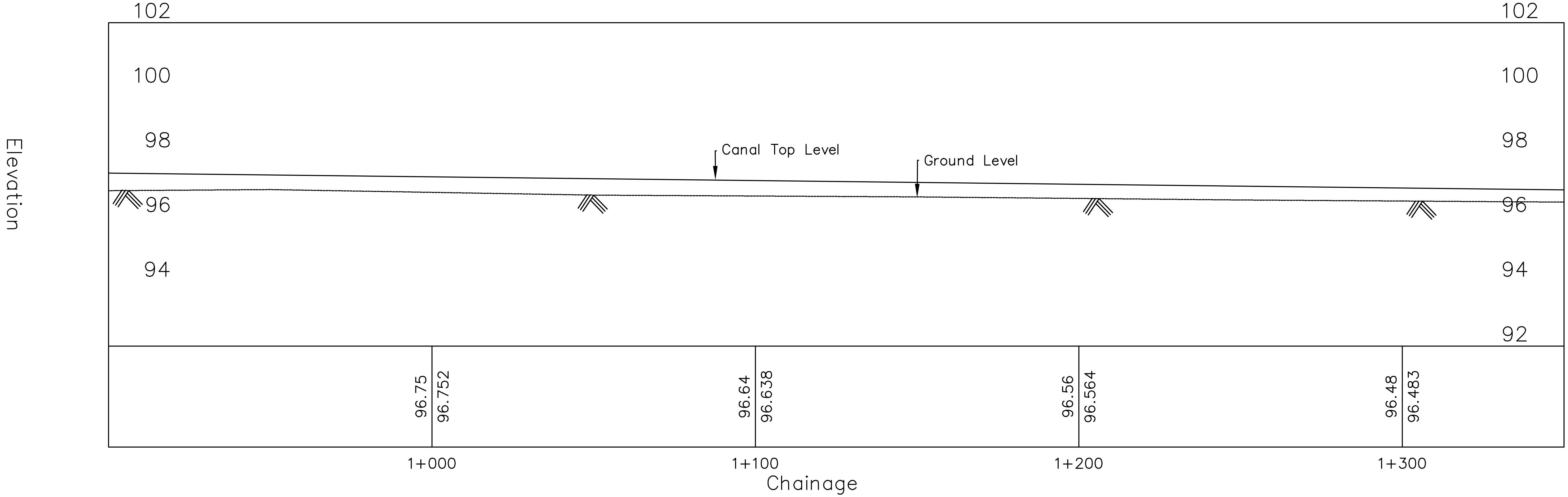
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:1250 DrawingNo.6	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



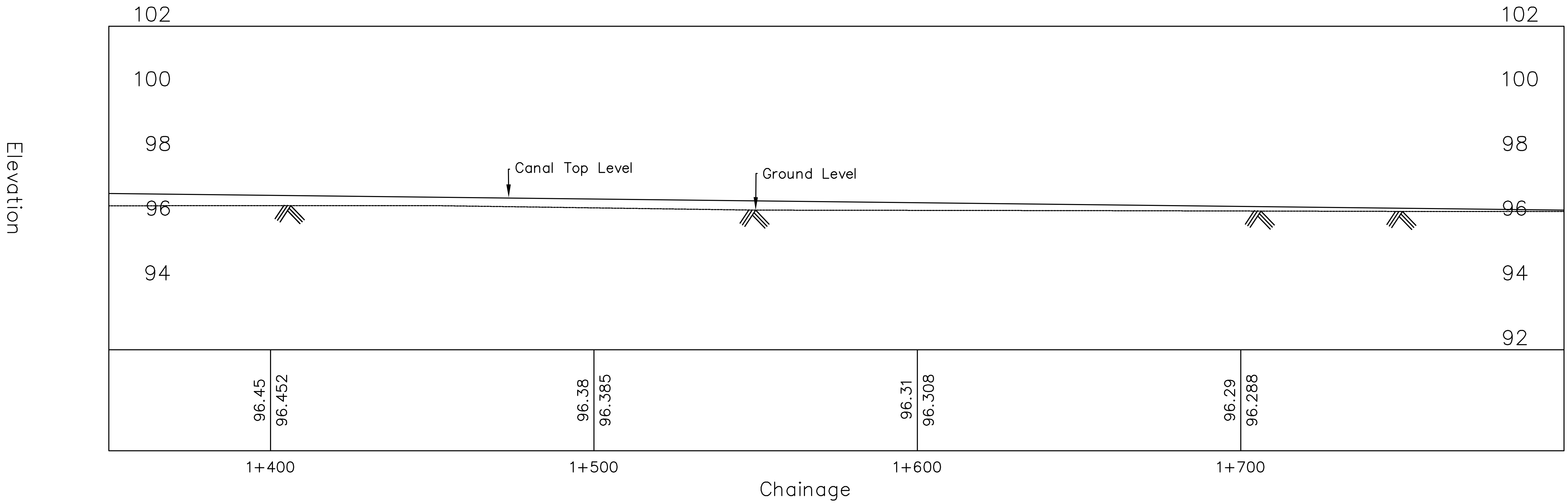
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:1250 DrawingNo.7	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



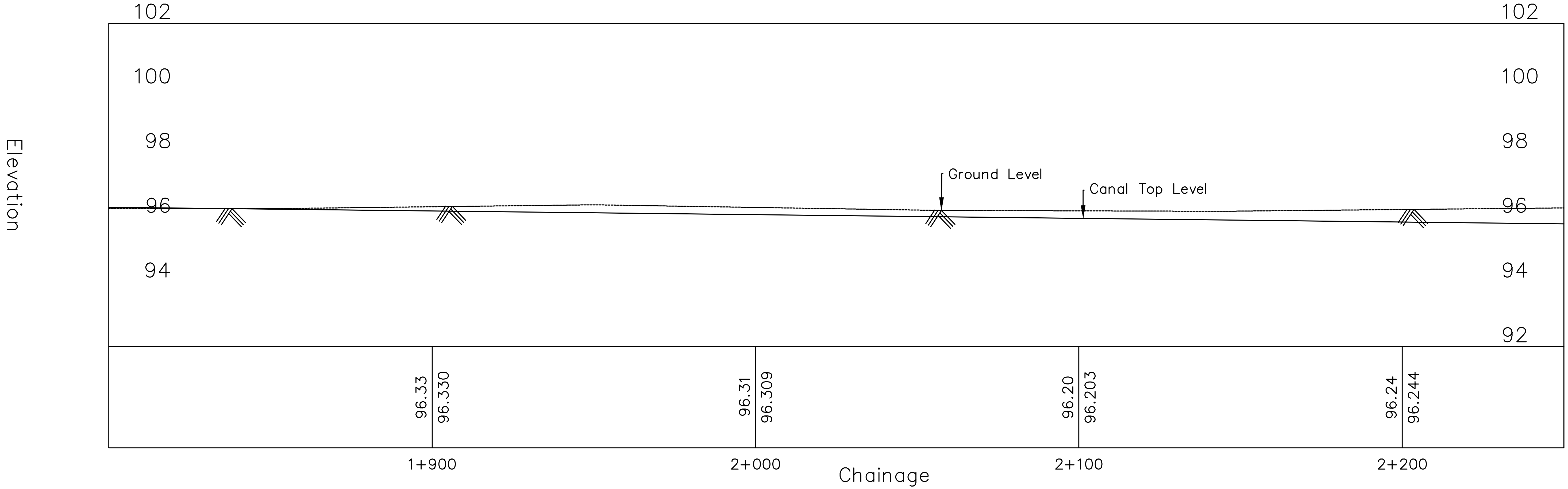
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:1250 DrawingNo.8	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



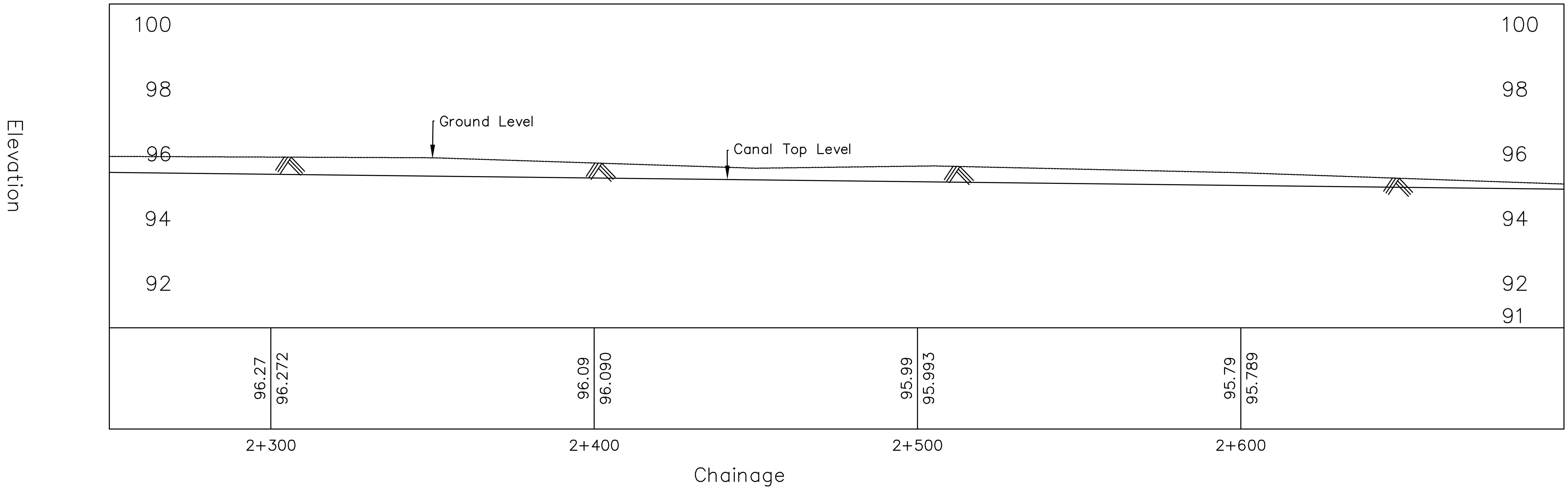
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Siddhartha Kandel Sundar Mani Bhattrai	Shital Upadhea Sudip Adhikari Suwaj Aryal	Scale 1:1250 DrawingNo.9	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel — Shital Upadhea Siddhartha Kandel — Sudip Adhikari Sundar Mani Bhattrai — Suwaj Aryal	Scale 1:1250 DrawingNo.10	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



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PULCHOWK, LALITPUR

CANAL LONGITUDINAL PROFILE

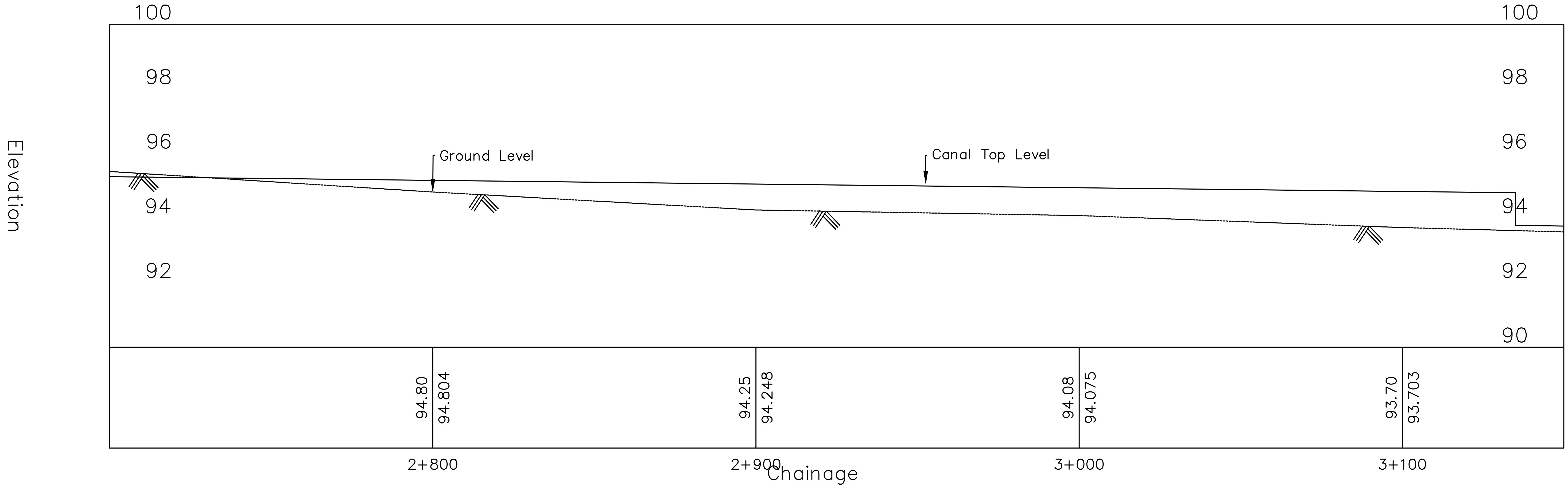
PREPARED BY:
Saugat Paudel
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:1250

