TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

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

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

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CERTIFICATE

This certifies that the project work titled "Pre-feasibility Study of Manohara Irrigation Project" has been evaluated and found to be satisfactory in meeting the requirements for the academic program leading to the awarding of a Bachelor of Civil Engineering degree.

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ACKNOWLEDGEMENT

We would like to thank Prof. Dr. Prem Chandra Jha, our project supervisor and mentor, for his essential advice and assistance during the study of this irrigation project. His experience and skill in irrigation proved to be a fantastic advantage to our team, and we were able to accomplish our objectives with his continuous support and advice. We also appreciate the irrigation department officers' cooperation and support throughout the project. Their resources and experience were essential to our project's success. We also want to express our gratitude to all the people and groups that helped us better our project by offering insightful criticism and recommendations.

Lastly, we would want to express our gratitude to our teammates for their persistent efforts in making this project a success. We are grateful to have had the chance to collaborate with such a brilliant and motivated group of people; their commitment and hard work have been helpful. We feel grateful towards Prof. Dr. Prem Chandra Jha once more, as well as everyone else who helped with this project. Our achievement has been attributed to your assistance and direction.

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ABSTRACT

To promote crop output and plant growth, agricultural land is artificially supplied with water using irrigation systems. The medium-sized Manohara Irrigation Project has been chosen for the Pre-Feasibility study. There is an abundance of land available for food development, but just a single crop is cultivated in some areas because of an inadequate irrigation infrastructure. This project's primary motive is to assess the viability of upscaling the commanding zone with three distinct crop kinds in a year-round manner.

The Department of Hydrology and Meteorology's metrological datasets were utilized for this study to analyze monthly flows and floods. The discharge measurement was utilized to compute the monthly flows using the MIP approach, and the irrigation water need was determined using these data. The structures were created to satisfy the specifications.

Situated in Changunarayan Municipality of Kathmandu Valley is the Manohara River that contributes to the Bagmati Basin. Its catchment area is 30.24 square kilometers. On 4th December, the river's discharge measured was 875 liters per second. The maximum required canal discharge for a command area of 45.8 ha is 0.241 m^3/s. A weir will be used in the project to pond 1.6 meters of water for agriculture. The canal is intended to receive water supply from the frontal kind of intake. A trapezoidal canal is intended for that. The purpose of the settling basin is to empty the canal from silt. Sarda fall type drops are intended for locations where the bed level abruptly changes throughout the canal in accordance with requirements. Distribution boxes are designed to divide and regulate flow at diversion points. A flood discharge of 133 m^3/s can be safely passed by the headwork structure during a 100-year return period. An under-sluice will discharge around ten percentage of the water downstream to ensure the water rights are safe. Every effort has been made to obtain the most trustworthy data while considering every aspect. The environmental assessment stages up to the project's actual pre-feasibility stage have been completed. It refers to the process of Scoping, Screening, and preparing Terms of Reference.

The irrigation system is expected to yield net revenue of 25.37 million Rupees a year, while the project's total anticipated cost is 176.1 million Rupees. The total revenue over a 20-year period is determined by the annual revenue generated. The obtained B/C ratio is 1.116. Project is therefore feasible.

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NOTATIONS

π = 3.14

ρ = density of material

n = manning's coefficient

γ = specific weight

K_a = active pressure

K_p = passive pressure

ϕ = friction angle

g = acceleration due to gravity

° = degree

°C = degree Celsius

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 Model
DHM	Department of Hydrology and Meteorology
d/s	downstream
DWRI	Department of Water Resources and Irrigation
d	diameter
e_d	Actual vapor pressure in millibar
EIA	Environmental Impact Assessment
FDC	Flow Duration Curve
FSL	Full Supply Level
GoN	Government of Nepal
GNSS	Global Navigation Satellite System
HFL	High Flood Level
HGL	Hydraulic Gradient Line
Hl	Head Loss
Ht	Height
IWR	Irrigation Water Requirement
K_c	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
MT	Metric Ton
MWI	Monsoon Wetness Index
NPR	Nepalese Rupees
NRs	Nepalese Rupees
PCC	Plain Cement Concrete
PDSP	Planning and Design Strengthening Project
Q	Discharge

RCC	Reinforced Cement Concrete
R_H	Relative Humidity
S	Longitudinal Slope
SB	Settling Basin
TEL	Total Energy Line
u/s	upstream
WECS	Water Energy Commission Secretariat

1. INTRODUCTION

1.1 Background

Our proposed Manohara River Irrigation Project is based on Direct Irrigation Method and is a medium scaled Irrigation project located in Changunarayan Municipality of Bhaktapur district. Its source is Manohara River which is a perennial river. Water accumulated by the weir is transferred for irrigation on the right side of the weir. The project shall cover approximately 45.8 ha. area of Changunarayan. Based on all the conditions encompassing various weather conditions, temperature, soil, and its characteristics, there is a persistent demand for water that can be supplied from the neighboring river with the help of canals.

The temperature condition of the site is medium, i.e., neither too cold in the winter nor too hot in the summer. Several factors, including crop water demands, trends and structures of cropping, and economic and financial gains, must be evaluated to judge how well the Manohara Irrigation project will perform. To ensure effectiveness and sustainability, examination of feasibility, financial and economic viability are necessary.

Identification of project's shortcomings by evaluation of every detail encompassing the project, improvement of performance and effectiveness of the project by continuous hearings from people/farmers directly affected by it and consideration of all valuable critiques is of utmost necessity too.

Fig 1.1: Kathmandu district and Location of catchment area in Nepal

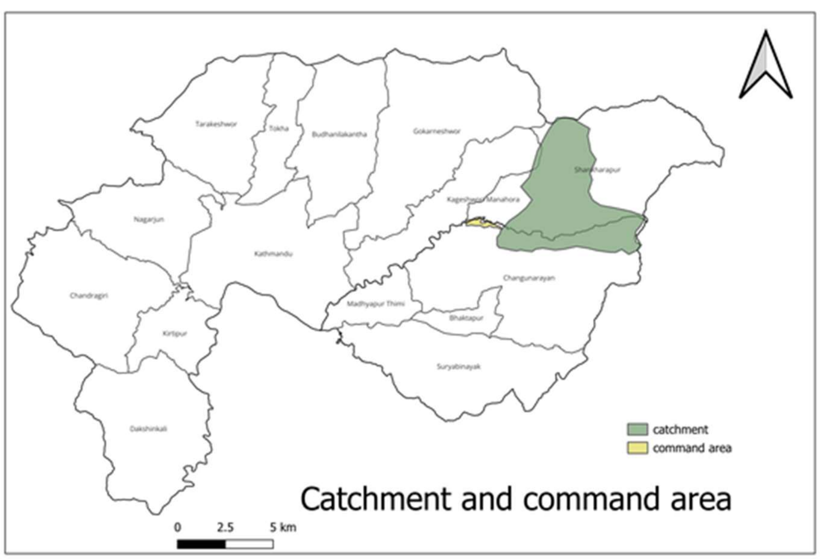


Fig 1.2: Location of Catchment and Commanded area in Kathmandu district

1.2 Need of Study

Improving agricultural productivity, efficiency, and practices in the project region is the primary goal of this study. The main channel alignment, headworks, offtake canal alignment, irrigation water distribution system, and all other significant and minor details of the works involved, from the process of accumulating water from a river by weir to the process of distributing it to the field, will all be suggested by this study, which will also aid in understanding the common traditional irrigation practices in the area.

1.3 Objectives

Major and direct objectives of the project are:

- To evaluate the requirements of irrigation and canal discharge.
- To design the intake and conveyance structure and headworks to control flood.
- To investigate the economic feasibility of the project.

Other Objectives/Goals:

- Increase agricultural productivity.
- Providing reliable and sustainable water supply in the region to increase crop yield.
- Owing to seasonal rainfall pattern in the region, enable multiple cropping seasons.
- Economic growth and supporting job opportunities in agricultural field by maximizing farmer's output.
- Upliftment of overall living standards of agriculture dependent population.

1.4 Other Scopes

- Data collection on cropping pattern, crop water requirements.
- Data collection on hydrological aspects like HFL, debris accumulations.
- Test and analysis of collected soil samples.
- Interpretation of the data collected from the primary on-field and secondary sources.
- Investigation of groundwater options.
- Assessing the project's financial and commercial viability.
- Examining the water demands of various crops to determine which are the most practical and profitable.

- Outlining and analyzing the catchment area.
- Elaborating the needs of IEE study.
- Preparation for TOR of the same.

1.5 Approach and Methodology

While it was not strictly required, we were instructed to prepare surveys and communicate with farmers and residents in a kind and grounded manner while maintaining a professional demeanor. The recommendations and viewpoints expressed by the beneficiaries throughout various meetings have received the consideration they deserve. Multiple meetings with the recipients were held to gather information. In addition, the farmers were apprised of the protocols and norms pertaining to project approval as well as their respective roles in project implementation. The design, drawing, rate analysis, cost estimate, and other elements of the Manohara Irrigation Project pre-feasibility study have been prepared in accordance with industry standards, design manuals, and customs. For computation of water requirement of different crops, Penman's equation was used in Microsoft Excel for computing ET_0 and CROPWAT has also been used to form a basis of comparison between values.

1.5.1 Desk Studies

- Topographic Studies: Topographic surveys were conducted to understand the land contours and slopes. QGIS, Google Earth surveys and Topographic maps were used in this process. Delineation of catchment and command area was done to understand the extent of irrigation and the magnitude of floods.
- Hydrological and Hydrogeological Studies: It includes assessment of water availability and potential sources. Hydrogeological studies to understand groundwater conditions. Flood estimation was done using various methods such as WECS/DHM 1990, DHM 2004, MHSP 1997, PCJ 1996, Rational and Modified Dicken's Formula. Finally, Rational Method was adopted for the design flood.
- Climatic Studies: Analysis of historical weather patterns and climate projections. Identification of potential climate-related risks (e.g., droughts, floods).
- Land Use and Land Cover Analysis: Examination of current land use patterns. Assessment of potential impacts on land cover due to the irrigation project.

- Environmental Impact Assessment (EIA): Evaluation of potential environmental impacts. Identification of mitigation measures to minimize negative effects.
- Socio-economic Studies: Assessment of the local population and demographics. Analysis of the social and economic impacts of the irrigation project.
- Legal and Regulatory Compliance: Review of existing laws and regulations related to water use and land use. Identification of permits and approvals required for the project. Legitimate permissions were granted by the Department of Civil Engineering, Pulchowk Campus and Changunarayan Municipality.
- Water Quality Studies: Analysis of water quality in potential water sources. Identification of potential contaminants and water treatment requirements.
- Cost-Benefit Analysis: Economic assessment of the project, including construction and operational costs. Estimation of potential economic benefits and returns on investment.
- Risk Assessment: Analysis of potential risks and uncertainties. Development of risk management strategies.
- Stakeholder Analysis: Identification of key stakeholders (local communities, government agencies, NGOs). Assessment of their interests, concerns, and potential contributions.
- Institutional and Organizational Framework: Analysis of existing institutions involved in water management. Assessment of their capacity and potential roles in the project.
- Historical Water Use: Review of historical water use in the project area. Identification of potential conflicts and synergies with existing water users.
- Technology and Innovation Review: Evaluation of available irrigation technologies. Assessment of innovative solutions for water efficiency.

1.5.2 Site Investigation

a. Walkthrough Survey:

All project members took the walkthrough with a helpful local man of old age. Major things in the surroundings were noted and rough sketch of topography, contours were drawn too. Special considerations were given to things said by locals during discussion before walkthrough. Walk along strips of main and branch canal alignment were also taken. Mobile application software for GNSS (SW Maps) was used to tentatively fix the major and minor survey stations.

b. Agro-economic and social survey:

Questionnaires were prepared and randomly asked of some locals in a professional and friendly fashion. The main aim was to know about the sentiment regarding the project in the minds of people affected by it. Along with that, the sense of unity for social causes and general relationship of land users around the site was also of concern to us.

1.5.3 Office Work

A map of the headwork location was created using information from the topographical survey. The river's two banks, flood lines on either side, lines with spot elevations that represent the river's bed slopes, traverse lines, benchmarks, reference lines, or points that the current topo map is based on are all depicted on the drawn map.

1.5.4 Evaluation of finances and Estimation of cost

The unit cost approach was adopted since the provided data was not appropriate for an approximative calculation.

2. HYDROLOGICAL ANALYSIS

We used topographic maps and computer tools such as GIS (Geographic Information System), Google Earth Pro, etc. to delineate catchment area and commanded areas. Multiple results obtained were compared to get precise result and to use same for further computations.

2.1 Delineation of Catchment Area and Command Area

2.1.1 Catchment Area

A catchment area, also known as a watershed or drainage basin, refers to the area of land that collects and drains water into a specific river, lake, or other water body. It is defined by the topography of the land, with all the precipitation that falls within the boundaries of the catchment area eventually flowing into a common outlet point, such as a river mouth or a lake. The size and shape of a catchment area can vary, ranging from small, localized areas to large river basins that span multiple states or countries. Understanding the characteristics of a catchment area is crucial for studying and managing water resources, as it helps in predicting and managing water flow, flood risk, and water quality within a specific region.

Methods of catchment area delineation:

a. Topographic Map

The following procedures were followed to connect all the points on the boundary of watershed on a topographic map. It will be much simpler to perform tasks by visualizing terrain that depicts topographic map rather than following route by route.

- At the catchment's downstream or outflow point, a circle was drawn.
- The watercourse's high points on both banks were identified, and the path upstream to the watershed's headwaters was charted.
- Lines that intersected the contours at right angles were drawn, beginning with the circle created in the first stage.
- The line went on till it went down to the other side of the river and around the head of the watershed. It eventually contacted the circle that initiated the process. The watershed border was now clearly defined.