DrawingNo.11

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



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CANAL LONGITUDINAL PROFILE

PREPARED BY:
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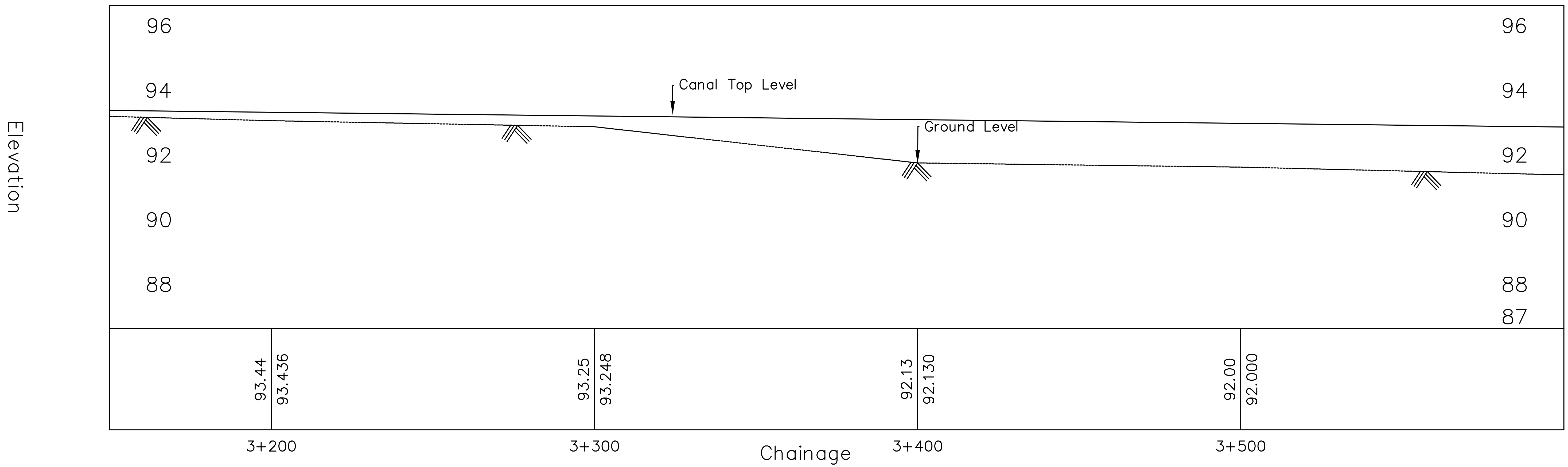
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:1250

DrawingNo.12

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Siddhartha Kandel Sundar Mani Bhattrai Shital Upadhea Sudip Adhikari Suwaj Aryal	Scale 1:1250 DrawingNo.13	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

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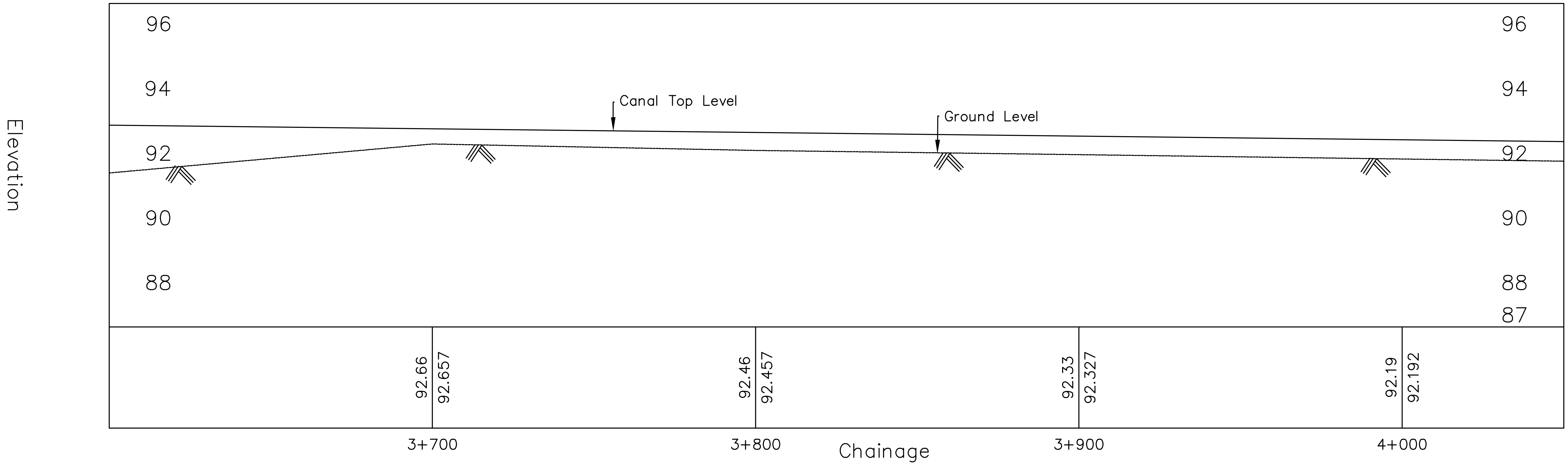
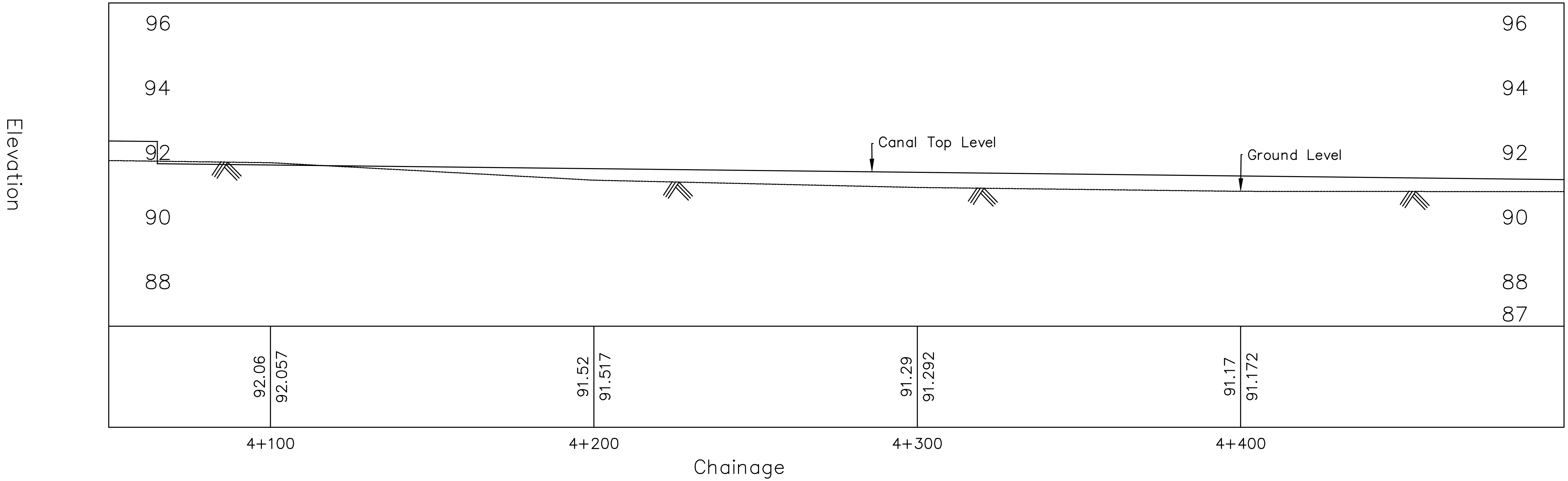


TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:1250 DrawingNo.14	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



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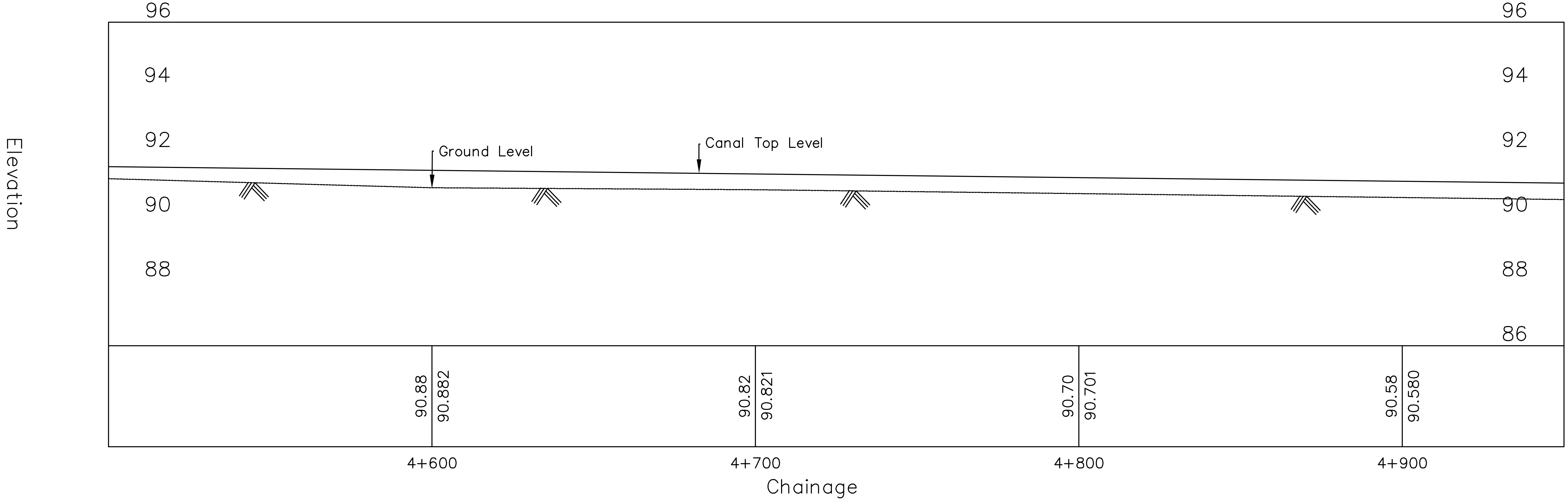

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PULCHOWK, LALITPUR

CANAL LONGITUDINAL PROFILE

PREPARED BY:
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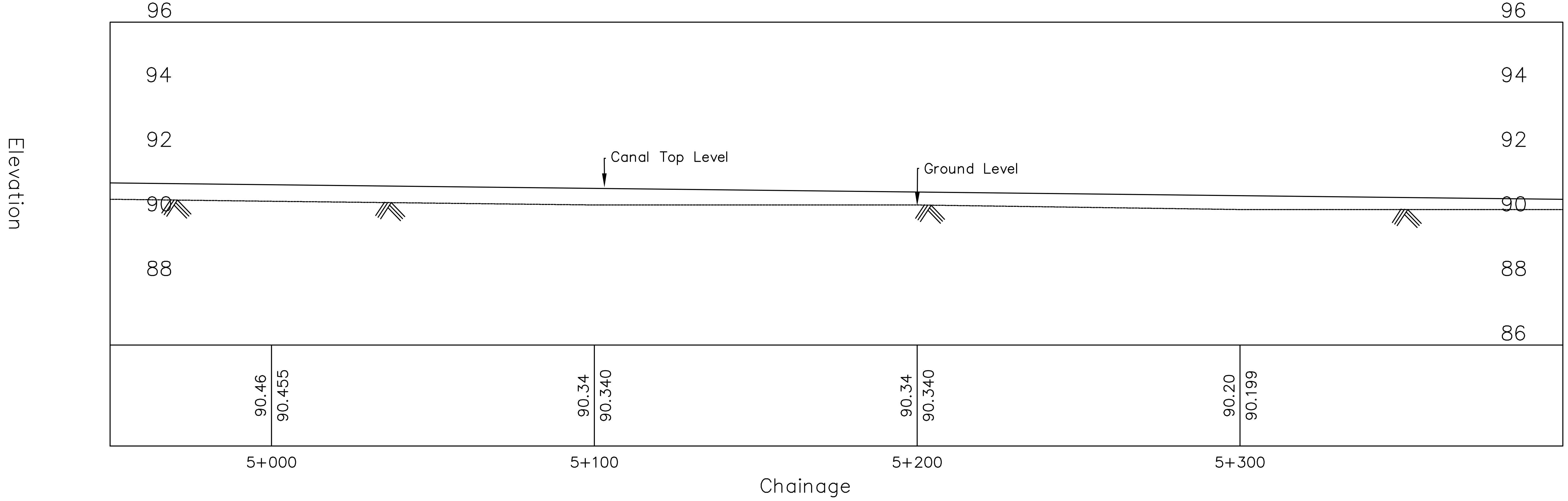
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:1250

DrawingNo.16

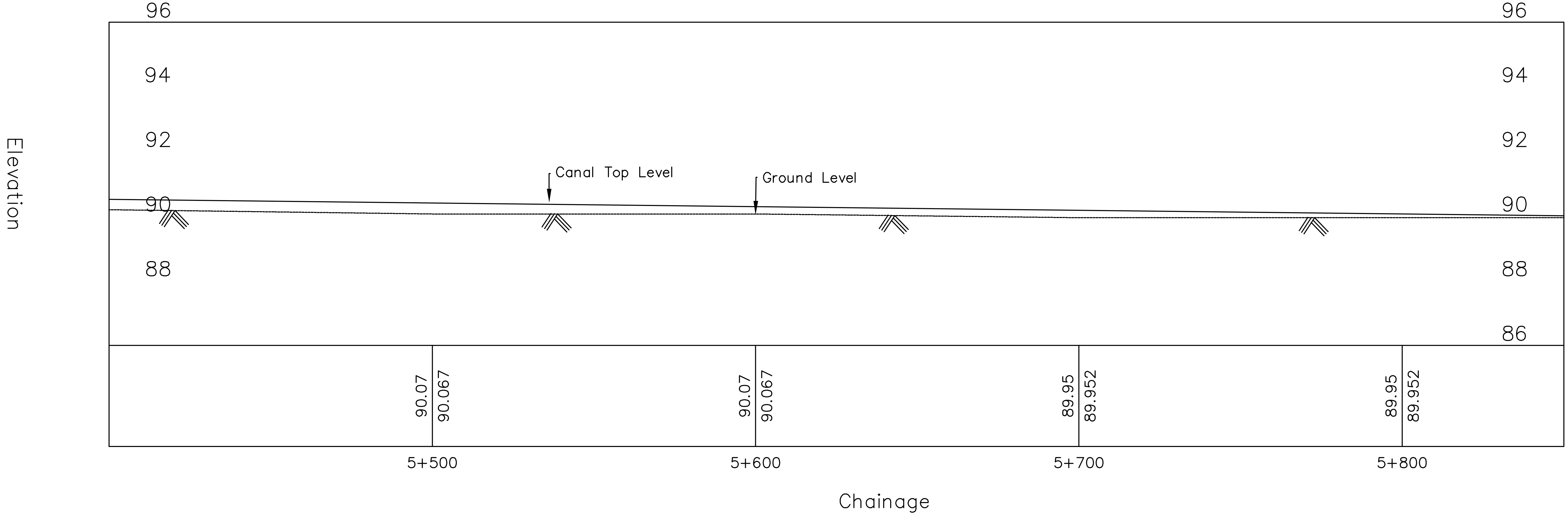
PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



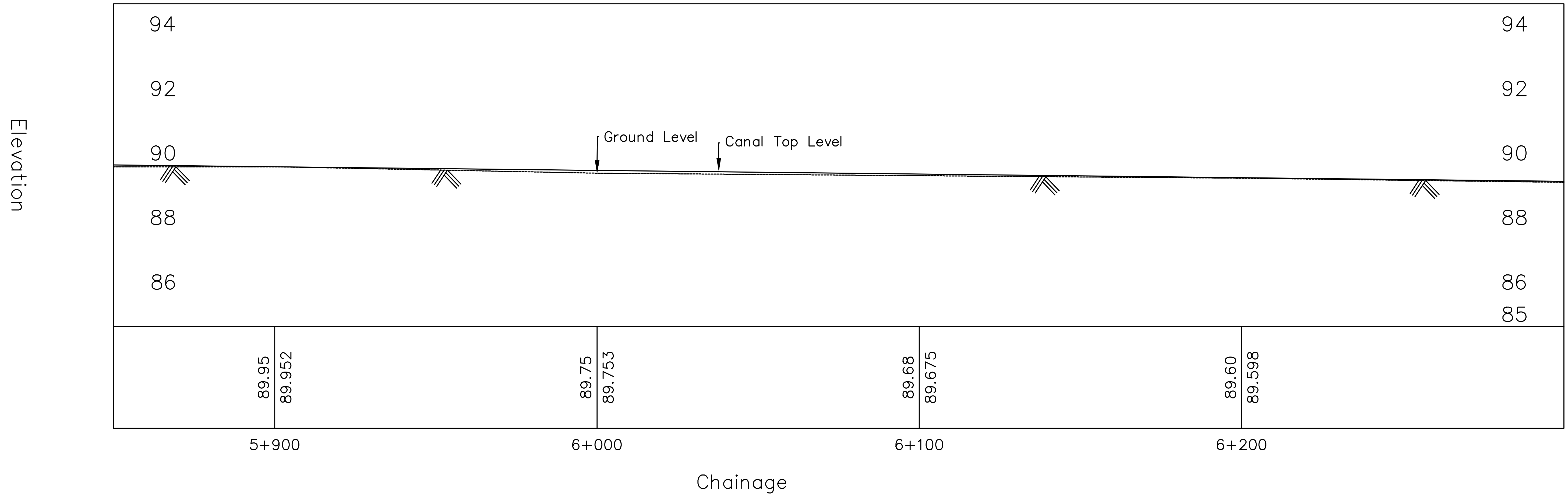
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel Siddhartha Kandel Sundar Mani Bhattrai Shital Upadhea Sudip Adhikari Suwaj Aryal	Scale 1:1250 DrawingNo.17	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL LONGITUDINAL PROFILE	PREPARED BY: Saugat Paudel — Shital Upadhea Siddhartha Kandel — Sudip Adhikari Sundar Mani Bhattrai — Suwaj Aryal	Scale 1:1250 DrawingNo.18	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

CANAL LONGITUDNAL PROFILE



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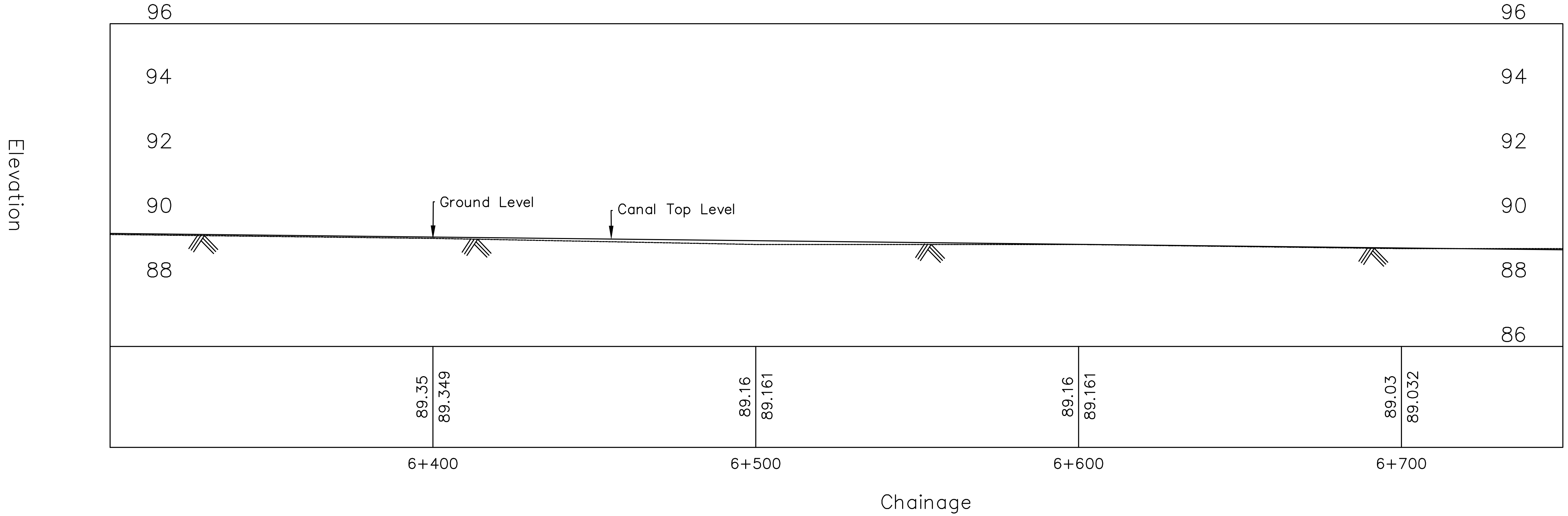
CANAL LONGITUDNAL PROFILE



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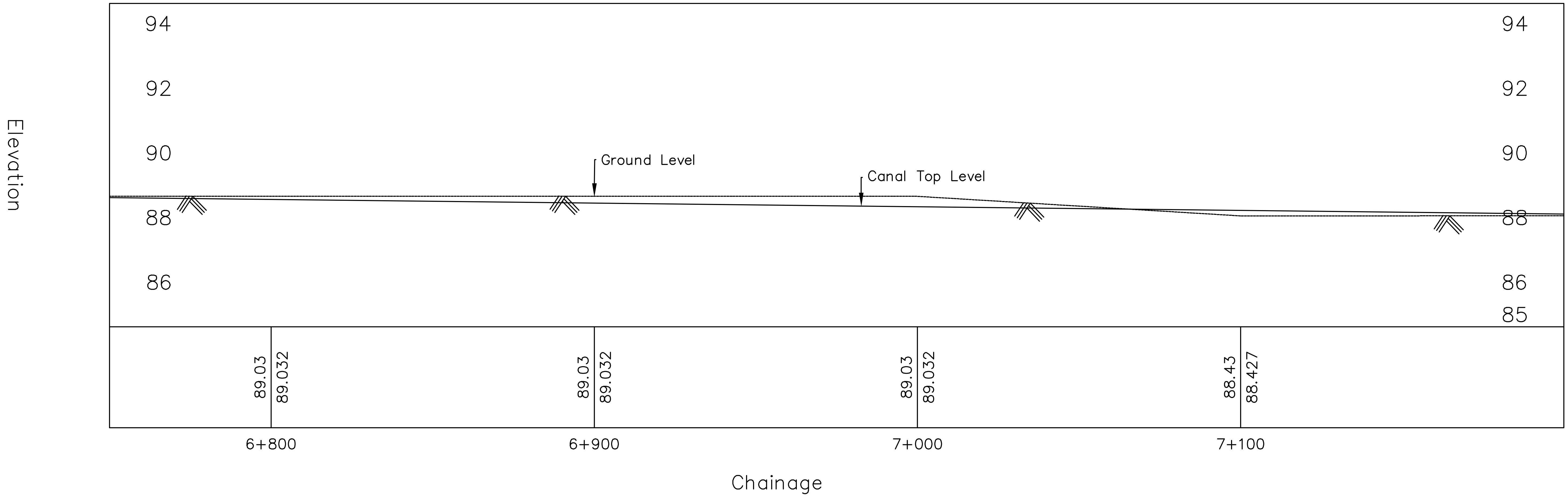
CANAL LONGITUDNAL PROFILE