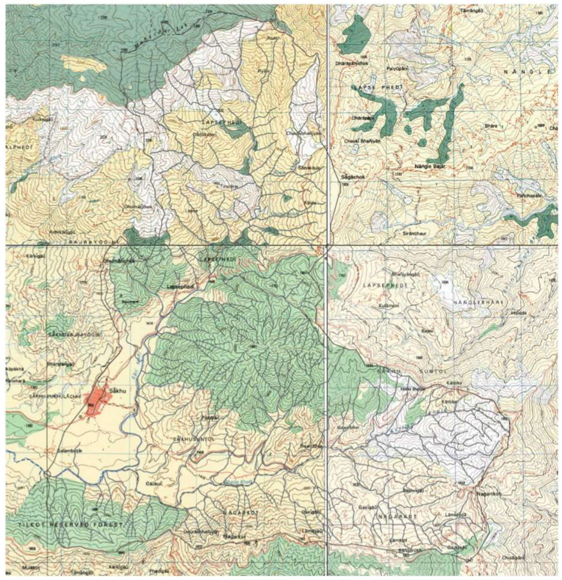


Fig 2.1: Catchment delineation using Topographic Maps

b. Google Earth

- ➢ A location marker at the outflow was placed.
- ➢ The Polygon option was used to draw the boundaries of the catchment region.
- ➢ After drawing the polygon over the ridge line of the watershed, the measured area was recorded and stored.
- ➢ After choosing the Path option, the longest channel length was determined.
- ➢ The mouse pointer hovered over the catchment area to determine the highest point of elevation.

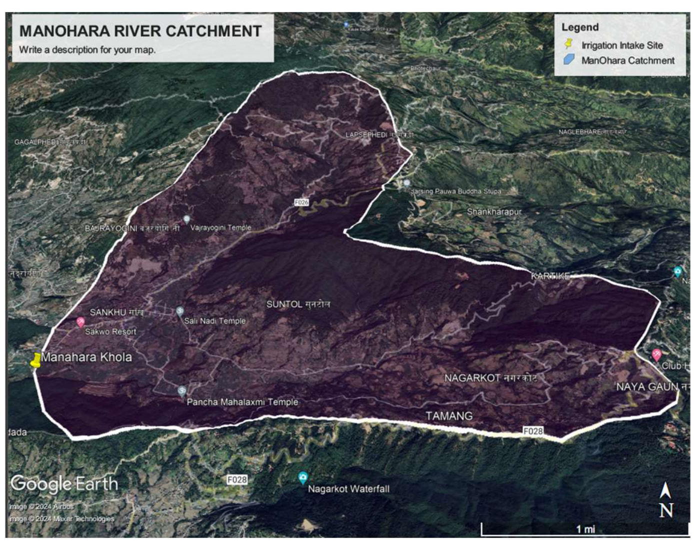


Fig 2.2: Catchment area delineated using Google Earth

c. GIS

The resultant catchment area obtained with this tool was like that of the other two tools.

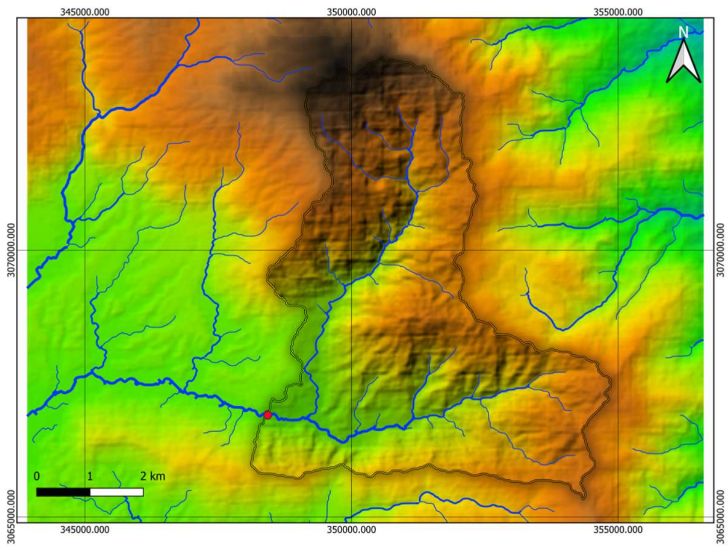


Fig 2.3: Catchment area delineated using QGIS

For further computations, we opted for the result obtained from QGIS, which was 30.24 km^2.

2.1.2 Commanded Area

In the context of irrigation systems, the command area refers to the area of land that is served by a particular irrigation system or project. It is the total land area that receives water from the irrigation system for agricultural purposes. The command area is typically defined by the physical boundaries of the irrigation infrastructure, such as canals, pipelines, or sprinkler systems, which deliver water to the fields. The size of the command area can vary depending on the capacity and reach of the irrigation system, as well as the availability of water resources. Proper management and distribution of water within the command area are essential to ensure efficient and sustainable irrigation practices.

Following steps were followed:

a. We requested the local farmers' assistance for the field survey and analysis.

b. Farmers' actual needs were also considered while fixing the command area.

c. Next, Google Earth was used to draw the boundaries of the commanded region.

Fig 2.4: Commanded Area delineated using Google Earth

The gross command area was found to be 52.4 ha using google earth pro. After examining both the commanded area and the truly viable irrigation area, 45.8 ha is determined to be the net commanded area.

2.2 Hydrometeorological Database

A hydro-meteorological database is a collection of data related to hydrology and meteorology. It includes several types of data such as rainfall, temperature, humidity, wind speed, river flow, groundwater levels, and other relevant parameters. These databases are typically maintained by meteorological and hydrological agencies, research institutions, or government bodies. Hydro-meteorological databases serve as valuable resources for studying and understanding weather patterns, climate change, water resources management, flood forecasting, and other related fields. They are used for analyzing historical trends, developing models and forecasts, and making informed decisions regarding water management and disaster preparedness. These databases often include long-term historical data as well as real-time or near real-time data collected from weather stations, river gauges, groundwater monitoring wells, and other measurement devices. The data is usually organized, quality-controlled, and stored in a structured format that allows for easy retrieval and analysis.

2.2.1 Representative rainfall stations

The hydro-meteorological stations close to our catchment region along with their Index No. are highlighted in table below:

Table 2.1: Rainfall Stations

Station	Index No.	Thiessen Area	Weightage (%)
Changunarayan	1059	0.088	0.291
Nagarkot	1043	6.422	21.242
Sankhu	1035	23.723	78.467
Total		30.233	100

As a result, rainfall stations — Sankhu, Nagarkot, and Changunarayan — are utilized to compute rainfall data as well as to compute floods and monthly flow when necessary.

2.2.2 Maximum daily rainfall and frequency analysis

The hydro-meteorological stations are used to determine the maximum amount of rainfall. The maximum rainfall for various return periods were determined by doing frequency analysis on rainfall data that was gathered from DHM. There are several ways to go about it, including the Gumbel approach, the Log Normal method, and the Log Pearson III method. By using various methods, we can achieve correlation between calculated and observed data; the one with the highest correlation is chosen. Using these techniques, the rainfall for various return periods is calculated using the best fit method based on correlation.

To calculate the flood for various return periods, the rainfall data acquired as shown in Table 2.2 are employed in both rational and empirical methods (e.g., PCJ 1996).

Table 2.2: Maximum rainfall for different return periods

Recurrence Intervals	Rainfall Stations			Mean Rainfall
	Changunarayan	Nagarkot	Sankhu	
2	89.239	72.638	75.619	79.165
10	119.597	103.536	113.966	112.366
25	133.120	120.669	132.418	128.736
50	142.659	134.120	145.899	140.893
100	151.803	148.205	159.165	153.058
200	160.723	163.020	172.420	165.388
1000	180.744	201.079	203.241	195.021

- The maximum correlation value for Changunarayan, or $r^2 = 0.997$, was discovered using the Log Pearson III method.
- The greatest correlation value for Sankhu was determined using the Log Normal technique, or $r^2 = 0.9973$.

- The greatest correlation value for Nagarkot was determined using the Log Normal technique, or $r^2 = 0.9904$.

The calculations involved are shown in **Appendix – A**.

2.2.3 Potential Evapotranspiration

The phrase "evapotranspiration" refers to the combination of evaporation and transpiration. In general, it refers to the losses brought about by transpiration and evaporation. Climate data are typically used to estimate evapotranspiration. Climate data that is necessary are listed in Table 2.4. The most popular techniques are:

a. Penman's Method

The Penman's equation for reference crop evapotranspiration is:

$$ET_0 = c[(w * R_n) + (1 - w) * f(u) * (e_a - e_d)]$$

where,

ET_0= evapotranspiration in mm/day

c = adjustment factor depending on maximum relative humidity (RH_{max}), total solar radiation (R_s), daytime wind speed (U_{day}), and the ratio of daytime wind to nighttime wind ($U_{day} - U_{nigh}$). The relation is shown in **Appendix – B**.

w = weighing factor depending on temperature and altitude

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

$f(u)$ = wind function

$(e_a - e_d)$ = vapor pressure deficit in millibar

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

T_{mean} = mean daily temperature in °C

RH_{mean} = mean relative humidity in %

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

n = actual daily sunshine in hours

e_d = actual vapor pressure in millibar

Table 2.3: DHM data for calculation of Penman's evapotranspiration

Month	Rhmean	Rhmax	Tmax	Tmin	U	n
Jan	76.412	79.609	12.727	2.822	2.460	5.944
Feb	75.094	75.791	14.640	4.231	2.624	6.953
Mar	68.012	68.016	18.873	7.403	2.666	7.416
Apr	66.813	71.485	22.283	10.594	3.473	7.511
May	79.414	79.881	22.789	12.379	2.832	6.971
Jun	90.265	90.488	22.973	14.624	2.245	5.140
Jul	95.756	96.294	22.407	15.410	1.756	3.611
Aug	95.078	96.960	22.530	15.250	1.510	4.090
Sep	94.349	95.844	21.864	14.148	1.230	5.026
Oct	87.353	90.802	20.335	10.808	1.140	7.348
Nov	81.527	86.661	17.332	7.091	0.672	7.285
Dec	77.253	81.177	13.958	4.102	0.658	6.075

Source: DHM

The Penman's calculation is shown in **Appendix – B**.

Table 2.4: Monthly ET_0 values computed using Penman's Method

Month	ET0 (mm/d)	Month	ET0 (mm/d)
Jan	1.804	Jul	3.213
Feb	2.531	Aug	3.203
Mar	3.531	Sep	3.047
Apr	4.717	Oct	2.939
May	4.531	Nov	2.082
Jun	3.874	Dec	1.535

b. CROPWAT

The Food and Agriculture Organization of the United Nations (FAO) developed CROPWAT, software for scheduling irrigation and calculating crop water requirements. The database registry of CROPWAT software does not recognize any of the hydrological stations covering our catchment area. Thus, we go along with the nearest registered hydrological station, which is Kathmandu Airport. The ET_0 values obtained are thus given below.

Table 2.5: CROPWAT results for potential evapotranspiration

Country	Location 20			**Station**	KATHMANDU-AIRPORT		
Altitude	1337 m			**Latitude**	27.70°N	**Longitude**	85.36°E
Month	**Min Temp °C**	**Max Temp °C**	**Humidity %**	**Wind km/d**	**Sun hours**	**Rad MJ/sqm/d**	**Eto mm/d**
January	1.9	17.3	66	43	7.1	13.3	1.62
February	3.5	19.5	62	61	7.6	16	2.22
March	7.2	23.9	62	69	7.6	18.5	3.08
April	11.6	26.7	53	86	7.7	20.6	4.02
May	15.6	27.5	61	95	7	20.4	4.25
June	18.9	28.1	78	86	3.8	15.9	3.49
July	20	27.5	78	69	2.9	14.4	3.21
August	19.8	27.8	76	61	2.9	13.8	3.11
September	18	26.5	80	43	4.9	15.3	3.12
October	13.2	25	72	43	6.7	15.5	2.8
November	7.4	21.9	67	35	7.6	14.3	2.11
December	2.7	18.6	63	35	7.4	13	1.61
Average	11.7	24.2	68	60	6.1	15.9	2.89

CLIMWAT is used in calculation of ET_0 within the CROPWAT. CROPWAT was used to calculate the IWR of the crops in the command area but there is very less interaction available in calculation of ET_0. Hence the ET_0 was calculated manually in Microsoft Excel and provided to CROPWAT as input for the calculation of IWR. The values obtained are thus given below.

Table 2.6: CROPWAT results for Irrigation Water Requirements

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation deficit													
1	Rice	0.0	0.0	0.0	0.0	0.0	91.1	90.0	0.0	16.1	95.8	29.4	0.0
2	Maize (Grain)	0.0	5.3	42.8	144.3	111.5	4.7	0.0	0.0	0.0	0.0	0.0	0.0
3	Potato	68.2	36.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.3	49.0
4	Small Vegetables	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.5	6.0	0.0	0.0
5	Small Vegetables	62.1	40.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.9	42.5
Net scheme irr.req.													
in mm/day		2.2	1.5	1.4	4.8	3.6	2.7	2.5	0.0	0.5	2.7	1.5	1.5
in mm/month		67.3	42.4	42.8	144.3	111.5	82.1	76.5	0.0	14.7	82.3	44.7	48.0
in l/s/h		0.25	0.18	0.16	0.56	0.42	0.32	0.29	0.00	0.06	0.31	0.17	0.18
Irrigated area (% of total		100.0	200.0	100.0	100.0	100.0	185.0	85.0	0.0	100.0	100.0	185.0	100.0
Irr. Req. for actual area		0.25	0.09	0.16	0.56	0.42	0.17	0.34	0.00	0.06	0.31	0.09	0.18

We used Excel and calculated the ET_0 half monthly values manually. The climate data of the command area was obtained from DHM and Penman's equation was used to obtain monthly

ET_0 value. Half monthly values of ET_0 were then obtained. The values of ET_0 obtained are as shown below.

Table 2.7: Half monthly values of potential evapotranspiration

Month	Section	Half Monthly Eto	Month	Section	Half Monthly Eto
Jan	1	1.737	Jul	1	3.379
	2	1.986		2	3.211
Feb	1	2.35	Aug	1	3.206
	2	2.781		2	3.164
Mar	1	3.281	Sep	1	3.086
	2	3.828		2	3.02
Apr	1	4.421	Oct	1	2.966
	2	4.671		2	2.725
May	1	4.577	Nov	1	2.296
	2	4.367		2	1.945
Jun	1	4.038	Dec	1	1.671
	2	3.709		2	1.602

2.3 Field Investigation

2.3.1 Discharge measurement

In the field, we measured the discharge of the river using the floating head and flowmeter method.

a. Floating head Method

In AutoCAD, the cross section seen in Figure 2.5 had been generated. The discharge was then determined by calculating the cross-sectional area.