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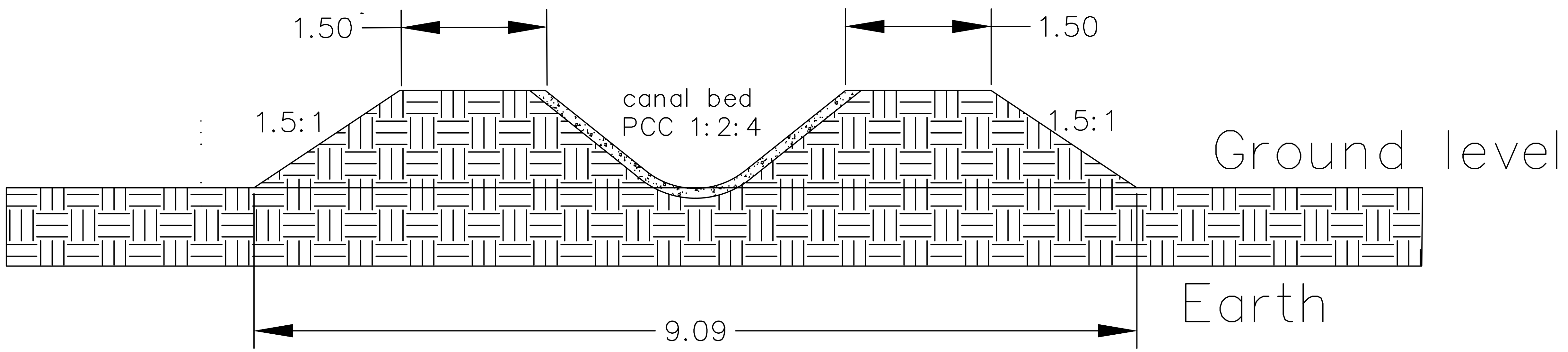
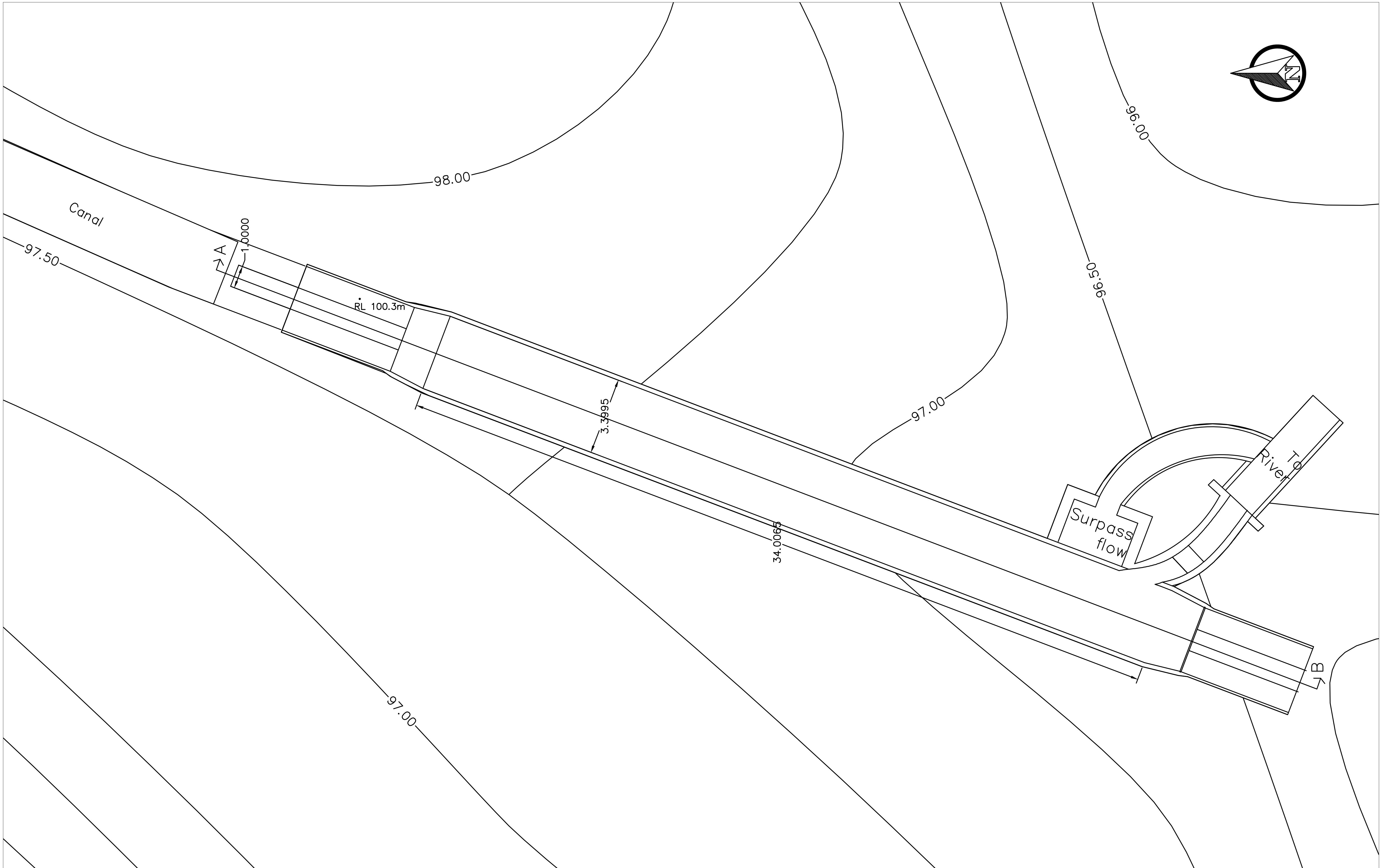


TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	CANAL SECTION	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:50 DrawingNo.22	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



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PULCHOWK CAMPUS
PULCHOWK, LALITPUR

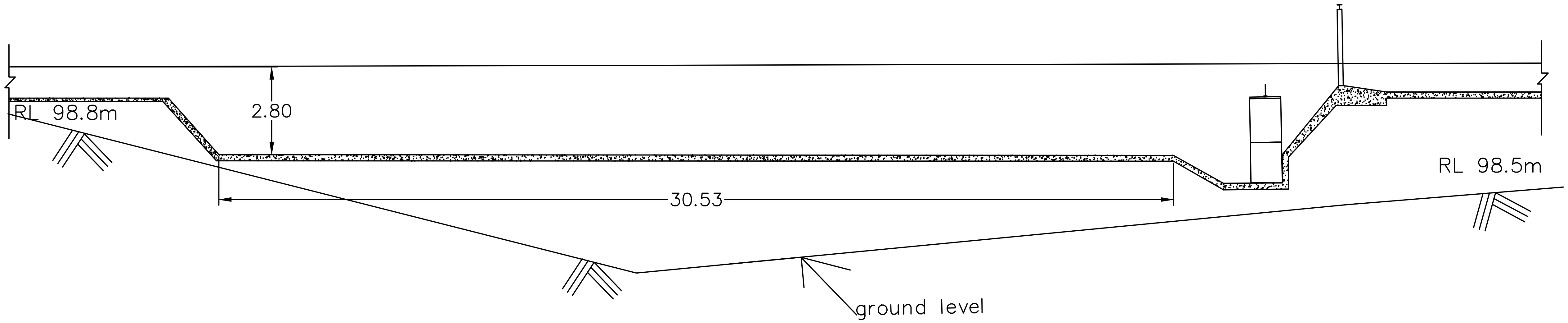
SETTLING BASIN PLAN

PREPARED BY:
Saugat Paudel
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

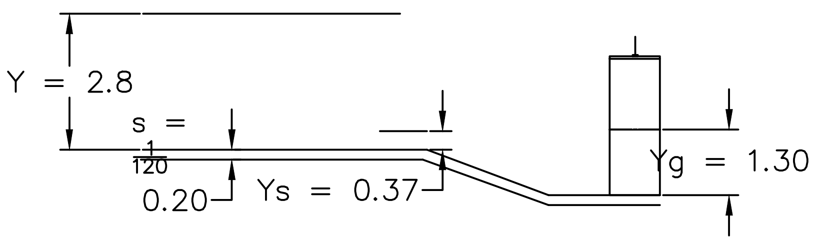
Scale 1:150

DrawingNo.23

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

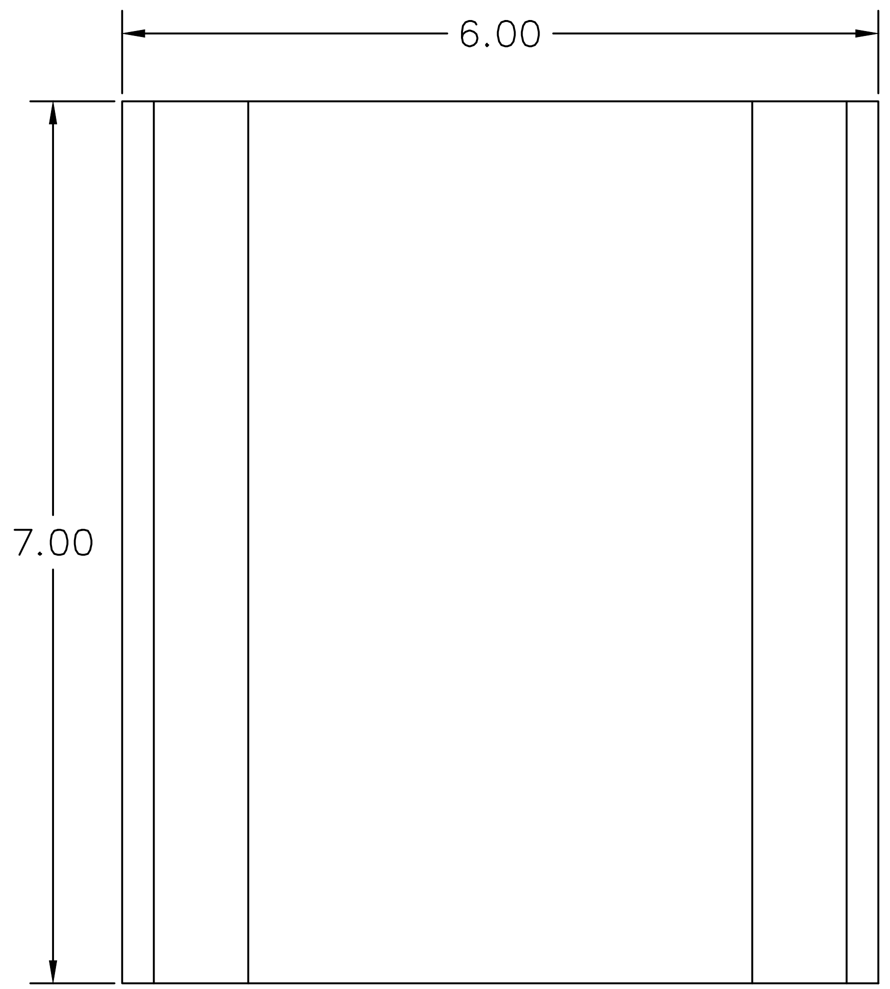


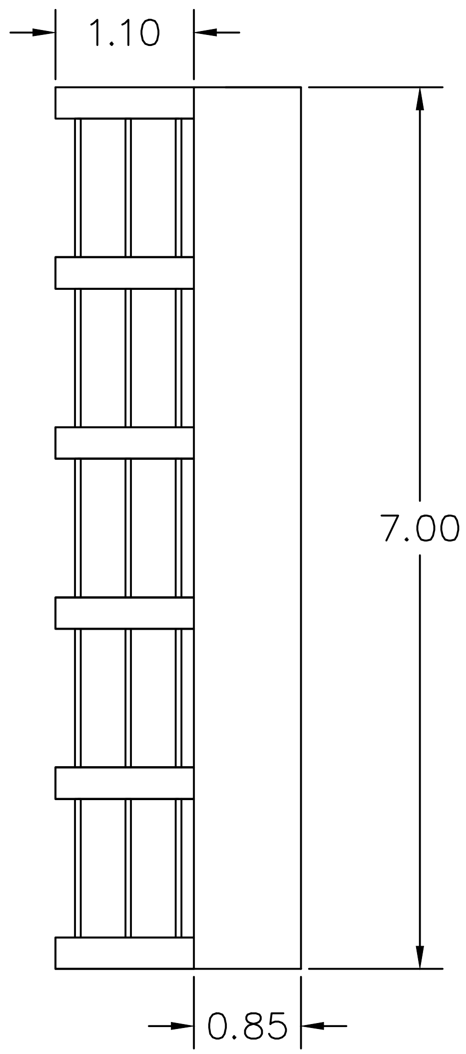
SECTION AT A—B

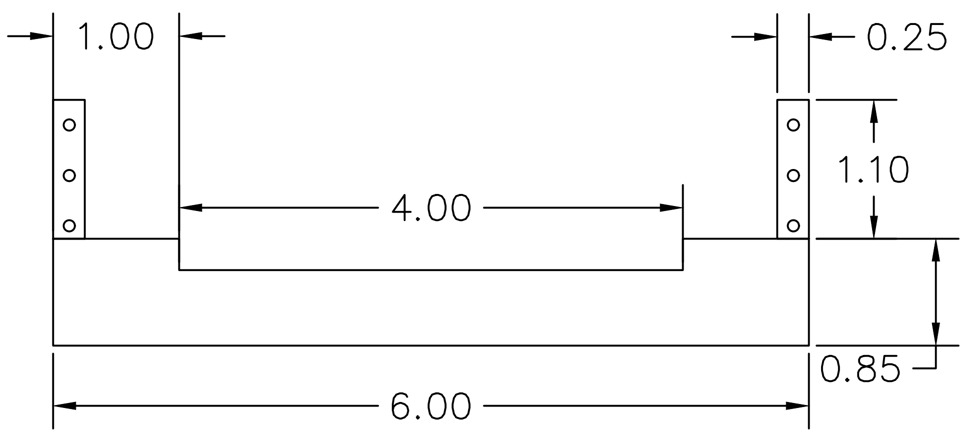


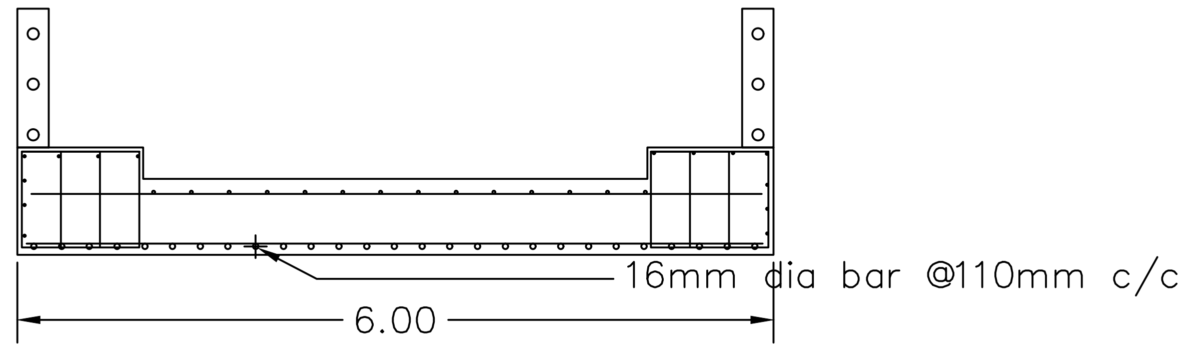
ENLARGED SECTION
AT GATE

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	SETTLING BASIN SECTION	PREPARED BY: Saugat Paudel Siddhartha Kandel Sundar Mani Bhattrai Shital Upadhea Sudip Adhikari Suwaj Aryal	Scale 1:150 DrawingNo.24	PRE—FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT









reiforcement detail

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

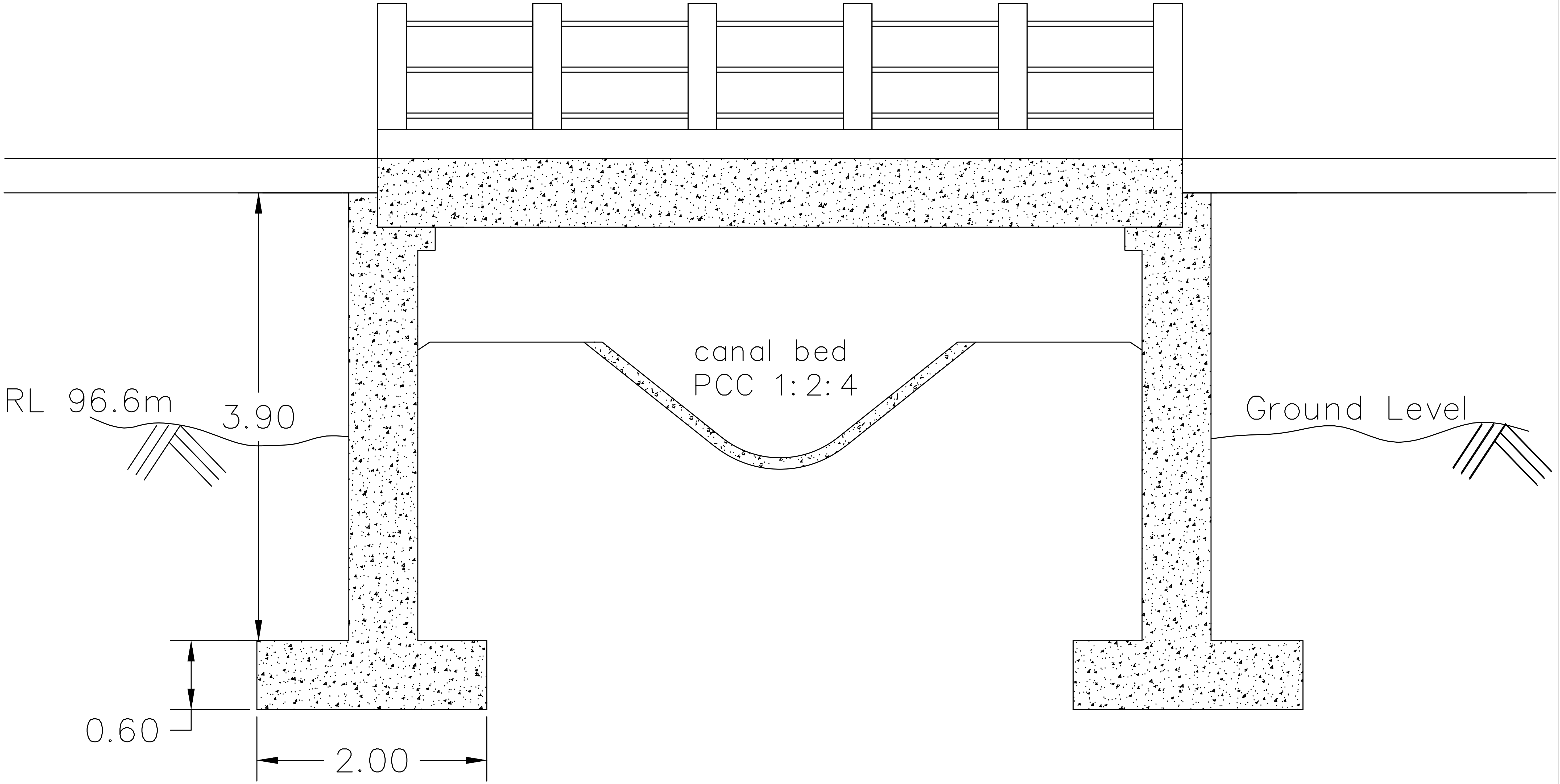
Bridge Deck
Plan And Section

PREPARED BY:
Saugat Paudel
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:25

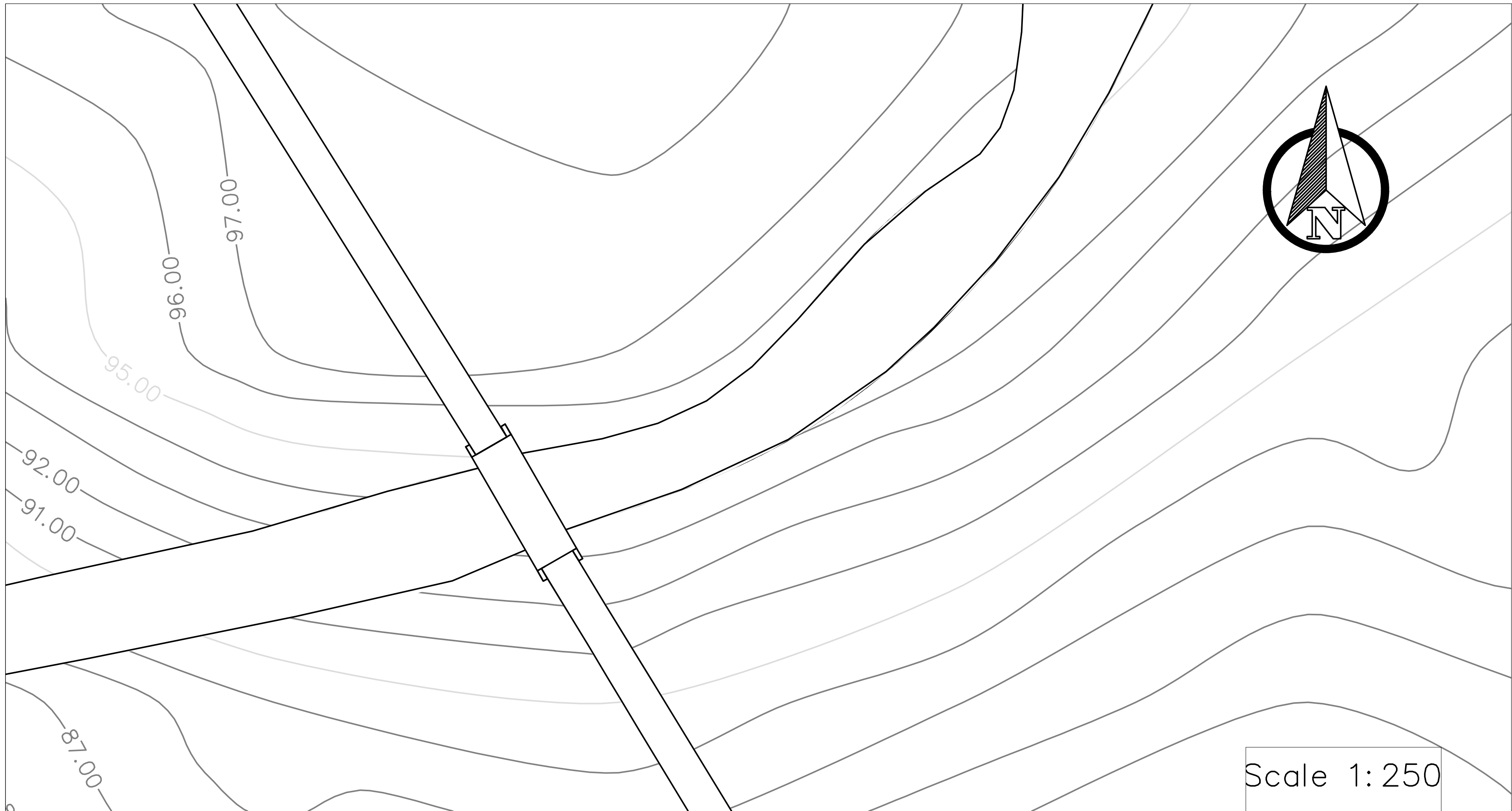
Drawing No. 25

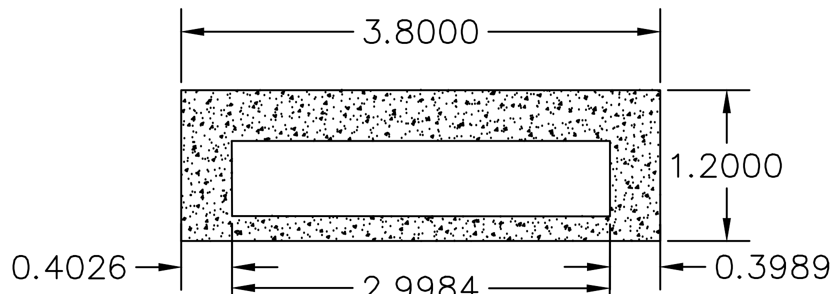
PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