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

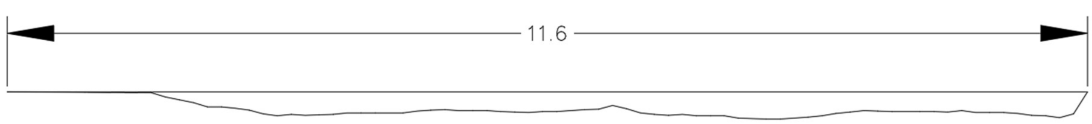


Fig 2.5: Cross section measured at Manohara for discharge measurement

The steps taken were:

i. Two sections near the proposed headworks were selected.
ii. Cross sectional area of the headworks section was surveyed through measuring tape.

iii. Floating objects of varying densities, such as plastic bottles, sandals, and wooden sticks, were utilized to compute the time to reach the destination.

iv. Velocity was determined using the time and distance traveled as follows:
velocity = length/time.

v. The same process was repeated at the second downstream section. The discharge obtained by the floating method was found to vary a lot from the flow meter. So, the data obtained from the floating method was discarded and further calculation was proceeded with flowmeter.

b. Flowmeter Method

Table 2.8 shows the computed velocity and the obtained flowmeter data. Flow was measured using flowmeter at two sections and the depth at different chainage was also calculated. The calculation is shown in **Appendix – D.**

Table 2.8: Flowmeter calculations

	Section 1		**Section 2**	
	Speed (m/s)	Area (sqm)	Speed (m/s)	Area (sqm)
	0.0610	0.0090	0.3500	0.0210
	0.8840	0.0816	0.7100	0.1060
	1.1280	0.1360	0.8230	0.0910
	1.3720	0.1950	1.2400	0.2570
	1.2800	0.1710	1.1000	0.2280
	1.0360	0.1595	0.9700	0.1580
	0.0610	0.0280	0.1300	0.0050
Q (cumecs)	0.8790		0.8810	

2.3.2 Survey

A complete topographic survey was carried out to create a topographic map and a longitudinal profile for the canal. The topographic map was then utilized to determine the location of the headwork. The longitudinal canal profile is used to align the canal bed while optimizing the cut and fill that happened during the alignment.

2.4 Design Flood Estimation

A design flood is a fabricated runoff discharge (peak discharge or hydrograph) that serves as a template for project component engineering design. Typically, observed plotting position data is fitted with a suitable probability distribution to estimate the design flood. This study's goal is to see if model selection criteria seldom used in hydrological applications will help identify the best probability model for this purpose.

Table 2.12 shows some rational, geographical, and empirical strategies for estimating design floods.

2.4.1 Theoretical Method (Rational Method)

A sensible calculation for flood discharge considers the area, slope, and permeability of the basin in addition to the amount, distribution, and duration of rainfall. This approach can be compared to the empirical approach because it too relies on the idea of the correlation between rainfall and runoff. Nonetheless, the Rational Method gets its name from the fact that the quantities utilized have approximate numerical consistency in their units. Due to its ease in practice, this method has gained popularity. Small rural catchments can use this strategy. A typical rational formula is:

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

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.

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

Table 2.9: Runoff coefficients for type of basins

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

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

In the absence of rainfall intensity data, the mean rainfall intensity (in cm/hr.) can be approximated using Sherman's equation and the coefficients provided by Rambabu et. al.

$$i_{(t_c,p)} = \frac{KT^a}{(t_c + b)^n}$$

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

For Nepal, this value can be assumed as for Northern 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's equation.

$$i_{(t_c,p)} = \left(\frac{R_p}{24}\right)\left(\frac{24}{t_c}\right)^{0.667}$$

where,

R_p = the 24-hr maximum rainfall for probability P

t_c = time of concentration

It is the time that the precipitation takes to travel from the drainage basin's furthest point to the stream or river's point of consideration.

t_c can be best estimated by Kirpich's equation given below:

$$t_c = 0.01947L^{0.77}S^{-0.385}$$

where,

t_c = time of concentration (minutes)

Also,

$$t_c = 0.000324L^{0.77}S^{-0.385}$$

where,

t_c = time of concentration (hours)

In both equations, L is length of the drainage basin in m measured along river channel up to the farthest point on the periphery of the basin and S is average slope of the basin from the farthest point to the discharge measuring point under consideration.

$$S = \frac{(h_2 - h_1)}{L}$$

where,

h_2 = maximum elevation (m)

h_1 = minimum elevation (m)

2.4.2 WECS/DHM 1990

The Department of Hydrology and Meteorology (DHM) and the Water and Energy Commission Secretariat (WECS) in Nepal have established empirical relationships for the analysis of floods with varying frequency. It is an adaptation of the 1982 WESCS methodology. The following formula yields the 2-year return period:

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

And the formula for 100-year return period is given by:

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

where,

Q = design flood in m^3/s.

A_{3000} = basin area (in km^2) below 3000 m elevation.

For other return periods,

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

where,

s is the standard normal variate.

And, $\sigma = \frac{ln\left(\frac{Q_{100}}{Q_2}\right)}{2.32}$

Limitations:

- It is limited to catchment areas within Nepal.
- It doesn't take the local rainfall patterns into account.
- It employs an empirical methodology.

2.4.3 DHM 2004 Method

The DHM (2004) method is an update to the WESCS/DHM 1990 method.

The formula for 2-year return period is given by:

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

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

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

where, Q is design flood in (m^3/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

And $\sigma = \frac{ln\left(\frac{Q_{100}}{Q_2}\right)}{2.32}$

Table 2.10: Standard Normal Variate for WECS/DHM 1990 and DHM 2004

T (yrs)	**S**	**T (yrs)**	**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. 4

Limitations:

- It is limited to catchment areas within Nepal.
- It doesn't take the local rainfall patterns into account.
- It employs an empirical methodology.

2.4.4 MHSP 1997

Based on this method, the peak flood 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

Table 2.11: Values of k & b for MHSP 1997

T	5	20	50	100	1000	10000
k	7.4008	13.0848	17.6058	21.5181	39.9035	69.7807
b	0.7862	0.7535	0.738	0.7281	0.6969	0.6695

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

Limitations:

- It is limited to catchment areas within Nepal.
- It doesn't take the local rainfall patterns into account.
- It employs an empirical methodology.

2.4.5 PCJ 1996

The PCJ 1996 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/s

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

$$a_p = a_{hr} k_t$$

where,

a_p = Hourly rainfall intensity for required exceedance probability (p) in mm/min at selected rainfall stations

k_t = Reduction coefficient of hourly rainfall intensity (depending 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 km^2

k_t = Coefficient for unequal distribution of rainfall in distinct 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

Advantages of PCJ method:

- While another technique considers the daily maximum, it considers the hourly maximum. It is therefore more accurate in this regard.

Limitations of PCJ method:

- Only the estimation of flood discharges up to a three-hundred-year return period may be done using this method.

- The values of this method were established specifically for Nepal; they are not applicable to other areas.

2.4.6 Modified Dickens' Formula

Dickens (1885) conducted research in India to determine the relationship between discharge rate and drainage area, and the following formula was constructed from his findings.

$$Q_T = CA^{\frac{3}{4}}$$

where,

Q_T = peak flow rate

C = Regression constant

A = Area of Drainage (km^2)

The modified Dickens' approach is an update to the original Dickens' method. After conducting frequency research on Himalayan Rivers, the Irrigation Research Institute in Roorkee, India, proposed the following updated relationship to calculate Dickens' constant.

$$Q_T = C_T A^{\frac{3}{4}}$$

where,

Q_T = maximum flood discharge (m^3/s) in T years

A = Catchment area (km^2)

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

$$C_T = 2.342 \log(0.6T) \log\left(\frac{1185}{P}\right) + 4$$

$$P = 100\left(\frac{a+6}{a+A}\right)$$

where,

a = perpetual snow area in km^2

T = return period in years

Limitations:

- It was developed for India.
- It is an empirical method.
- It does not consider any other parameter than catchment area.

Table 2.12: Flood discharge calculations in m^3/s from different methods

Projected Years	Methods					
	Rational	**WECS 1990**	**2004 DHM**	**MHSP 1997**	**Mod. Dickens**	**PCJ 1996**
2	61.48	39	43	18.05	56	24.52
5	78.2	68	80	45.13	70	33.69
10	90.85	91	111	55.56	93	44.07
20	103.39	116	145	76.43	110	61.06
33	112.64	132	168	87.3	121	81.93
50	120.21	153	197	101.52	131	99.14
100	132.87	183	241	123.89	147	122.42
200	145.51	216	290	150.65	165	137.36
300	152.91	232	314	166.96	173	148.35
500	162.23	265	363	190.31	184	164.93
1000	174.88	305	425	224.05	201	198.95

As seen in Figure 2.6, we have discussed the values of flood discharge at various frequencies using various approaches in both tabular and graphical format. The comprehensive computation can be found in **Appendix - D**. Of those techniques, PCJ 1996 provides an increasing curve of predicted flood discharge since it considers the highest hourly rainfall, whilst another technique makes use of the area's daily maximum rainfall. However, the rainfall pattern in that location is not considered by WECS/DHM 1990, DHM 2004, and MHSP 1997. Therefore, the values derived from these techniques are not used. Therefore, we apply the Rational approach to determine the maximum flood discharge value of the remaining technique to be used for design. The approximate flow rate for a 100-year return period is 132.87 m^3/s or 133 m^3/s.

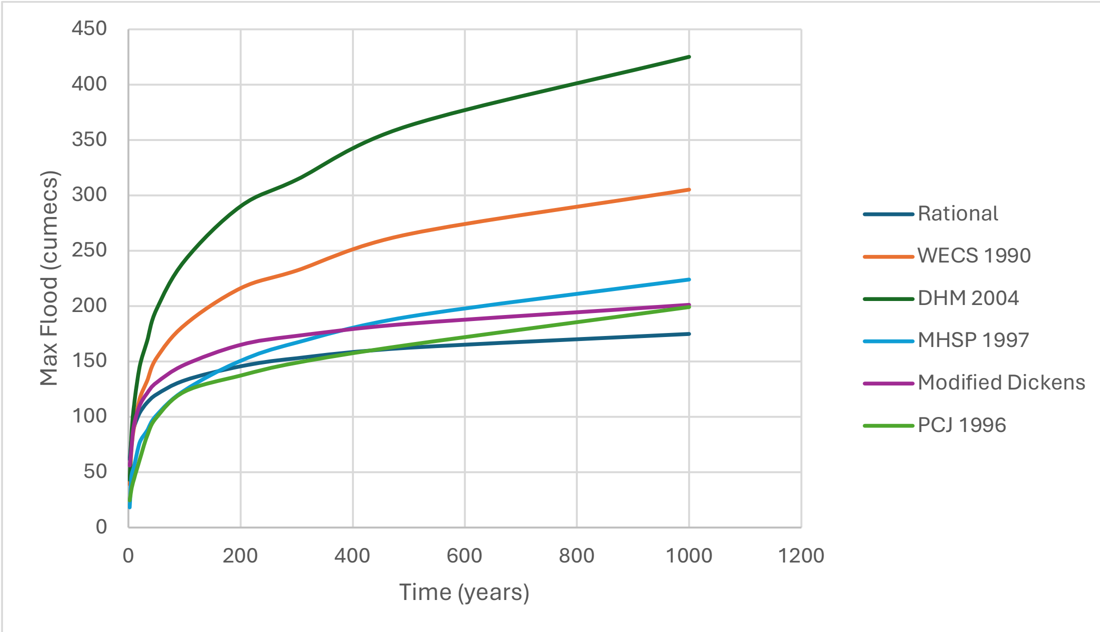


Fig 2.6: Graphical representation of flood v/s time by different methods

2.5 Calculation of High Flood Level

A high flood level is the height or elevation at which floodwaters rise during a flood. The weir, a structure that allows surplus water to flow over the top and discharge safely downstream, must have a capacity defined by the high flood level. To ensure the safe and efficient operation of weirs during flood occurrences, it is imperative that the high flood level be accurately estimated throughout the design process. In our case we have measured the high flood level with the use of HEC-RAS with DEM and flow data available of the area.

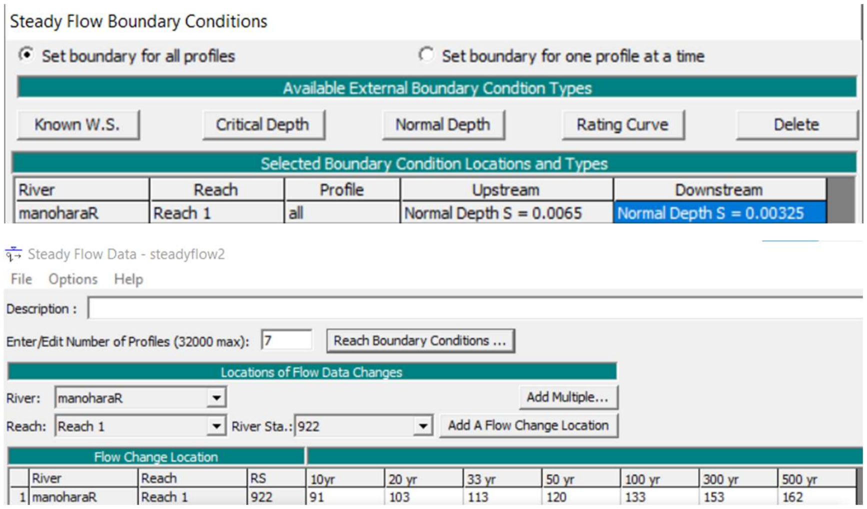


Fig 2.7: Boundary conditions for river flow

The High Flood level in HEC-RAS is estimated using the subsequent procedures:

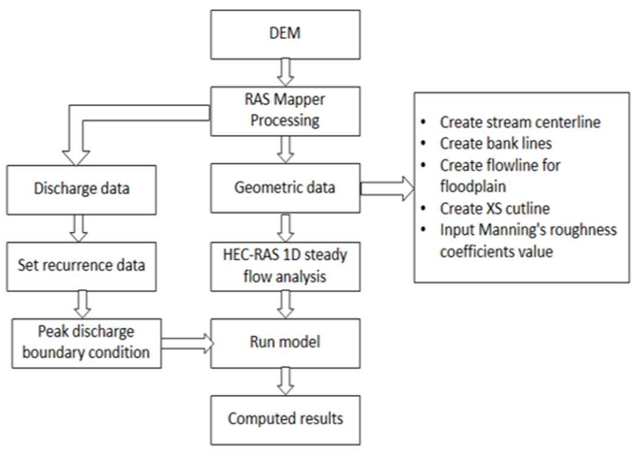


Fig 2.8: Flowchart of modelling process in the HEC-RAS

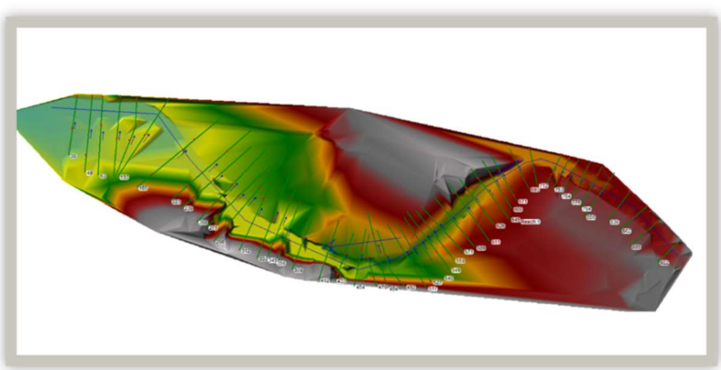


Fig 2.9: Geometric Data obtained in HEC-RAS through DEM

The outputs obtained are as follows:

2.5.1 Rating Curve

A rating curve is plotted between water level and calculated flood discharge. As mentioned earlier, the recommended design flood discharge is 133 m^3/s. From the graph below, the corresponding value of high flood level is 1305.23 m. Similarly, the cross section of river at weir site is studied which also concludes value of HFL as 1305.23 m.

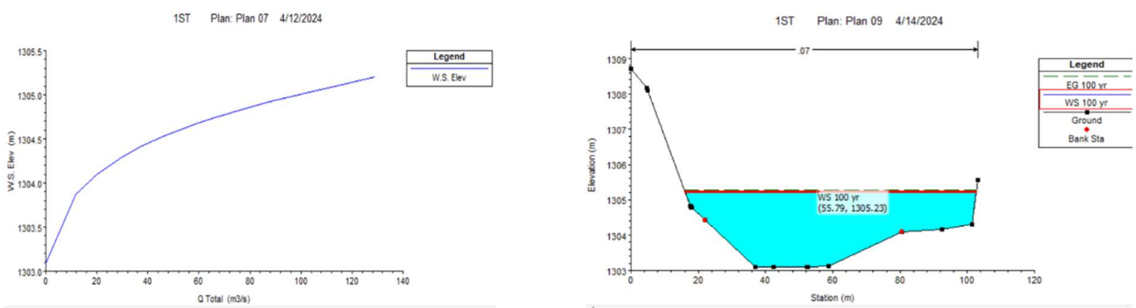


Fig 2.10: Rating curve and channel section indicating HFL

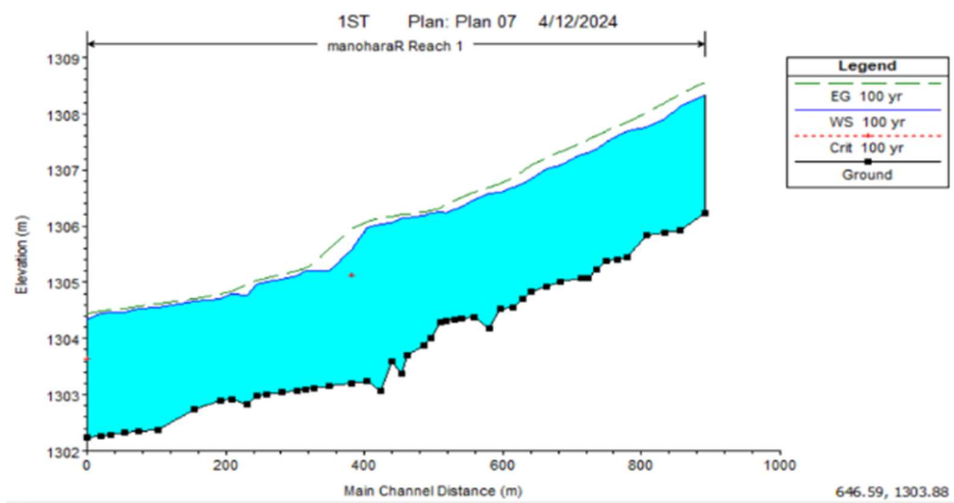


Fig 2.11: Longitudinal section of river channel at intake site

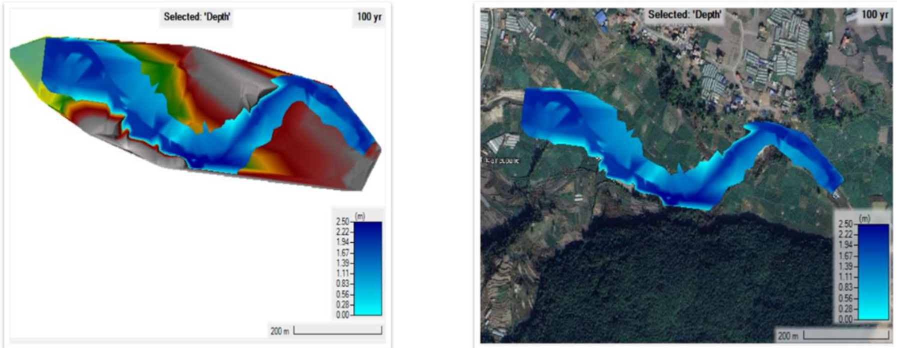


Fig 2.12: Depth map of Manohara River

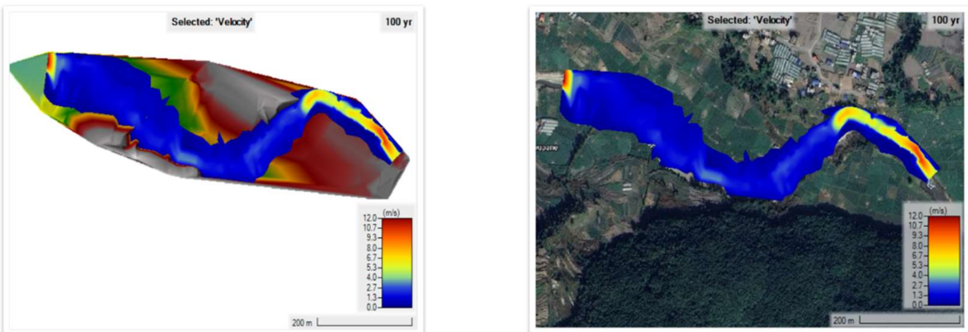


Fig 2.13: Velocity map of Manohara River

2.6 Availability of Monthly Flows

The monthly flow is the volume of water that flows through a river or stream each month, and it can be expressed in cubic meters per second or any other appropriate unit. To understand the seasonal variability of water availability and plan for water consumption appropriately, this knowledge is crucial for the management and planning of water resources. "Designed monthly flow" is the expected or projected rate of flow in a river or stream for every month of the year. The estimate is based on several variables, including water supply, demand, and other pertinent variables related to the management of water resources. The idea is typically used in water management and allocation tasks as a standard or goal.

We may also compute the monthly flows by using the input parameters of the flood computation algorithms. But since we used the area velocity method to measure the flow in this instance, the monthly flows are accurate.

2.6.1 MIP Method

Nepal is classified into numerous hydrological regions using this strategy. Nepal has been split into 22 hydrological areas in MIP 2016 compared to 7 hydrological regions in MIP 1990. Measuring the discharge throughout the dry season (November to April) is the only way that the MIP approach provides a reliable projection. Periodic measurements serve as the foundation for the MIP technique. A unit hydrograph (lps/km^2) is used to predict the mean monthly discharge of a certain place based on the measurement of lowest flow, which is typically April. This hydrograph is used to build a non-dimensional hydrograph for seven regions.

We initially determined the April flow in MIP 1990. This April flow was then used as the April flow in the MIP method, and additional values were acquired with the aid of the monthly hydrograph readings. Since we used the area velocity method to measure the flow in our example, the monthly flows were accurate. The monthly mean and reliable discharge were calculated using MIP non dimensional coefficients. Since our river is in region 3, we can obtain the monthly discharge by using the coefficients of region 3.

The non-dimensional coefficients are given below:

Table 2.13: Mean flow coefficients

Month	Region 1	Region 2	Region 3	Region 4	Region 5	Region 6	Region 7
Jan	2.4	2.24	2.71	2.59	2.42	2.03	3.3
Feb	1.8	1.7	1.88	1.88	1.82	1.62	2.2
Mar	1.3	1.33	1.38	1.38	1.36	1.27	1.4
Apr	1	1	1	1	1	1	1
May	2.6	1.21	1.88	2.19	0.91	2.57	3.5
Jun	6	7.27	3.13	3.75	2.73	6.08	6
Jul	14.5	18.18	13.54	6.89	11.21	24.32	14
Aug	25	27.27	25	27.27	13.94	33.78	35
Sep	16.5	20.19	20.83	20.91	10	27.03	24
Oct	8	9.09	10.42	6.89	6.52	6.08	12
Nov	4.1	3.94	5	5	4.55	3.38	7.5
Dec	3.1	3.03	3.75	3.44	3.33	2.57	5

Table 2.14: 80% reliable flow coefficients

Month	Region 1	Region 2	Region 3	Region 4	Region 5	Region 6	Region 7
Jan	2.41	2.15	2.47	2.61	2.38	1.64	3.48
Feb	1.78	1.62	1.82	1.89	1.77	1.29	2.3
Mar	1.31	1.31	1.35	1.33	1.35	1.03	1.52
Apr	1	1	1	1	1	1	1
May	2.06	0.77	0.82	1.33	1.08	0.85	4.35
Jun	3.91	2.31	1.29	2.11	2.23	2.88	7.83
Jul	9.38	9.23	4.71	3.56	6.15	18.64	23.91
Aug	21.25	20.77	20.59	0	13.85	25.42	40
Sep	11.88	15	17.65	0	10.77	20.34	34.78
Oct	6.56	5.38	7.65	6.94	6.54	4.07	16.52
Nov	4.38	3.62	4.82	5	4.42	2.63	8.7
Dec	3.19	2.77	3.53	3.61	3.27	2.03	5.22

The discharge was measured at Manohara River on 4th Dec 2023 and was found to be 875 liters per second. The discharges at other months were calculated using the non-dimensional coefficients and the result is as follows:

Table 2.15: Monthly flows from MIP Method

Month	Mean Monthly flow (lps)	80% rel. monthly flow (lps)
Jan	706.986	225.531
Feb	490.455	166.181
Mar	360.015	123.266
Apr	260.881	91.308
May	490.455	74.873
Jun	816.556	117.788
Jul	3532.322	430.062
Aug	6522.013	1880.035
Sep	5434.141	1611.589
Oct	2718.375	698.508
Nov	1304.403	440.105
Dec	978.302	322.318

Both long-term flows and a catchment with a similar hydrology do not exist here. Therefore, the most effective approach is to estimate monthly flow using regional approaches.

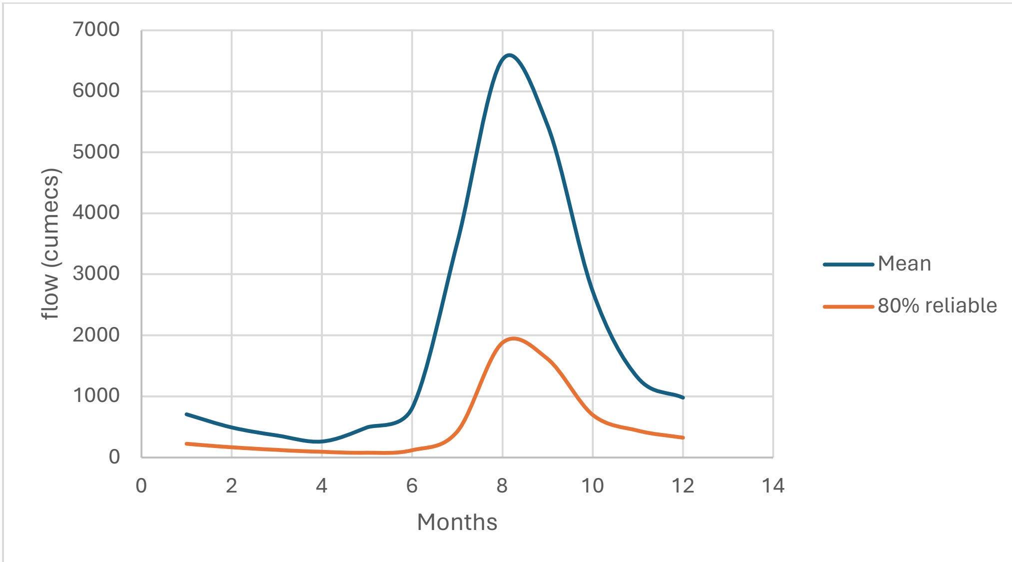


Fig 2.14: Graphical representation of monthly flow by MIP Method

2.7 Design Monthly and Half monthly flow

The MIP approach is used to obtain the design monthly flows. The monthly numbers are divided into half monthly values for an accurate estimate of the total amount of water needed. The climate records are kept for half-monthly flows for accurate measurement. For those half-monthly flows that cannot be collected, the figures are computed as follows:

$$a_2 = \frac{3a + b}{4}$$

$$b_1 = \frac{3b + a}{4}$$

Using the corresponding monthly values a and b, these equations are used to determine the values for the second half of month A (a_2) and the first half of the subsequent month B (b_1). Thus, the half-monthly flows are calculated using the previously mentioned idea and are provided in Table 2.16.

Table 2.16: Monthly and half monthly flows

Month	MIP monthly	MIP 1st half monthly	MIP 2nd half monthly
Jan	0.23	0.25	0.21
Feb	0.17	0.18	0.16
Mar	0.12	0.13	0.12
Apr	0.09	0.1	0.09
May	0.07	0.08	0.09
Jun	0.12	0.11	0.2
Jul	0.43	0.35	0.79
Aug	1.88	1.52	1.81
Sep	1.61	1.68	1.38
Oct	0.7	0.93	0.63
Nov	0.44	0.5	0.41
Dec	0.32	0.35	0.3

3. CROP WATER AND IRRIGATION WATER REQUIREMENTS

Crop water requirements, which are influenced by soil type, crop type, growth stage, climate (humidity, rainfall, temperature, etc.), and management techniques, relate to the quantity of water that crops require for healthy growth and development during a specific time. It is a projection of the entire volume of water used by a crop throughout its growth cycle, including water lost through transpiration and evaporation from the soil. Calculating the amount of water required by crops is essential since it enables farmers to plan water use efficiently, reduce water waste, and boost crop output.