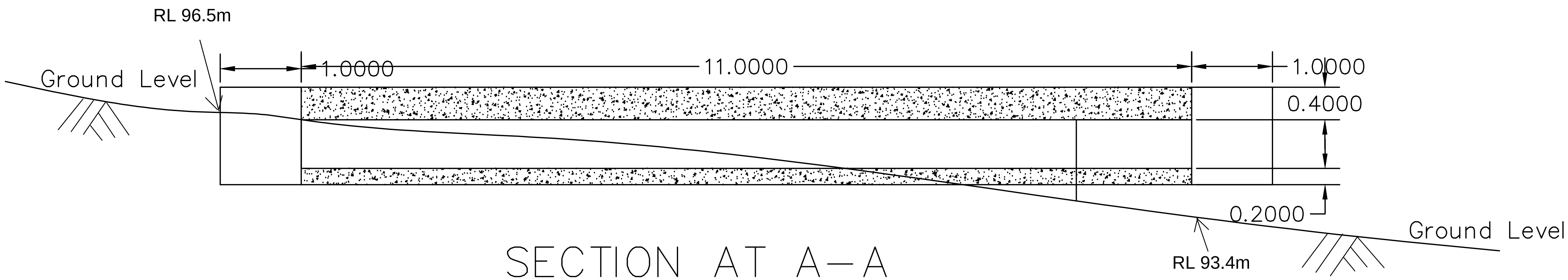
SECTION OF ABUTMENT

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	Bridge Abutment Section	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:35 Drawing No. 26	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT





SECTION AT B–B



SECTION AT A–A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	Culvert Plan and Section	PREPARED BY: Saugat Paudel — Shital Upadhea Siddhartha Kandel — Sudip Adhikari Sundar Mani Bhattrai — Suwaj Aryal	Scale 1:60 Drawing No. 27	PRE–FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

DROP PLAN

PREPARED BY:
Saugat Paudel
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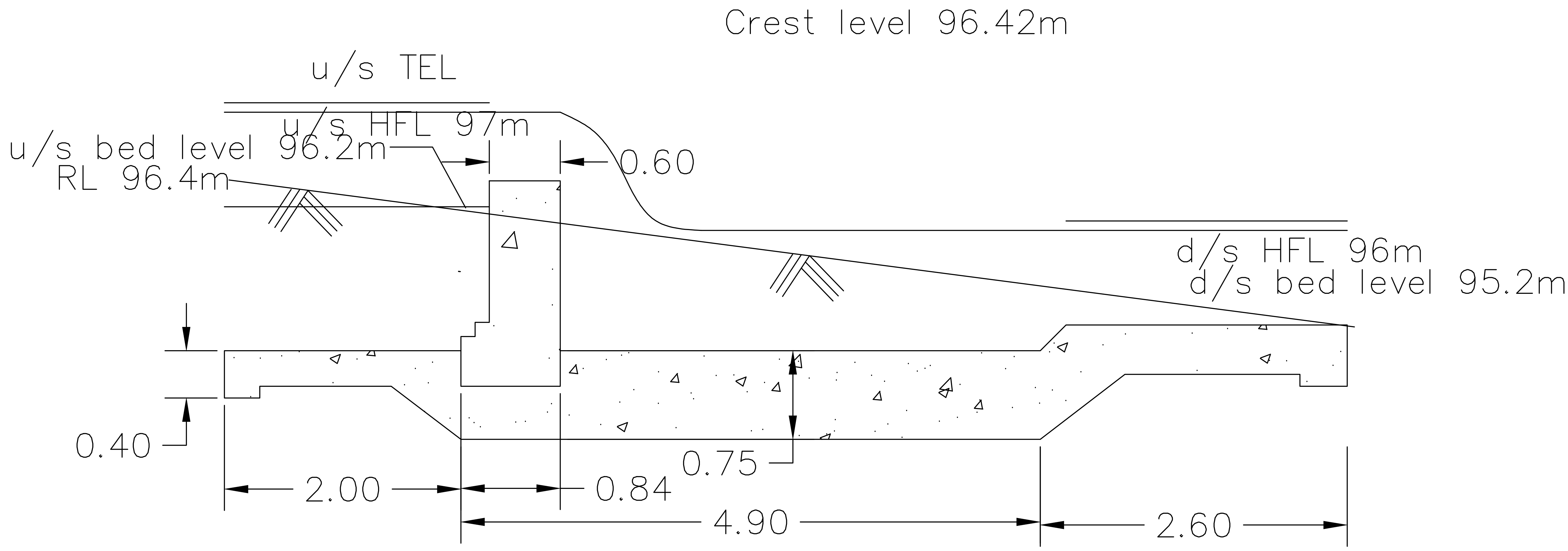
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

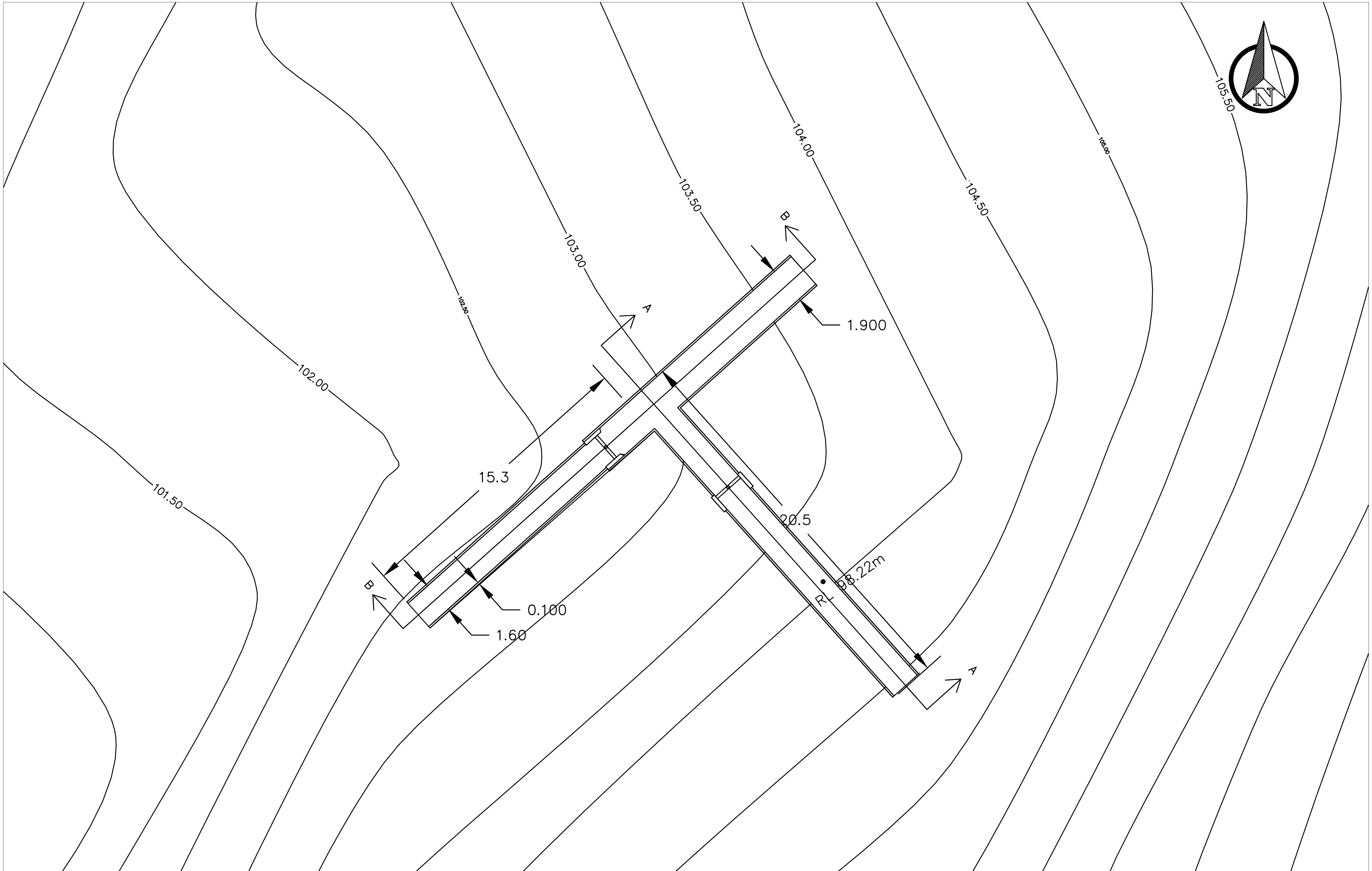
Scale 1:150
DrawingNo.28

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



All dimensions are in m

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	DROP SECTION AT A–A	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:35 DrawingNo.29	PRE–FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



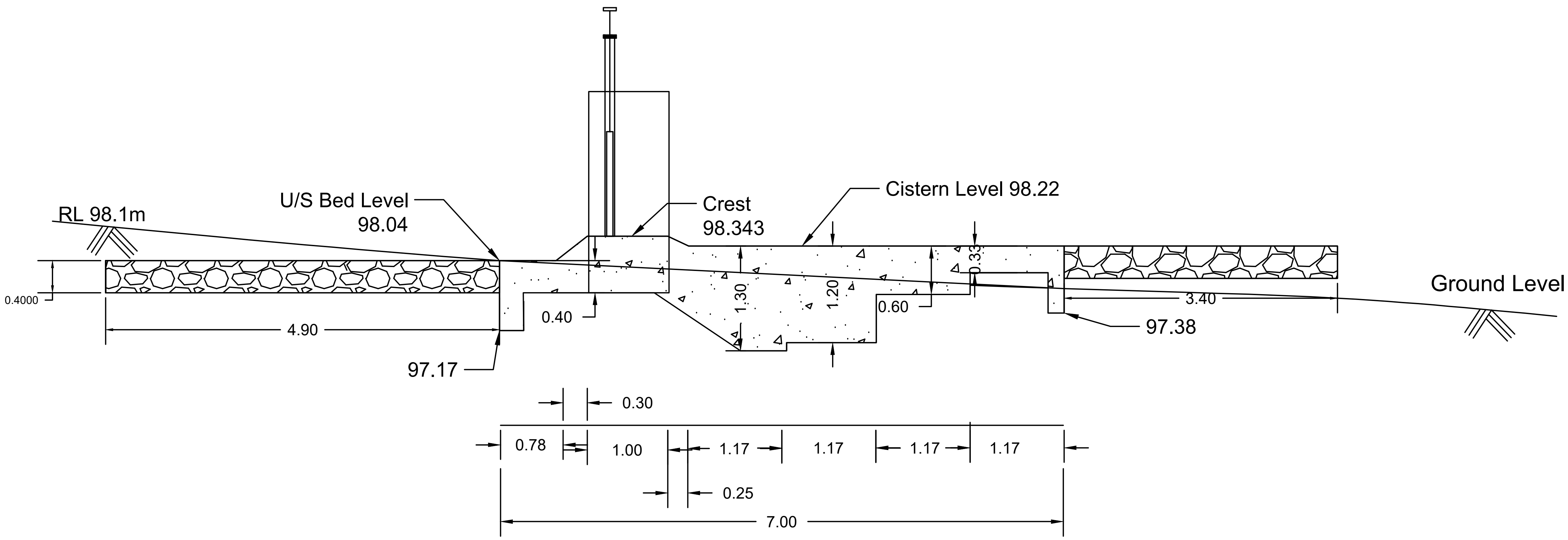
TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
PULCHOWK, LALITPUR