This represents an approximation of the total volume of water used by a crop over its growth cycle, considering both soil evaporation and transpiration during plant development. To replenish the water lost to evapotranspiration, the crop must receive a certain amount of water through irrigation. In contrast, the quantity of water required for irrigation is to be supplied by the headworks to replenish the water lost due to conveyance, seepage, spillage, field capacity, or any other losses that may arise during the supply process.

3.1 Cropping Pattern

A cropping pattern is the configuration and timing of various crops grown on a plot of land throughout a given time frame. Farmers make this strategic choice based on a variety of criteria, including market demand, crop rotation considerations, soil type, climate, and availability of water. Cropping pattern selection has a big influence on sustainability, resource efficiency, and agricultural production.

3.1.1 Previous Cropping Pattern

As known from the locals, previous cropping patterns included paddy cultivation during monsoon and potato cultivation two times a year viz. during winter and before paddy season (monsoon). No new cultivations were tried because of following reasons:

- Farmers help each other with fieldwork during preparation of land, cultivation and harvesting as there are a lot of manual works to be performed. If anyone decided to try new cultivation and others did not follow, then they would be left out.
- If few perform new cultivations, there is a chance of crops being stolen.
- No engineering consult and soil tests have been performed till date for optimizing crop type for plantation.

3.1.2 Planned cropping pattern

Monsoon paddy was the current farming pattern used for this season, followed by winter vegetables and pulses afterwards. They are used because they are the ideal mix of cash crop and food crop, and because the cropping pattern meets farmers' needs and the availability of irrigation systems. Potatoes, maize, and monsoon rice are grown on 85% of the cultivable area that is irrigated.

With the remaining 15%, winter and summer vegetables are grown.

3.2 Computational Method

The information needed to determine the amount of water needed by crops is:

3.2.1 Rainfall and its contribution

IWR calculations typically assume 80% dependable rainfall.

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

where,

P_{mean} = mean rainfall

0.8416 is the reduced variate

3.2.2 Effective rain

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

where,

for paddy crops:

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

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

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

for upland crops:

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

3.2.3 Crop coefficient (K_c)

A crop's water requirements are estimated using the crop coefficient, a dimensionless factor used in irrigation and agricultural engineering that considers the crop's growth stage, leaf area, and other plant properties. It is the proportion of the crop's actual evapotranspiration to that of the reference crop grown in the same conditions.

Crop coefficient values for rice are derived from Table 3.1.

Table 3.1: K_c values 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

Source: Ministry of Water Resources M.3 Hydrology and Agro-meteorology Manual (1990), pg. 23

K_c values for other crops are selected from Table 3.2.

Table 3.2: K_c values 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

Source: Ministry of Water Resources M.3 Hydrology and Agro-meteorology Manual (1990), pg. 24

3.2.4 Land Preparation

The process of preparing agricultural land for cultivation, which includes tasks like leveling, harrowing, fertilizing, and plowing, is referred to as "land preparation." Only paddy crops are prepared on land. The FAO states that 55 mm is typically taken for land preparation.

3.2.5 Percolation Losses

The quantity of water lost through seepage into the soil or subsurface from a canal, reservoir, or other water storage structure is referred to as percolation loss. Percolation loss is also considered when determining the amount of water required for crops, but only for rice. The computation of irrigation water requirements involves the accumulation or definition of other losses, such as canal losses.

The amount of water needed for crops was determined using these characteristics. According to FAO (Food and Agricultural Organization) guidelines, the losses—typically field application losses, farm distribution channel losses, and main canal losses—were added as an efficient way to determine the amount of water needed for irrigation.

The comprehensive computation is displayed in **Appendix – C.**

Table 3.3 contains a tabulation of the IWR achieved for each half month.

Table 3.3: Gross Irrigation Water Requirement for each half month

Month	Half	I-gross (lps/ha)	Month	Half	I-gross (lps/ha)
Jan	1	0.521	Jul	1	0.013
	2	0.604		2	0.625
Feb	1	0.592	Aug	1	0.763
	2	0.598		2	0.273
Mar	1	0.471	Sep	1	0.624
	2	0.238		2	1.159
Apr	1	0.386	Oct	1	1.482
	2	0.537		2	1.611
May	1	0.662	Nov	1	0.168
	2	0.394		2	0.273
Jun	1	0.000	Dec	1	0.333
	2	0.000		2	0.415

3.3 Duty of Water

The amount of land that is irrigated by delivering one m^3 of water per second to support crop growth is known as the duty of water. The area is commonly measured in hectares. And, $ha/m^3/s$ is the unit of duty, therefore.

Regarding our irrigation setup,

- Net command area = 45.8 ha
- Maximum discharge = 0.241 m^3/s
- Thus, the duty of water = 190.041 $ha/m^3/s$

4. DESIGN OF HEADWORKS

4.1 Design of Weir

4.1.1 Introduction

The construction and design of weirs are helpful in areas where water needs to be stored for periods and diverted to the command area when required. Since Manohara River is a river of inconsistent flow, a weir must be used to store and irrigate the land during low flow periods. Therefore, a sloping glacis one bay weir is constructed such that water level is raised up to the required pondage level and flood can be bypassed during probable case of flood of one hundred years return period.

4.1.2 Design Parameters for weir and under-sluice

- The high flood discharge, calculated using the rational method for 100 years return period, is 133 m^3/s.
- High flood level that corresponded to the high flood discharge was found out by the rating curve.
- Pond level was obtained by finding out the weir crest that must be raised to store the water required for pondage.
- Afflux was taken as 1m.
- Retrogression was taken as 0.5m.
- The concentration factor was taken as 20%.
- From stability analysis, the width of the weir crest is 1 m.
- The upstream weir slope is 2:1 and downstream glacis slope is taken as 3:1.
- Lacey's silt factor = 1.2 and Safe exit gradient = 1/6 for medium sand.

4.1.3 Design Process

Following data were prepared/collected before design of weir:

- High flood discharge
- River cross section at the weir site
- Stage discharge curve i.e., rating curve for the river at weir site

Factors to be decided while designing a weir are:

- The full supply level of canal was obtained by subtracting 0.5 m to the pond level.
- The waterway was measured from Lacey's wetted perimeter equation.

$$P = 4.75\sqrt{Q}$$

Steps to be followed during designing a weir are:

- The discharge is calculated for a broad crested weir using the following formula:

  $$Q = 1.7(L - KnH)^{3/2}$$

  where,

  Q = discharge in m^3/s

  H = Total head in meter including velocity head

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

  L = clear waterway length in meters

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

- Determine head loss (HL) for different flow conditions.
- For known values of q and HL, read corresponding values of Ef_2 from Blench curve (Figure 4.1).

  $Ef_1 = Ef_2 + HL$

  Cistern level = downstream TEL - Ef_2

- Knowing Ef_1, Ef_2 and q, calculate values of d_1 and d_2 using given formulae.

$$Ef_1 = d_1 + \frac{\left(\frac{q}{d_1}\right)^2}{2g}$$

$$Ef_2 = d_2 + \frac{\left(\frac{q}{d_2}\right)^2}{2g}$$

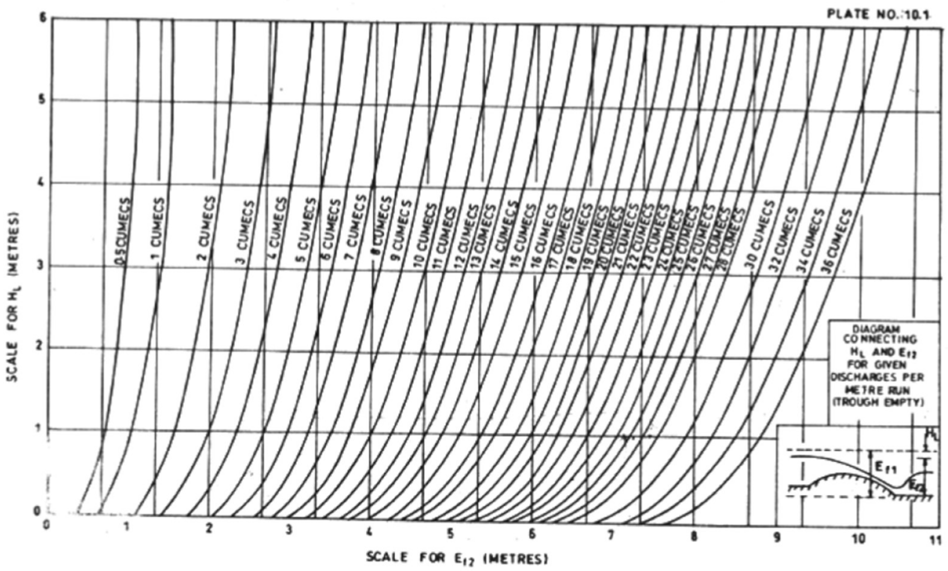


Fig 4.1: Blench Curve

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

- Determine scour depth from the formula:

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

Depth of upstream sheet pile from scour consideration = 1.5R

Depth of downstream sheet pile from scour consideration = 2R

- The hydraulic jump 's features for High Flood conditions are evaluated for given situations:
    - No flow concentration and No retrogression
    - 20% Concentration and 0.5 m Retrogression

  Because of damming of water during pond level conditions, only high flood evaluation is needed.

- Find the value of $\frac{1}{\pi\sqrt{H}}$ from the equation of $\frac{1}{\pi\sqrt{H}} = G_E\frac{d}{H}$ for the adopted value of G_E and the known value of H (maximum static head). Corresponding to the value of $\frac{1}{\pi\sqrt{H}}$, value of d is computed.

  Total length of floor $b = \alpha d$ is provided.

  Deposition of total floor length is as follows:

- Cistern length = 5*(d_2 - d_1) to 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

➢ The percentage pressures at the sheet piles on both the ends are calculated to find out the uplift pressure on the floor. For ease in computation, the pressure distribution from upstream end to the downstream end is supposed to be linear.

- Correction due to floor thickness:

Nominal floor thickness = 1 m (assumed at both ends)

Correction upstream = $\frac{t_1}{d_1} * (\phi_D - \phi_E)$

where,

t_1 = floor thickness at upstream end

d_1 = depth of sheet pile

Correction downstream = $\frac{t_2}{d_2} * (\phi_E - \phi_D)$ which is negative

where,

t_2 = floor thickness at downstream end

d_2 = depth of sheet pile

- 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 is done in case of provision of intermediate pile.

➢ The corrected percentage pressures are used to compute the hydraulic gradient line and subsoil pressure gradient line.

➢ For drawing the pre jump profile, depths at different locations of glacis are computed using the corresponding values of Ef1 and q.

➢ The water profile after the jump can be plotted after knowing the Froude number and using the relation between x and y values of the profile from the curve in Figure 4.2.

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

➢ Plot post jump profile using the following curve.

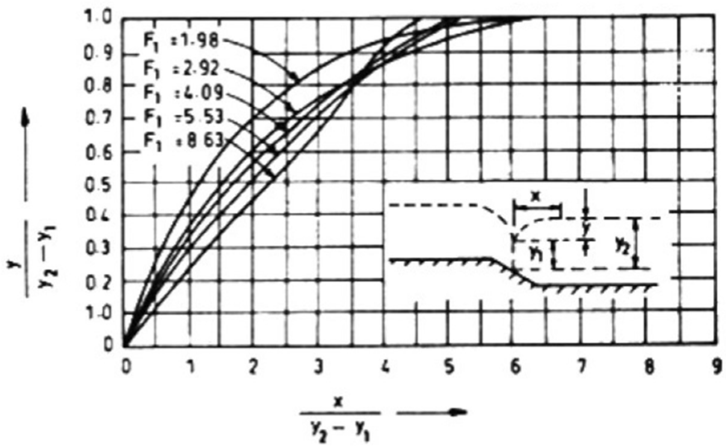


Fig 4.2: Curve for plotting post jump profile (weir)

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

➢ 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 (D_1)

Adopt certain thickness of gravel filter (taken as 0.75m).

Length of launching apron = 1.5 * scour depth below upstream bed (D_1)

Thickness of launching apron= 2.24 * D_1

- For d/s protection works:

The length of concrete block = 1.5 * scour depth below downstream bed (D_2)

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

Length of launching apron = 1.5 * scour depth below downstream bed (D_2)

Thickness of launching apron= 2.24 * D_2

4.1.4 Information

Table 4.1: Weir design data

Descriptions	**Obtained Value**
D/S HFL	1305.23 m
Waterway	57 m
Crest Level	1305.15 m
Pond Level	1305.15 m
Height of weir from river bed	2.1 m
Crest Width	1 m
U/S slope	1:1
D/S sloping glacis	3:1
RL of u/s floor of weir (river bed)	1303.05 m
RL of d/s floor of weir	1302.8 m
Length of u/s impervious floor	2 m
Length of d/s impervious floor	8 m
Thickness at toe of glacis	2.2 m
Length of concrete block at u/s of weir	2.5 m
Length of launching apron at u/s of weir	3 m
Length of concrete block at d/s of weir	3.8 m
Length of launching apron at d/s of weir	4 m

Here, the weir's position was chosen such that it was confined to a straight, small, and well-defined channel between banks that was not submerged during the highest flood. When a gradual slope is required, as in the case of regulating the water flow rate in a canal or other watercourse, sloping glacis weirs are usually employed. In accordance with the specifications given above, a sloping glacis weir was selected. The silt factor and the exit gradient are assumed to be 1.2 and 1/6 respectively based on the type of material—medium sand—found in the river.

4.2 Design of Under-sluice

4.2.1 Introduction

By building an under-sluice section separated from the weir section with a divide wall, a relatively less turbulent pocket of water is formed near the canal head regulator. Typically, the sluice's crest is maintained at the river's lowest bed level. The purposes of an under-sluice are to maintain a distinct and unobstructed river route that leads to the regulator, to manage the entry of silt into the canal, and to pass low-level floods. Section 4.1.2 has offered design considerations also for the under-sluice.