HEAD AND CROSS REGULATOR PLAN

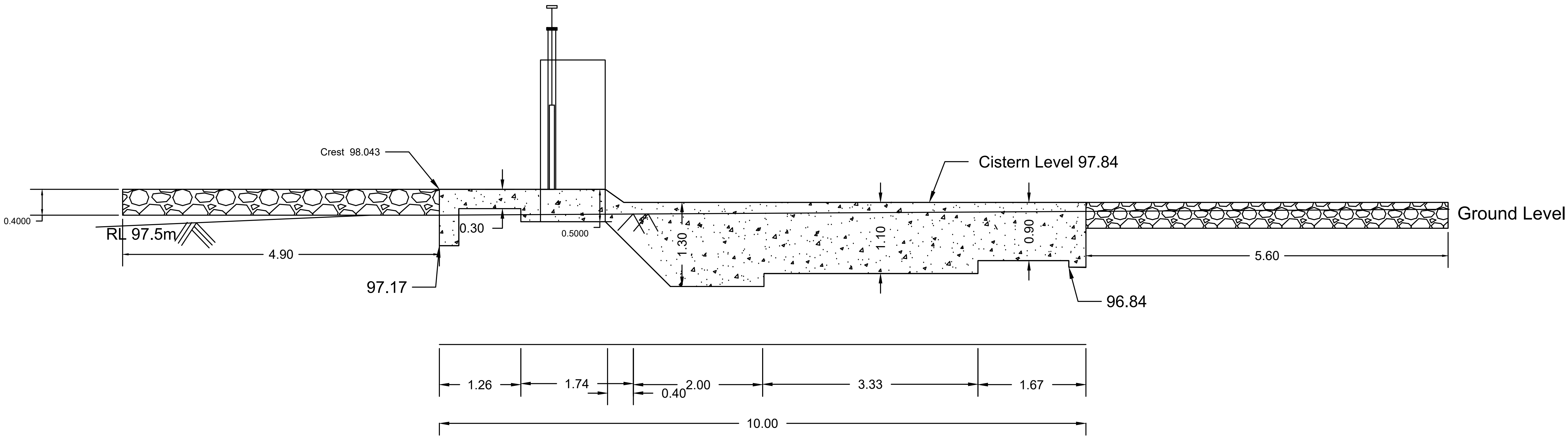
PREPARED BY:
Saugat Paudel
Siddhartha Kandel
Sundar Mani Bhattrai
Shital Upadhea
Sudip Adhikari
Suwaj Aryal

Scale 1:85
DrawingNo.30

PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT



HEAD REGULATOR SECTION AT A-A



CROSS REGULATOR SECTION AT B-B

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS PULCHOWK, LALITPUR	HEAD REGULATOR AND CROSS REGULATOR	PREPARED BY: Saugat Paudel Shital Upadhea Siddhartha Kandel Sudip Adhikari Sundar Mani Bhattrai Suwaj Aryal	Scale 1:60 Drawing No.31	PRE-FEASIBILITY STUDY OF BHUTENI IRRIGATION PROJECT

Appendix H:
Cost Estimation and Economic Analysis

I Economic Analysis

I.1. Crop Revenue Without Project

Without project, it was found approximately 300 ha of land cultivate the monsoon paddy and no other crop planting was possible.

The total benefit is shown in table below:

Table I.1: Crop Revenue Without Project

Crops	**Monsoon Paddy**	**Vegetables Winter**	**Pulses**
Production (t/ha)	**2**	**8**	**3**
Price (Rs/t)	25000	35000	65000
Value (Rs/ha)	50000	280000	195000
By product (Rs/ ha)	30000		
A) Gross value of Production (Rs/ ha)	**80000**	**280000**	**195000**
Seed (kg/ha)	60	0.8	21
Price (Rs/Kg)	50	50000	70
Value (Rs/ha)	3000	40000	1470
Organic Manure (t/ha)	4.2	7	1.4
Price (Rs/t)	2500	2500	2500
Value (Rs/ha)	10500	17500	3500
Chemical Fertilizer			
a) Urea (Kg/ha)	113.28	61.95	61.95
Price (Rs/Kg)	20	20	20
Value (Rs/ha)	2265.6	1239	1239
b) DAP (Kg/ha)	0	32.5	32.5
Price (Rs/Kg)	50	50	50
Value (Rs/ha)	0	1625	1625
c) Potash (Kg/ha)	0	25	25
Price (Rs/ha)	40	40	40
Value (Rs/ha)	0	1000	1000
Pesticides (Rs/ha)	1400	3000	200
a) Labor md/ha	104	120	64
Price (Rs/md)	400	400	400
Value (Rs/ha)	41600	48000	25600

b) Bullock ad/ha	25	25	25
Price Rs/ ad	500	500	500
Value (Rs/ha)	12500	12500	12500
B) Total Cost of Production (Rs/ha)	**71265.6**	**124864**	**47134**
C) Benefit per ha (A-B) Rs/ha	8734.4	155136	147866
D) Cultivated Area (ha)	300	0	0
E) Benefit Rs	2620320	0	0
E) Total Benefit Rs.	**2620320**		

I.2. Crop Revenue with Project

With project, we go for 3 types of crops. For each type of crops the command area is varied according to the amount of water we have.

The revenue with project is shown below:

Table I.2 : Crop Revenue with Project

Crops	**Monsoon Paddy**	**Vegetables Winter**	**Pulses**
Production (t/ha)	2	8	3
Price (Rs/t)	25000	35000	65000
Value (Rs/ha)	50000	280000	195000
By product (Rs/ ha)	30000		
A) Gross value of Production (Rs/ ha)	**80000**	**280000**	**195000**
Seed (kg/ha)	60	0.8	21
Price (Rs/Kg)	50	50000	70
Value (Rs/ha)	3000	40000	1470
Organic Manure (t/ha)	4.2	7	1.4
Price (Rs/t)	2500	2500	2500
Value (Rs/ha)	10500	17500	3500
Chemical Fertilizer			
a) Urea (Kg/ha)	113.28	61.95	61.95
Price (Rs/Kg)	20	20	20
Value (Rs/ha)	2265.6	1239	1239
b) DAP (Kg/ha)	0	32.5	32.5
Price (Rs/Kg)	50	50	50
Value (Rs/ha)	0	1625	1625

c) Potash (Kg/ha)	0	25	25
Price (Rs/ha)	40	40	40
Value (Rs/ha)	0	1000	1000
Pesticides (Rs/ha)	1400	3000	200
a) Labor md/ha	104	120	64
Price (Rs/md)	400	400	400
Value (Rs/ha)	41600	48000	25600
b) Bullock ad/ha	25	25	25
Price Rs/ ad	500	500	500
Value (Rs/ha)	12500	12500	12500
B) Total Cost of Production (Rs/ha)	**71265.6**	**124864**	**47134**
C) Benefit per ha (A-B) Rs/ha	8734.4	155136	147866
D) Cultivated Area (ha)	725	300	150
E) Benefit Rs	6332440	46540800	22179900
E) Total Benefit Rs.	**75053140**		

Thus, total benefit = 75053140 – 2620320 = Rs. 72432820 per year

I.3. Material estimates of structures:

Material volume was estimated using AutoCAD Drawing.

Table I.3 : Quantity Estimation

S N.	Description of work	Nos	L	H	B	Qty	Unit
			m	m	m		
1	Drop						
1.1	Earth Excavation		Area = 12.98 m2		1.9	20.76	m3
1.3	PCC M15						
	Floor		Area = 6.6450 m2		1.6	10.63	m3
	Canal		11	1.308	0.2	2.8776	m3
1.4	RCC in sheet pile		1% of volume of PCC in pile				
		=	0.01* 0.192*1.46		0.0028 m3	21.98	kg
2	Settling Basin						

2.2	PCC M15						
	Floor		30	2.8	2.7	226.8	m3
	Side walls	4	2.8	2.7	0.2	6.048	m3
	For side spillway and floor					7.5	m3
2.3	Gate						
	1 m height gate for flushing			0.16	m3	1256	kg
	gate for canal flow			0.12	m3	942	kg

	Weir							
SN	Description of work	No	Length (m)	Width(m)	Height/Depth (m)	Qty	Unit	Remarks
1	Weir (PCC)	1	17	area=43.699		742.883	m^3	M25
	sheet pile(upstream)	1	17	0.08	2.73	3.7128	m^3	steel
	sheet	1	17	0.08	3.683	5.0088	m^3	steel
	pier	3	3	1.5	7.343	99.130	m^3	RCC
	abutment	1	3	1	7.343	22.029	m^3	RCC
	pier foundation	4	3.5	3	1	42	m^3	RCC
	divide wall with					271.94	m^3	
	excavation. for	1	area=13.	1.5		20.085	m^3	
	exca. for pier	4	3.5	3	1	42	m^3	
	excavation for other works					1884.75	m^3	
	gate	3	4.67	0.01	0.7	0.098	m^3	steel
	spindle		area=	0.02	3.53	0.070	m^3	steel
2	Protection work							
	u/s protection work							
	concrete block		4.28	17	0.5	36.38	m^3	
	gravel	1	4.28	17	0.5	36.38	m^3	
	Launching apron	1	4.1	17	1.5	104.55	m^3	
	d/s protection							
	concrete block		5.9	17	0.5	50.15	m^3	
	gravel	1	5.9	17	0.5	50.15	m^3	
	Launching apron	1	6	17	1.5	153	m^3	

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3	Under sluice (PCC)	1	2.5	area=50.201		125.5	m^3	M25
	sheet pile(upstream)	1	2.5	0.08	5.355	1.071	m^3	steel
	sheet pile(downstream)	1	2.5	0.08	6.483	1.2966	m^3	steel
	gate	1	2.5	0.01	1.4	0.035	m^3	steel
	spindle	1	area=	0.02	3.53	0.0706	m^3	steel
	excavation for floor					671.576	m^3	
4	Protection works							
	u/s protection work							
	concrete block		8.6	2.5	0.5	10.75	m^3	
	gravel	1	8.6	2.5	0.5	10.75	m^3	
	Launching apron	1	8.1	2.5	2.3	46.575	m^3	
	d/s protection							
	concrete block		10.22	2.5	0.5	12.775	m^3	
	gravel	1	10.22	2.5	0.5	12.775	m^3	
	Launching apron	1	10	2.5	1.5	37.5	m^3	
5	Shear wall		126.8	0.5	3.5	221.9	m^3	

Cross Regulator

sn	Description	nos	area	width	quantity	unit
	launching apron u/s	1	1.96	1.8	3.528	m3
	PCC	1	9.089	1.9	17.2691	m3
	launching apron d/s	1	2.24	1.9	4.256	m3
	gate	1	0.12	0.009	8.478	kg
	pier	2	2.5	0.2	1	m3
	Excavation				12.307	m3

Head regulator

Description	nos	area	width	quantity	unit
launching apron u/s	1	1.8	1.96	3.528	m3
PCC	1	5.5343	1.8	9.96174	m3

launching apron d/s	1	1.8	1.36	2.448	m3
gate	1	0.12	0.009	8.478	kg
pier	2	2.5	0.2	1	m3
Excavation				10.456	m3

Canal

Description	nos	area	width	quantity	unit
Earthwork in excavation					
Cut volume	1			13338.7	m3
Canal lining		18799.92	0.1	1879.99	m3
Canal head regulator					
excavation		53.12	3.2	169.984	m3
M25 concrete				125.939	m3
Parapet wall				1.494	m3
Iron work				8.8862	m3

Bridges

Description	nos	area	width	quantity	unit
Excavation		9.382	6	56.292	m3
Filling		6.37	6	38.22	m3
RCC (abutment)				41.58	m3
RCC(slab)		4.2	6	25.2	m3

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Table I.4 : Economic Analysis for Canal Lining

Canal design using Lacey's Regime Theory		
Assume Lacey's silt factor(f) =1.5 Canal side slope (H: V) =1.5:1 [Table 3.4.8, M8] for alluvial soil		
	$(f^2Q)^{1/6}/140$	0.56 m/s
Area (A) =	Q/V	3.42 m^2
Perimeter (P) =	$4.75(Q)^{1/2}$	6.56m
Applying the equation for area and perimeter we get,		
Bottom width (b)=	5.57m	
Hydraulic depth (y)=	0.55m	
Free board=	0.5m	[3.5.5,
Top width (B) =	8.72	
Area of Cross section =	8.48m^2	
Hydraulic Radius(A/P) =	0.52131m	
Check For Hydraulic	$5V^2/(2f)$	
=	.522m	(Ok)
Bed Slope (S) =	$f^{5/3}/(3340*Q^{1/6})$	
=	1:3246	