4.2.2 Design Process

The discharge across the under-sluice section needs to be determined before the under-sluice's real design is started. The under-sluice section's discharge ought to comply with these conditions:

a. $\geq 2 * canal\ supply\ discharge$

b. $> minimum\ dry\ weather\ flow\ of\ the\ river$

c. Usually in practice, discharge through under sluice is taken to be 20% of total discharge.

Steps:

- Fix the crest on the level of riverbed and take downstream glacis slope as 3:1.
- Fix the water way for the under-sluice section.
- Fix the waterway and calculate the discharge using the broad crested weir formula:

  $$Q = 1.7(L - KnH)^{3/2}$$

  where,

  Q = discharge in m^3/s

  H = Total head in meter including velocity head

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

  L = clear waterway length in meters

  K = coefficient of end contraction (0.1 for blunt noses to 0.04 for thin pointed noses)
- The hydraulic jump 's features for High Flood conditions as well as Pond Level conditions are evaluated for given situations:
  - No flow concentration and No retrogression
  - 20% Concentration and 0.5 m Retrogression

- Calculate the normal scour depth and determine the bottom levels of the upstream and downstream piles.
- Determine the total length of the impervious floor from the exit gradient considerations. Also, fix the lengths and levels of the upstream and downstream floors. The d/s floor is fixed below the point at which the hydraulic jump is formed.
- The percentage uplift pressures are calculated at the key points of all piles by Khosla's theory for the following conditions:
  - No flow condition
  - High flood condition, with flow concentration and retrogression
  - Pond level condition, with flow concentration and retrogression
- The subsoil hydraulic gradient line (HGL) is drawn for each case.
- Determine the uplift pressure at various points from the subsoil HGL for no flow condition. Also, the suction pressure at the jump through for high flood condition and pond level condition are determined.
- Plot post jump profile with the help of curve in Fig 4.2.

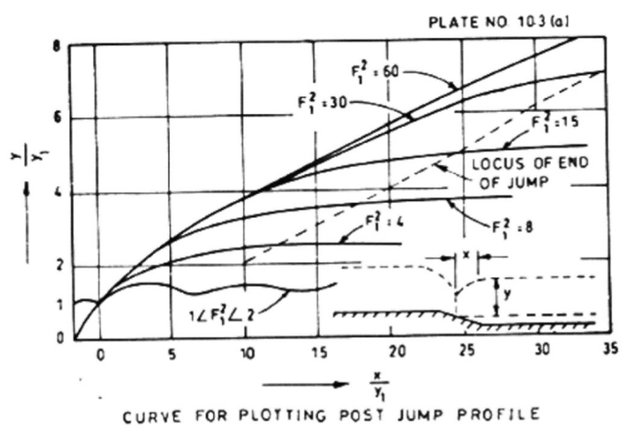


Fig 4.3: Curve for plotting post jump profile (under sluice)

Source: Jha, P.C. & Devkota (2074), pg. 276)

Following the procession of finding out the floor length, all the following processes from finding thickness at different points of floor to the protection works at upstream end and downstream end resemble the process followed during weir construction.

4.2.3 Information

Table 4.2: Under sluice design data

Descriptions	Obtained Value
Waterway	5 m
Crest Level	1303.05 m
D/S sloping glacis	3:1
RL of d/s floor of weir	1299.1 m
Length of u/s impervious floor	20.014 m
Length of d/s impervious floor	17 m
Thickness at toe of glacis	4.7 m
Length of concrete block at u/s of weir	7.7 m
Length of launching apron at u/s of weir	7.5 m
Length of concrete block at d/s of weir	9 m
Length of launching apron at d/s of weir	9 m

4.3 Canal Head Regulator

4.3.1 Introduction

At the canal entry, a regulator for the canal head must be installed. It's located on the riverbank, directly above the under-sluice. The head regulator in the Manohara irrigation project is positioned 3.1 meters upstream of the under-sluice.

4.3.2 Design Parameters

Design Consideration for Canal:

- Lacey's silt factor = 1.2
- U/S floor thickness = 1 m
- Exit Gradient = 1/6

4.3.3 Design Process

The crest level of the head regulator is kept at 0.5 m higher than the bed level. The d/s FSL is 1304.65 m, and the u/s pond level is 1305.15 m (0.5m modular head is maintained), the crest level is kept at 1303.55 m. The crest width is calculated as:

$$Q = \frac{2}{3} C_{d_1} \sqrt{2g} B (h)^{3/2} + C_{d_2} B h_1 \sqrt{2gh}$$

where,

h = head causing flow = pond level - FSL

h_1 = depth of water over crest on d/s

B = length of crest

$C_{d_1} = 0.577$

C_{d_2} = 0.80

HFL before construction of weir is 1305.23 m, the gate opening (x) required for the passing of 0.24 m^3/s discharge is given by $C_d A\sqrt{2gh_1}$.

where,

h_1 = head causing flow (u/s HFL – d/s FSL)

A = area of flow = $x.B$

C_d = 0.62 (coefficient of discharge)

The value of crest width has been decided to be 0.5 m (minimum required dimension) and opening size is 0.139 m. For the crest width of 0.5 m the discharge intensity was found to be 0.48 cumecs/m. Then, the total floor length was calculated following a procedure like that of weir and under sluice.

Length of downstream impervious floor $L = 5(y_2 - y_1) \cong$ 4 m.

Total length of impervious floor = 9 m (from exit gradient consideration)

Salient features of the canal head regulator floor are given as follows:

- Length of u/s floor = 3.8 m
- Length of d/s floor = 4 m
- Length of crest = 1 m
- Length of sloping glacis = 1.2 m
- Scour depth = .78 m (approximated to 1 m)

for f = 1.2 and q = 0.48 cumecs/m

- Depth of u/s pile = 3.57 m
- Depth of d/s pile = 3 m

After the application of Khosla's theory to find out pressure percentages at upstream and downstream ends of the platform and followed by corrections to the pressure percentages, curve for uplift pressure was obtained that was ultimately used in finding the thickness of the floor at key points. For the head regulator, the uplift pressure downstream is maximum in no flow condition. In this case, the seepage head was found to be 0.89 m.

The floor thickness is calculated as:

$$\frac{Uplift\ Pressure}{Specific\ Gravity\ of\ Concrete} - 1$$

- Floor thickness at d/s floor = 0.721 m adopt 0.9 m
- Floor thickness at sloping glacis = 0.861m adopt 0.9 m
- Length of d/s apron = 2 m
- Length of d/s inverted filter = 1.2 m

5. DESIGN OF MAIN CANAL AND CANAL STRUCTURES

5.1 Design of Main Canal

Water conveyance in canal system is done through open channel flow, tunnel flow, pipe flow or any other mode seen suitable according to the field condition. In Manohara irrigation project, an open channel flow system is used, and a 2.7 km long primary canal alignment is selected for commanded area of 45.8 Ha. From IWR calculation for planned cropping pattern 0.24 m^3/s peak discharge was put forward into design. Lining canals is preferred to avoid huge loss of water due to conveyance and percolation.

5.1.1 Design Parameters for Canal

M15 PCC lined canal is selected and its manning's coefficient value is 0.015.
For design discharge of 0.24 m^3/s (< 85 m^3/s) trapezoidal section is considered.

- Longitudinal slope = 1:500
- Side slope = 1:1
- Free board = 0.3 m
- Lining = 8 cm thick

5.1.2 Design of Canal Cross Section

For a lined section we require a section with the least wetted perimeter. For determining the section manning's equation is used in calculating canal dimensions.

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

where,
$R = A/P$ (hydraulic radius)
S = longitudinal slope of canal
Q= Canal discharge

The designed canal parameters are:

- Cross sectional area = 0.0375 – 0.35 m^2
- Wetted Perimeter = 0.52 – 1.61 m

- Mean velocity = 0.75 m/s
- Hydraulic Radius = 0.072 – 0.217 m
- Canal depth (y) = 0.45 – 0.8 m

5.1.3 Canal Alignment Selection

Primary canal runs along the contour. Secondary canals take water down the slope from the primary canal and distribute it directly to the fields through distribution boxes.

5.1.4 Canal Longitudinal Profile

Longitudinal profile is selected with consideration to balance out cut and fill volume or at least keep the filling work in excavation lower than cutting work, a residual cut of 788.15 m^3 is obtained. Longitudinal profile is selected using hit and trial method. The profile kept will keep the flow velocity of water in range so that no scouring or silting is seen.

5.1.5 Canal Lining

As per the guidelines for canal with discharge less than 3 m^3/s lining thickness of 8cm is suggested. The area for cultivation will increase on completion of irrigation project. Lined canals will have less maintenance cost, low seepage loss which will save lots of water in dry season in return saving potential revenue of project. With these factors into consideration, we have selected the lined canal section for the project. The economic calculation is shown in **Appendix – H.**

5.2 Design of Settling Basin

5.2.1 Introduction

A settling basin is a structure that removes the sediments from the water by settling down the sediments of specific sizes. A flush gate is provided at the bottom of the settling basin to flush out the settled down sediments. hydraulics is taken into consideration to maintain the transition in settling basin, ensuring no turbulence is seen in flow of water if such turbulence is seen all particles will not settle down.

5.2.2 Design Parameters

- Smooth transition is provided at inlet-outlet to prevent turbulence formation.
- Minimum diameter of settling particle is taken as 0.5 mm.
- Maximum diameter of particle to be flushed out is taken as 0.9mm.

- A gate is provided at bottom of settling basin to flush out settled particles.
- Automatic flushing mechanism selected for which 25% extra discharge is provided.
- The floor of settling basin is lowered than bed level of canal so as to prevent the settled particles at the head of gate from scouring.
- Escape is provided before entrance of settling basin to prevent over loading of structure.

5.2.3 Design Process

The design velocity for settling down sediment, critical bottom velocity and flushing velocity, all were determined from Nomograms (Fig. 5.1, Fig. 5.2, and Fig. 5.3 respectively). Plan area of settling basin is computed by knowing the velocity of settling of particles, such as:

$$Area = Q/V$$

where,

Q = discharge in m^3/s

V = settling velocity in m/s

Settling depth is calculated by the given expression:

$$Y = Q/(B * v)$$

Scour flow is taken as 1.25Q and scour slope is taken to be 1/310.

Gate height (Y_g) is calculated as: $Q = 1.7BY_g{}^{1.5}$

The dimensions of the settling basin are fixed as 12 m x 1.2 m x 1.1 m.

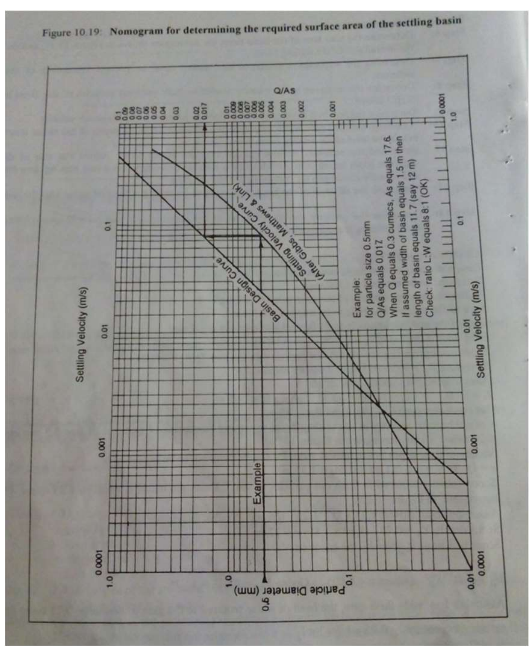


Fig 5.1: Graph of settling velocity v/s particle diameter

Source: Institute of Engineering (1993), pg. 182

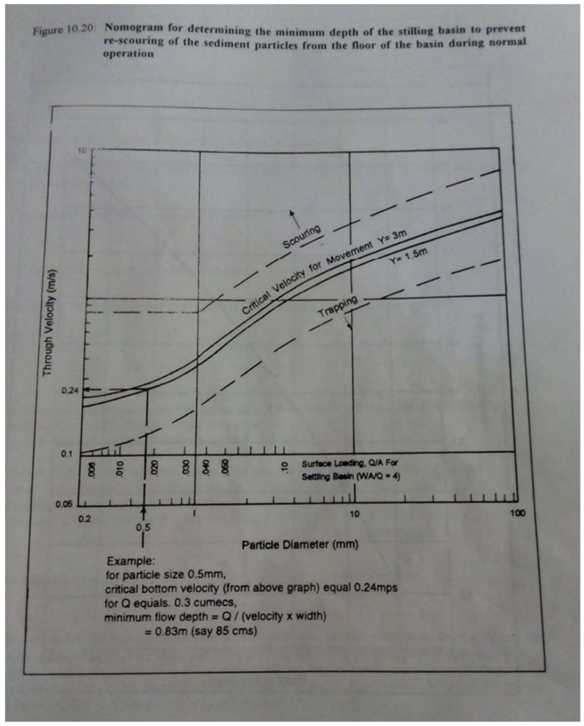


Fig 5.2: Graph of through velocity v/s particle diameter

Source: Institute of Engineering (1993), pg. 183

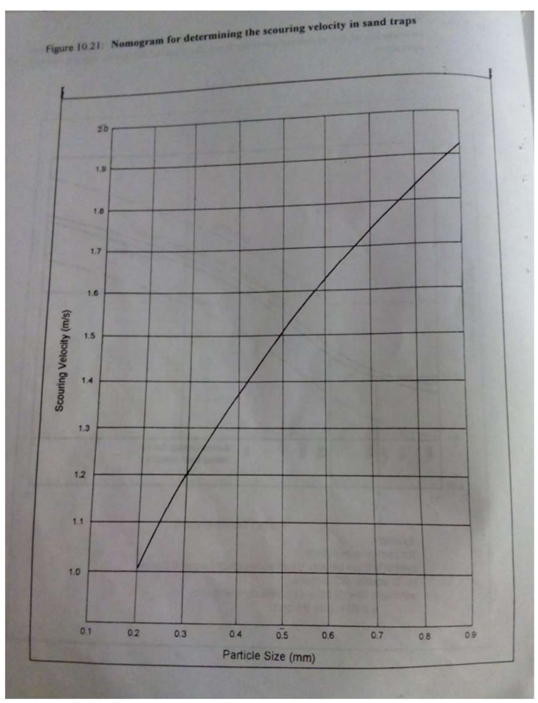


Fig 5.3: Graph of scouring velocity v/s particle diameter

Source: Institute of Engineering (1993), pg. 184

5.3 Design of Drop Structure (Sarda Fall)

5.3.1 Introduction

Where there is a vertical impact and a raised crest, it is referred to be a vertical drop fall. It got its name because the Sarda canal system in Uttar Pradesh, India, originally used it to replace notch valves. Its straightforward design and construction make it more cost-effective than others. It was discovered that using more small drops was more effective than using fewer large drops. In this kind of fall, the cistern serves as a site for energy dissipation where a tiny pond forms and water hits the water.

5.3.2 Design Parameters

- A rectangular section is made in approach of drop.
- Bed and crest are taken of equal length.
- Specific gravity of material (PCC) used is 2.4.
- The crest wall is rectangular shaped as the design discharge is less than 14000 liters per second.
- Bligh's approach is used for drop design.

5.3.3 Design Process

0.6 m top width was obtained by the formula:

$$Q = 1.84LH^{1.5}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

where,

Q = discharge (m^3/s)

L = Canal length (m)

H = Head over crest (m)

B_t = top width (m)

For cistern design,

Length of cistern (L_c) = $5(H*H_L)^{0.5}$

Cistern depth (x) = $0.25*(H*H_L)^{0.67}$

where,

H_L = Loss of head (m)

Cistern length is 4.06m, cistern depth is 0.19m. The design of the drop is included in **APPENDIX – F** and the drawing is included in **APPENDIX – G.**
As per the requirements, drops of 0.5m and 1m are designed.

5.4 Design of Distribution Box

5.4.1 Introduction

A distribution box is a flow dividing structure built to divide the flow among secondary and tertiary canals or farm ditches in medium hill irrigation schemes. The following figure shows an uncomplicated design that allows a high degree of flexibility to vary the flow among three downstream branches.

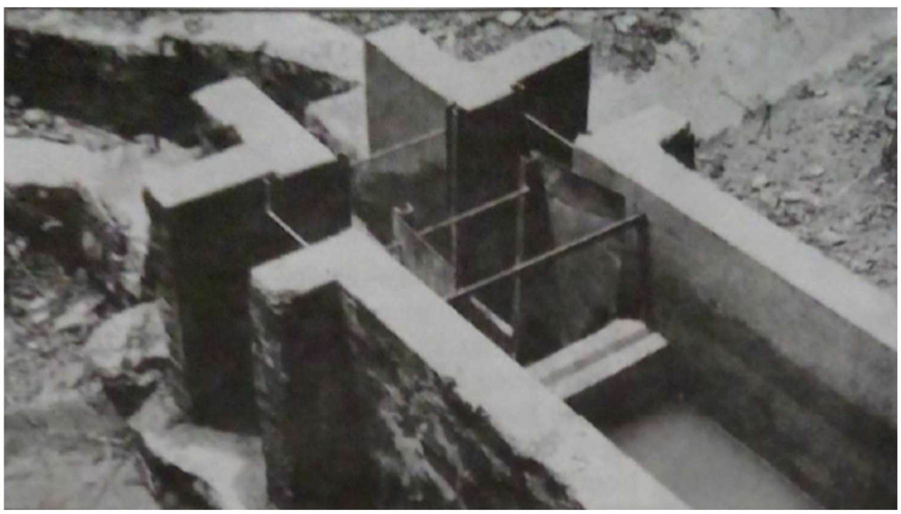


Fig 5.4: A distribution box operated and maintained by farmers

5.4.2 Design Parameters

The design of the distribution box is simple, yet efficient. The water entering the structure drops over a level sill made of brick. Two heavy gauged galvanized steel sheets divide the stream of flow as it drops over the sill. The brick sill is covered with cement mortar plaster to reduce leakage. The upstream ends of the steel sheets are adjustable. They are held in position by a slotted angle iron and the relative portion of the flow can be adjusted by loosening the angle and deflecting the steel sheet to either side into the desired slots thus changing the relative size of the openings. Wooden stoplogs are placed on the outlet side of the structure to shift the flow among the different downstream channels. The primary purpose of the stoplogs is to facilitate the rotational flow.

5.5 Design of Aqueduct

5.5.1 Introduction

An aqueduct is an irrigation canal that crosses a drainage channel without lowering the channel's bed to facilitate water transportation. At both upstream and downstream, the canal and drainage channels cross at a right angle and are reasonably straight in length. At the crossing, the area of the drainage channel and the sectional area of the canal at the aqueduct are typically smaller. Because of this increased velocity, which is likely to scour the bed, the drainage channel's sides and the aqueduct's upstream and downstream sides are paved with stone boulders set atop concrete blocks.

5.5.2 Design Process

➢ Estimation of design (maximum) flood discharge of a drain:

Analysis or prediction of peak flood from the drain must be done beforehand. The size of the drain might vary but it has a significant impact on the design.

➢ Waterway requirement for a drain:

Total waterway is calculated by Lacey's regime perimeter equation. The equation is given by:

$$P = 4.825\sqrt{Q}$$

where,

P = waterway for drain (m)

Q = drain flood discharge (m^3/s)

As the waterway is reduced by abutments the regime perimeter is increased by 20%. Guide banks confine the water flow into desired waterway.

➢ Flow velocity through barrel:

Flow velocity ranges from 1.8 - 3.0 m/s. This range asserts a proper value for non-silting and non-scouring velocities.

➢ Height of opening:

Adequate clearance must be provided between the drainage and bottom of slab of barrel. Clearance = 1 m or ½ height of culvert (whichever is less)

Opening height = Flow depth + Clearance

➢ Span number:

The number of spans to be provided is fixed based on structural strength required and economic consideration.

➢ Canal waterway

The floating ratio is taken to be 1/2 such that velocity in the trough does not exceed critical limit and no hydraulic jump is formed. A sufficient head is lost in this process and the cross-drainage structure is subjected to enormous stresses.

➢ Length of contraction or approach transition:

The length of contraction can simply be found out from the throat width and convergence ratio. The convergence ratio is taken as 2:1, i.e., not steeper than 30°.

➢ Length of expansion or departure transition:

Similarly, expansion length can also be simply found after knowing expansion ratio. The expansion ratio is taken as 3: 1, i.e., not steeper than 22.5°. To maintain streamlined flow and to reduce loss of head the transitions are generally made up of curved and flared wing walls. For the project, Mitra's method is used for hyperbolic transition. According to A.C. Mitra, the channel width at any section X-X, at a distance x from flumed section is given by:

$$B_x = \frac{B_n B_f L_f}{B_n L_f - (B_n - B_f)x}$$

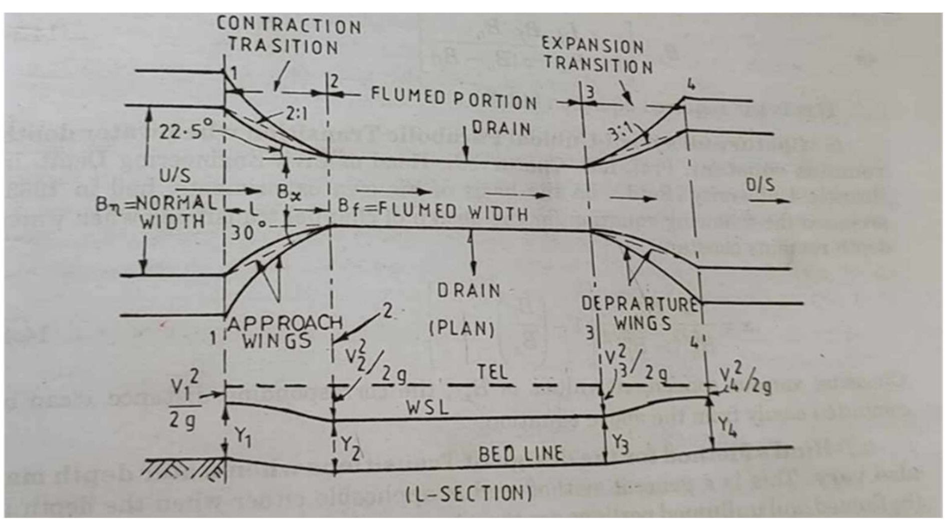


Fig 5.5: Plan and L-Section of transition in aqueduct

5.5.3 Pucca Canal Trough Design

In an aqueduct, the bed of canal is subjected to deadload and water load in downward direction as shown in Figure 5.6.

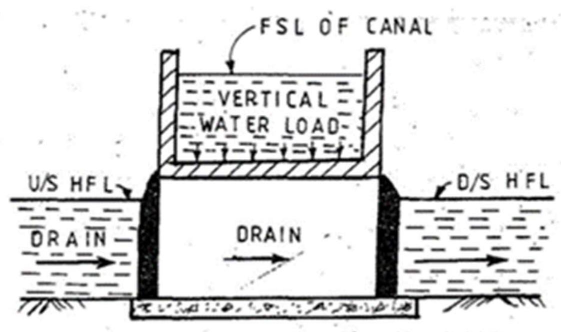


Fig 5.6: Loads acting in an aqueduct

In this case, canal bottom receives no uplift force from HFL of river, so it designed to resist the dead load of canal and the load of flowing water at FSL. Canal is selected of adequate thickness. And reinforcement can also be provided to maintain the thickness of canal within permissible limit. The side walls of canal must resist the entire horizontal force exerted by the canal water like a retaining wall.

For a Syphon:

In the case of an aqueduct syphon, the uplift pressure exerted by the drain water also comes into action. Although the uplift pressure acts in opposite direction to dead load and canal water load, the canal is designed considering the worst case when there is no uplift or when the drain/ river is empty when there is full supply in canal. Canal slab in syphon is designed for:

- dead load, full supply water load when there is no drain flow
- uplift from drain flow when the canal is empty

While calculating the thickness of slab for resisting the uplift of drain sometimes the thickness may exceed the design slab thickness for dead load of canal. It is suggested to not increase slab thickness more than this as further increase will result in more uplift force. Therefore, reinforcement is provided for resisting excess uplift force.

5.5.4 Bottom Floor Design

Uplift force is maximum in bottom floor of syphon when there is full canal flow and no water flow in drain/ river. The uplift is due to the seepage from canal to ground, as shown in Figure 5.7.

The seepage from canal to the bottom floor takes place in three dimensions. Thus, the computations can be complex and difficult since the flow net can be approximated to two dimensions. For smaller works Bligh's Creep theory may be used for checking seepage pressures.

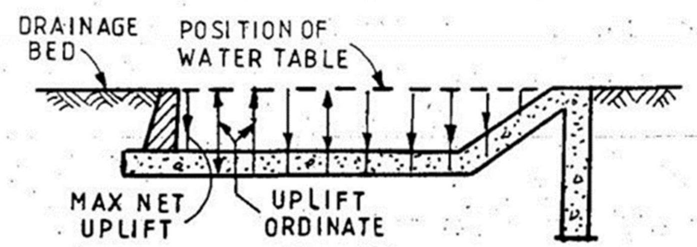


Fig 5.7: Action of water table on drainage

6. FINANCIAL ANALYSIS

6.1 Benefit Analysis

The project's income is exclusively reliant on the crops' productivity. The entire profit is derived from producing revenue after all costs are subtracted. The variables that come into play include production and expenses. Crop output and byproduct revenue are included under production. Certain crops, like maize and paddy, produce enormous amounts of byproducts that are suitable for feeding cattle.

The aspects under expenses are:

- Seed
- Organic manure
- Chemical fertilizer like Urea, DAP, Potash
- Labor and Bullock

Before irrigation project i.e., at present condition, the crops grown are:

- Monsoon Paddy
- Potato
- Other crops like Tomato (inside greenhouse), Onion, Garlic

The other crops were grown on negligibly less amount of land. If some farmers grew out of trend crops of that season, there were chances of them being stolen, as per interviews.

After the success of irrigation project, planned crops are as follows:

- Monsoon Paddy
- Potato
- Maize
- 4.Winter Vegetables
- Summer Vegetables

Data from the Department of Water Resources and Irrigation, which is shown in Table 6.1, is the source of the rate.

Table 6.1: Revenue through crops

SN	Description	Crops				
		Winter Vegetables	Summer Vegetables	Potato	Maize	Paddy
a	Production (t/ha)	7	9	40	8	2
b	Price (Rs/t)	18720	15120	8400	42000	40000
c	Value (Ra/ha)	131040	136080	336000	336000	80000
d	Byproduct (Rs/ha)					30000
A	Gross value of production (Rs/ha)	131040	136080	336000	336000	110000
	Expenses					
a	Seed (kg/ha)	0.8	0.8	1600	25	60
	Price (Rs/kg)	50000	50000	55	300	50
	Value (Rs/ha)	40000	40000	88000	7500	3000
b	Organic Manure (kg/ha)	7	7	7	7	4.2
	Price (Rs/kg)	2500	2500	2500	2500	2500
	Value (Rs/ha)	17500	17500	17500	17500	10500
c	Chemical Fertilizer					
i	Urea (kg/ha)	61.95	61.95	61.95	113.28	113.28
	Price (Rs/kg)	20	20	20	20	20
	Value (Rs/ha)	1239	1239	1239	2265.6	2265.6
ii	DAP (kg/ha)	32.5	32.5	32.5	0	0
	Price (Rs/kg)	50	50	50	50	50
	Value (Rs/ha)	1625	1625	1625	0	0
iii	Potash (kg/ha)	25	25	25	0	0
	Price (Rs/kg)	40	40	40	40	40
	Value (Rs/ha)	1000	1000	1000	0	0
iv	Pesticides (Rs/ha)	3000	3000	3000	1400	1400
d	Labor (md/ha)	120	120	120	104	104
	Price (Rs/md)	400	400	400	400	400
	Value (Rs/ha)	48000	48000	48000	41600	41600
e	Bullock (ad/ha)	25	25	25	25	25
	Price (ad/kg)	500	500	500	500	500
	Value (Rs/ha)	12500	12500	12500	12500	12500
B	Total cost of production (Rs/ha)	124864	124864	172864	82765.6	71265.6
C	Benefit per ha (A-B) (Rs/ha)	6176	11216	163136	253234.4	38734.4

Source: Department of Water Resources and Irrigation, Lalitpur

Here, multiplying the benefit per hectare by the command area yields the entire benefit.