Area of cross section of lined canal	1.255m^2
Area of cross section of unlined canal	7.51m^2
Area ratio	5.98
Volume of cut Ratio	5.98
Cost of excavation for unlined canal	5.98*cost of excavation of lined canal
	5.98*cut volume of lined canal*unit rate
	Rs.37989506.8
Cost for lined canal	Rs.25993100.73
Therefore, lined canal is chosen	

I.4. Cost Estimation

Table I.5:Unit Rate Analysis for PCC (1:1:2) for RCC

SN	Materials	Quantity	Unit	Rate (Rs.)	Standard for	Total (Rs.)	Remark
A. Materials (assuming 50% loss in dry wt.)							
1	Cement (OPC)	3.6	bags	825	per bag	2970	
2	Sand	0.125	cum	700	per cum	87.5	
3	Aggregates	0.25	cum	1400	per cum	350	
4	reinforcement	78.5	kg	95	per kg	7457.5	
5	binding Wire	0.785	kg	100	per kg	78.5	
6	Water	90	liters	1	per liters	90	
			Subtotal A			11033.5	
B. Manpower							
1	Skilled	0.8	nos.	920		736	
2	Unskilled	7	nos.	650		4550	
			Subtotal B			5286	
			Total (A + B)			16319.5	
C. Hire of tools and plants @ 3% of unskilled labor		3% of 36736.875				136.5	
			Total (A+B+C)			16456	
D. Contractors overhead and profit @ 15% of total (A+B+C)		15% of 101478.1613				2468.4	
			Grand total (A+B+C+D)			18924.4	

Table I.6 : Unit Rate Analysis for PCC (1:2:4) for RCC

SN	Materials	Qty	Unit	Rate (Rs.)	Standard for	Total (Rs.)	Remarks
1	Cement (OPC)	2.05	bags	825	per bag	1691.25	
2	Sand	0.142	cum	700	per cum	99.4	
3	Aggregates	0.285	cum	1400	per cum	399	
4	reinforcement	78.5	kg	95	per kg	7457.5	
5	binding wire	0.785	kg	100	per kg	78.5	
6	Water	51.42	liters	1	per liters	51.42	
			Subtotal A			9777.07	
B.	Manpower						
1	Skilled	0.8	nos.	920		736	
2	Unskilled	7	nos.	650		4550	
			Subtotal B			5286	
			Total (A + B)			15063.07	
C. Hire of tools and plants @ 3% of unskilled labor						136.5	
			Total (A+B+C)			15199.57	
D. Contractors overhead and profit @ 15% of total (A+B+C)		15% of	15199.6			2279.9355	
			Grand total (A+B+C+D)			17479.506	

Table I.7 : Unit Rate Analysis for PCC (1:2:4)

SN N	Materials	Qty	Unit	Rate (Rs.)	Standard for	Total (Rs.)
1	Cement (OPC)	2.05	bags	825	per bag	1691.25
2	Sand	0.142	cum	700	per cum	99.4
3	Aggregates	0.285	cum	1400	per cum	399
4	Water	51.42	liters	1	per liters	51.42
			Subtotal A			2241.07
B.	Manpower					
1	Skilled	1	nos.	920		920
2	Unskilled	4	nos.	650		2600
			Subtotal B			3520
			Total (A + B)			5761.07
C. Hire of tools and plants @ 3% of unskilled labor						78
			Total (A+B+C)			5839.07
D. Contractors overhead and profit @ 15% of total (A+B+C)		15%	5839.07			875.8605
			Grand total (A+B+C+D)			6714.9305

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Table I.8 : Unit Rate Analysis for Steel Work

SN	Material	Quant	Unit	Rate(	Standar	Total(
A. Materials (assuming 0.5% loss)						
1	steel	1050	kg	95	per kg	99750
2	Binding	10.5	kg	100	per kg	1050
			Subtotal A			100800
B. Manpower						
1	Skilled	12	nos.	920		11040
2	Unskille	12	nos.	650		7800
			Subtotal B			18840
			Total (A + B)			119640
C. Hire of tools and plants @ 3% of unskilled labor						234
			Total (A+B+C)			119874
D. Contractors overhead and profit @ 15% of total (A+B+C)		15%	119874			17981.
			Grand total			137855
					per m3	108216
					per kg	137.85

I.5. Cost Estimate

Table I.9: Cost Estimate of Drop Structure

	Drop structure (No. = 3)				
Sn	Description of work	unit	quantity	unit rate	Amount (Rs)
1	Earth Excavation	m3	20.768	473.8	9839.8784
2	PCC (m15)				
	floor	m3	10.632	6714.93	71393.1411
	canal	m3	2.8776	6714.93	19322.884
				Total	100555.903
			Grand Total		301667.71
	Settling Basin				
Sn	Description of work	unit	quantity	unit rate	Amount (Rs)
1	Earth Excavation	m3	484.92	473.8	229755.096
2	PCC M15				
a	Floor	m3	226.8	6714.93	1522946.24
b	Side walls	m3	6.048	6714.93	40611.8997
c	For side spillway and floor towards	m3	7.5	6714.93	50361.9788
3	Gate				
a	1 m height gate for flushing	kg	1256	6714.93	8433952.71
b	gate for canal flow	kg	942	6714.93	6325464.53
				Total	16603092.5

Table I.10 : Cost Estimate of Head Work

Sn	Description of work	unit	quantity	unit rate	Amount (Rs)
1	Weir (PCC)	m3	742.883	18924.4	14058615
a	sheet pile(upstream)	m3	3.7128	1082163	4017853.04
b	sheet pile(downstream)	m3	5.00888	1082163	5420422.25
a	pier	m3	99.13	17479.5	1732743.38
b	abutment	m3	22.029	17479.5	385056.027
a	pier foundation	m3	42	17479.5	734139.231
b	divide wall	m3	271.945	17479.5	4753464.12
a	excavation for abutment	m3	20.085	473.8	9516.273
a	exca. for pier foundation	m3	42	473.8	19899.6

b	excavation for other works	m3	1884.75	473.8	892994.55
a	gate	m3	0.09807	1082163	106127.679
b	spindle	m3	0.0706	1082163	76400.6746
2	Protection work				
	u/s protection work				
a	concrete block	m3	36.38	6714.93	244289.172
b	gravel	m3	36.38	1400	50932
c	Launching apron	m3	104.55	473.8	49535.79
3	d/s protection				
a	concrete block	m3	50.15	18924.4	949058.66
b	gravel	m3	50.15	1400	70210
c	Launching apron	m3	153	473.8	72491.4

Table I.11 : Cost Estimate of Under Sluice

4	Under sluice				
a	PCC	m3	125.5	18924.4	2375012.2
b	sheet pile(upstream)	m3	1.071	1082163	1158996.07
c	sheet pile(downstream)	m3	1.2966	1082163	1403131.94
d	gate	m3	0.035	1082163	37875.6886
e	spindle	m3	0.0706	1082163	76400.6746
f	excavation for floor	m3	671.576	473.8	318192.709
5	Protection works				
	u/s protection work				
a	concrete block	m3	10.75	18924.4	203437.3
b	gravel	m3	10.75	1400	15050
c	Launching apron	m3	46.575	473.8	22067.235
	d/s protection				
a	concrete block	m3	12.775	18924.4	241759.21
b	gravel	m3	12.775	1400	17885
c	Launching apron	m3	37.5	473.8	17767.5
6	Shear wall				
a	PCC	m3	221.9	18924.4	4199324.36

b	RCC	Kg	17419.15	137.85	2401229.83
				Total	46131878.59

Table I.12 : Cost Estimate of Distributary Cross Regulator

sn	Description	unit	quantity	unit rate	Amount
1	launching apron u/s	m3	3.528	473.8	1671.5664
2	PCC	m3	17.2691	18924.4	326807.356
3	launching apron d/s	m3	4.256	473.8	2016.4928
4	gate	kg	8.478	137.85	1168.6923
5	pier	m3	1	18924.4	18924.4
6	Excavation	m3	12.307	473.8	5831.0566
				Total	356419.564
			Grand Total		1782097.82

Table I.13: Cost Estimate of Distributary Head Regulator

sn	Description	unit	quantity	unit rate	Amount (Rs)
1	launching apron u/s	m3	3.528	473.8	1671.5664
2	PCC	m3	9.96174	18924.4	188519.952
3	launching apron d/s	m3	2.448	473.8	1159.8624
4	gate	kg	8.478	137.85	1168.6923
5	pier	m3	1	18924.4	18924.4
6	Excavation	m3	10.456	473.8	4954.0528
				Total	216398.526
			Grand Total		1081992.63

Table I.14 : Cost Estimate of Bridges and Culvert

	Bridges (3 nos.)				
sn	Description	unit	quantity	unit rate	Amount (Rs)
1	Excavation	m3	56.292	473.8	26671.1496
2	Filling	m3	38.22	473.8	18108.636
3	RCC (abutment)	m3	41.58	18924.4	786876.552
4	RCC (slab)	m3	25.2	18924.4	476894.88
				Total	1308551.22
			Grand total		3925653.65
	Culvert (7 nos)				

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sn	Description	Unit	Quantity	Unit rate (Rs)	Amount (Rs)
1	Excavation	m3	65.261	473.8	30920.6618
2	Filling	m3	2.169	473.8	1027.6722
3	PCC with RCC	m3	30.36	18924.4	574544.784
				Total	606493.118
			Grand Total		4245451.83

Table I.15 : Cost Estimation of Canal

S.N.	Description	Unit	Quantity	Unit Rate (Rs.)	Amount (Rs.)
1	Earthwork in excavation				
	Cut volume	m3	13338.7	473.8	6319876.06
2	Canal lining				
	(PCC 1:2:4) M15 concrete				
a	Inclined side wall	m3	2099.52	6714.93	14098130.9
b	Round bottom	m3	830.232	6714.93	5574950.18
3	Canal head regulator				
a.	Earthwork in excavation	m3	169.984	473.8	80538.4192
b.	M25 concrete work	m3	125.939	18924.4	2383322.66
c	Parapet wall	m3	1.494	137.85	205.9479
d	Iron for gate	m3	8.8862	1082163	9616312.67
				Total	38073336.8
4	Distributor canal	assume 50% of main canal			12996478.6
			Total		89143152.2
			Grand Total		163214987.2

I.6. B/C Ratio Calculation

Table I.16 : Project Data for B/C Calculation

Total cost of project		NRs	163214987.2
Economic cost	0.95*A	NRs	155054237.8
Net benefit		NRs	72432820
Maintenance cost	3% of A	NRs	4896449.616

Table I.17: B/C Analysis

S.N. (year)	Cash out flows NRs	PW of cost at 10%(NRs)	Benefit (NRs)	PW of benefit at 10%(NRs)
1	62021695.14	56383359	0	0
2	46516271.35	38443199	0	0
3	46516271.35	34948363	21729846	16325954.92
4	4896449.616	3344341	72432820	49472590.67
5	4896449.616	3040310	72432820	44975082.43
6	4896449.616	2763918.2	72432820	40886438.57
7	4896449.616	2512652.9	72432820	37169489.61
8	4896449.616	2284229.9	72432820	33790445.1
9	4896449.616	2076572.6	72432820	30718586.45
10	4896449.616	1887793.3	72432820	27925987.69
11	4896449.616	1716175.7	72432820	25387261.53
12	4896449.616	1560159.7	72432820	23079328.67
13	4896449.616	1418327	72432820	20981207.88
14	4896449.616	1289388.2	72432820	19073825.34
15	4896449.616	1172171.1	72432820	17339841.22
16	4896449.616	1065610.1	72432820	15763492.02
17	4896449.616	968736.45	72432820	14330447.29
18	4896449.616	880669.5	72432820	13027679.35
19	4896449.616	800608.64	72432820	11843344.87
20	4896449.616	727826.04	72432820	10766677.15
	Total	159284412		452857680.8
	B/C ratio	**2.84**		

Here, the value of B/C ratio is 2.84, which is greater than 1. Hence, the project is financially feasible.