6.1.1 Crop revenue without project

Without the initiative, it was discovered that roughly thirty hectares of land were planted with potatoes twice a year, and about thirty ha of land was used for monsoon paddy cultivation. In this instance, the crop revenue is: Rs. 32 x 38734.4 + 2 x 30 x 163136 = Rs. 1,10,27,660.8

6.1.2 Crop revenue with project

We planned for application of five different sorts of crops with the project. The commanded area varies depending on the available water for each type of crop. We planned a cropping of 45 ha of land by maize, 38 ha of land by potatoes twice a year, 6 ha of land by winter vegetables and summer vegetables each, and 38 ha of land by monsoon paddy.

With the project, crop revenue will be:

Rs. 38 x 38734.4 + 6 x 6176 + 6 x 11216 + 45 x 253234.4 + 2 x 38 x 163136 = Rs 2,53,70,143.2

Net benefit = Rs 2,53,70,143.2 - Rs. 1,10,27,660.8 = Rs. 1,43,42,482.4

Thus, the total benefit is Rs. Rs. 1,43,42,482.4 per year.

6.2 Cost Estimate

The unit rate approach is used to estimate the cost of the structure. Using estimating criteria, the type of materials utilized in those operations, their quantity, and the district-fixed unit rates, the cost of carrying out important activities is estimated. Most of the factors that go into construction work are added to the unit rates, including profit, overhead, and the cost of renting tools and equipment for unskilled labor. Thus, it provides a project cost estimate with a respectable degree of accuracy. It is a more realistic method of estimation because even losses are considered in accordance with the provided criteria.

Calculations are given in **Appendix – H**.

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

- M15 PCC (1:2:4) for RCC @ Rs. 15506 per m^3
- M20 PCC (1:1.5:3) for RCC @ Rs. 19699 per m^3
- First-class brickwork in 1:3 cement sand mortar @ Rs. 18121 per m^3
- Reinforcement works @ Rs. 150114.1 per MT
- Stonework in 1:3 cement sand mortar @ Rs. 15432 per m^3
- Earthwork excavation @ 746 per m^3
- Earthwork filling @ 533 per m^3
- Filling of gravel @ Rs. 3603 per m^3
- Stone Packing @ Rs. 2720 per m^3
- HDP piping for outlets @ Rs. 287 per kg

The project's overall construction cost comes out to be Rs. 17,61,25,553.

6.3 Cost Benefit Analysis

Although the initial investment cost in any project is relatively high, a project must be designed in such a way that the yearly revenue collected eventually exceeds the cost. Through discounted payback period and BCR analysis for 20 years period with a rate of interest 10%, the economic feasibility is done in this project. The project might undergo rehabilitation and the present trend of cost estimate might become irrelevant after 20 years, thus the chosen period.

Following things are considered before starting the economic analysis:

- Rate of interest = 10%, study period = 20 years
- Construction is scheduled to be complete in two years' time.
- Fifty percent of the construction is completed each year.
- Maintenance cost = 0.1% of total construction cost
- Yearly revenue = Rs. 2,53,70143.2 from Section 6.1.2

Table 6.2: Cumulative Cost and Benefits

SN	PW of cost	PW of benefit	Cumulative cost	Cumulative benefit	Net cumulative benefit
1	80057069.534	0	80057069.534	0	-80057069.534
2	72779154.122	0	152836223.656	0	-152836223.66
3	132325.735	19060964.087	152968549.390	19060964.087	-133907585.30
4	120296.123	17328149.170	153088845.513	36389113.257	-116699732.26
5	109360.111	15752862.882	153198205.624	52141976.139	-101056229.48
6	99418.283	14320784.438	153297623.907	66462760.577	-86834863.330
7	90380.257	13018894.944	153388004.165	79481655.521	-73906348.643
8	82163.870	11835359.040	153470168.035	91317014.561	-62153153.474
9	74694.428	10759417.309	153544862.462	102076431.870	-51468430.593
10	67904.025	9781288.463	153612766.487	111857720.332	-41755046.155
11	61730.932	8892080.421	153674497.419	120749800.753	-32924696.666
12	56119.029	8083709.473	153730616.448	128833510.226	-24897106.222
13	51017.299	7348826.794	153781633.747	136182337.020	-17599296.727
14	46379.363	6680751.631	153828013.110	142863088.651	-10964924.459
15	42163.057	6073410.573	153870176.167	148936499.224	-4933676.943
16	38330.052	5521282.339	153908506.219	154457781.564	549275.345
17	34845.502	5019347.581	153943351.721	159477129.145	5533777.424
18	31677.729	4563043.256	153975029.450	164040172.401	10065142.951
19	28797.935	4148221.142	154003827.385	168188393.543	14184566.158
20	26179.941	3771110.129	154030007.326	171959503.671	17929496.345

Thus, the payback period is 15.9 years. The benefit cost ratio is 1.116. Thus, the project is economically feasible.

7. ENVIRONMENTAL STUDY

7.1 Screening

As per EPA - 2076,

- Section 3: The proponent should conduct BES/IEE/EIA of prescribed proposals.

As per EPR - 2077,

- Schedule 1,2, and 3: The proponent is required to conduct the BES/IEE/EIA of the proposals as mentioned.

This process of determining whether a proposal needs full EIA study or not is called Screening, which is the first step of EIA.

It involves examining the project's characteristics, scale, location, and potential impacts on the environment.

(३) सिँचाइको नयाँ प्रणाली अन्तर्गतः-

(क) तराई वा भित्री मधेशमा २०० देखि २,००० हेक्टरसम्मको क्षेत्र सिँचाइ गर्ने,

(ख) पहाडी उपत्यका र टारमा २५ देखि ५०० हेक्टरसम्मको क्षेत्र सिँचाइ गर्ने,

(ग) पहाडी भिरालो पाखा वा पर्वतीय क्षेत्रमा २५ देखि २०० हेक्टरसम्मको क्षेत्र सिँचाइ गर्ने,

(घ) १०० हेक्टरभन्दा बढीको लिफ्ट सिँचाइ आयोजना निर्माण गर्ने,

Hence by Screening, our project requires IEE compulsorily.

Screening, Scoping and ToR preparation take place during the Pre-feasibility stage of the project cycle. Our project deals with 'Pre-feasibility Study of the Irrigation Project', hence the need of ToR preparation.

7.2 Scoping and ToR:

As per EPR - 2077, there is no need for Scoping for IEE. The format of ToR for IEE is given below, assuming Changunarayan Municipality itself to be the proponent:

7.3 ToR Document:

Changunarayan Municipality

Office of the Municipal Executive

Sankhu, Sankharapur

Bagmati Province, Nepal

Terms of Reference (ToR)

for

Initial Environmental Examination (IEE)

for

Manohara Irrigation Project

Manohara River, Changunarayan

Baisakh, 2081

ABBREVIATIONS AND ACRONYMS

CBS	Central Bureau of Statistics
CF	Community Forest
CFUG	Community Forest Users Group
CITES	Convention on International Trade of Endangered Species of Wild Flora and Fauna
EIA	Environmental Impact Assessment
EMP	Environmental Management Plan
EPA	Environmental Protection Act
EPR	Environmental Protection Regulation
GoN	Government of Nepal
HH	Household
HIA	High Impact Area
IEE	Initial Environmental Examination
km	Kilometer
LIA	Low Impact Area
m	Meter
m^2	Square meter
m^3/s	Cubic meter per second
masl	Meter above sea level
NTFPs	Non-Timber Forest Products
RA	Rapid Appraisal
ToR	Terms of Reference

TABLE OF CONTENTS

1. **(Assumption) Pulchowk Final Year Project Group, 076BCE (004, 005, 008, 033, 037, 038)**

1.1 Proposal Proponent

(Assumption) Changunarayan Municipality is the proponent of proposal for Manohara Irrigation Project. The Name and Contact Address of the proponent is as follows:

- Changunarayan Municipality
- Office of the Municipal Executive Tel.
- E-mail: ito.changunarayanmun@gmail.com

1.2 Organization responsible for preparing the report

The proponent is responsible for carrying out the Initial Environmental Examination (IEE) for the proposed proposal.

2. **General Introduction of the Proposal**

2.1 Introduction

The irrigation project proposed for a 45.8-hectare area in the hilly region of Nepal, situated along the Manohara River in Changunarayan Municipality, represents a significant endeavor aimed at enhancing agricultural productivity and ensuring sustainable water resource management in the region. This project is designed to address the challenges of water scarcity and inefficient irrigation practices, thereby promoting agricultural development, improving livelihoods, and fostering economic growth in the local community.

Key Components:

- Location and Context: The project site is situated in the hilly terrain of Nepal, characterized by its unique topography and climatic conditions. The Manohara River, which flows through the region, serves as the primary water source for irrigation purposes.
- Infrastructure Development: The project involves the construction of irrigation canals, reservoirs, and distribution networks to efficiently channel water from the Manohara River to the agricultural fields. This infrastructure will be designed to withstand the rugged terrain and accommodate the fluctuating water levels of the river.
- Water Management Systems: Advanced water management techniques, such as drip irrigation and sprinkler systems, may be employed to optimize water usage and minimize wastage. Additionally, measures to prevent soil erosion and sedimentation in

the irrigation channels will be implemented to ensure the long-term sustainability of the project.

- Community Engagement: The success of the project relies on the active involvement and participation of the local community. Therefore, stakeholders, including farmers, local authorities, and relevant government agencies, will be consulted, and engaged throughout the planning, implementation, and monitoring phases of the project.
- Environmental Considerations: Environmental impact assessments will be conducted to evaluate the potential ecological effects of the project and mitigate any adverse impacts on the surrounding ecosystem. Efforts will be made to preserve biodiversity, protect natural habitats, and promote ecofriendly agricultural practices.
- Capacity Building and Training: To enhance the skills and knowledge of farmers and stakeholders, training programs on modern agricultural techniques, water management practices, and maintenance of irrigation infrastructure will be organized. This capacity-building initiative aims to empower local communities and ensure the sustainable management of resources.
- Monitoring and Evaluation: Continuous monitoring and evaluation mechanisms will be established to assess the performance and effectiveness of the irrigation project. Key performance indicators, such as crop yields, water usage efficiency, and socio-economic impacts, will be monitored to gauge the project's success and identify areas for improvement.

Conclusion:

The irrigation project in the hilly region of Nepal, along the Manohara River in Changunarayan Municipality, represents a transformative initiative aimed at addressing water scarcity, enhancing agricultural productivity, and improving the livelihoods of local communities. Through strategic planning, robust infrastructure development, community engagement, and sustainable practices, this project has the potential to catalyze socio-economic development and contribute to the overall prosperity and well-being of the region.

2.2 Objectives of ToR

The basic objectives of the ToR are to set up a guideline for conducting IEE of the proposed Proposal to fulfil the requirements of EPR, 2077. The objectives of the ToR are as follows:

- To identify all the adverse and beneficial impacts likely to arise due to implementation of the proposal.

- To reflect the key issues identified in the form of ToR.
- To provide guidance for the study of IEE.
- To systematize the working procedure and listing of various activities to be performed.
- To delineate the working procedure and fitting the IEE report within the policy and legal context.

Hence, in overall context, ToR will be a Guidebook or Roadmap for conducting IEE study and preparing the IEE report.

2.3 Objective of IEE Study

The objectives of the IEE study are as follows:

- To document the existing physical, biological, socio-economic, and cultural environment of the Proposal affected area.
- To identify, evaluate and classify the impacts.
- To develop mitigation measures to avoid/reduce/mitigate adverse impacts due to implementation of the Proposal.
- To develop enhancement measures to enhance the beneficial impacts due to implementation of the Proposal.
- To formulate environmental management plan
- To advise the decision makers regarding further action on development of Proposal.

3. Rationality of conducting IEE study

Study & preparation of IEE report for the Manohara Irrigation Project:

The project may have some kind of adverse impact on the environment. Primarily, due to construction activities, there is an increased risk of soil erosion. To add to this fact, there are few settlements in those hills too where slope cutting and filling are to be performed. Also, during topographic and social surveys we did in the region, it was found that though little, the fish economy was prevalent there. Due to the construction of weirs, fish migration will be significantly affected. The process of land clearing and land acquisition will also have an impact on the PAF's (Project Affected Families). Also, the facilitation of improved land use pattern, growing more productive crops will affect positively on the economic and social status of the area.

To carry out environmental study to quantify such impacts and formulate mitigation measures, guidelines were prepared. In this regard, GoN has promulgated Environment Protection Act 2076 (EPA) and subsequently, Environmental Protection Rules, 2077 (EPR) as guiding principle for conducting such mandatory studies.

Overall, the project does not fall in any restricted areas, places of cultural, historical, and archaeological importance/monuments, conservation areas, wildlife national parks, wetland areas, environmental sensitive and/or fragile area, and any other places where the law of the land prohibits any construction activities. Hence, this Proposal falls in the Schedule-2 Category of the Proposals of EPR 2077. Therefore, it requires only Initial Environmental Examination (IEE) study as per legal requirement of EPA, 2076 and EPR, 2077.

4. Data requirement and study methodology

The IEE study of proposed Proposal will be conducted according to EPA 2076 and EPR 2077 and with its subsequent amendments. The following procedures will be considered to prepare the IEE study report.

4.1 Desk study

Secondary information will be collected by reviewing relevant documents. Information related to physical environment such as geographical location and land use patterns will be obtained by using topographic maps, municipality, and district profile. The biological information of flora and fauna will be obtained from reviewing relevant articles published by government and non-government agencies. Information on local community forests will be collected from reviewing operational plan of Community Forests (CF). Additionally, protected species of the Proposal area will be tallied with the protected species' list of GoN. The socio-economic information such as demographic pattern, social service centers, religion, occupation, etc. will be obtained by reviewing documents published by Central Bureau of Statistics (CBS), district profile, municipality profile and other Proposal related documents.

Furthermore, the collected data will be categorized in terms of physical, biological, socio-economic, and cultural environment. Existing acts, rules, policies, legislation, and guidelines related to irrigation systems will be reviewed and documented for the preparation of the IEE study report.

4.2 Field Survey and Site Investigation

To meet the above objectives, the different methods for information collection such as walk through survey, inventory surveys, sampling, focused group discussion and household surveys may be used. Secondly information through published and unpublished reports, maps and photographs interpretation and GIS may also be used.

The Proponent shall collect and analyze bio-physical environmental information in the Proposal area (High Impact Area within physical coverage of 50 to 150m and Medium Impact Area within the physical coverage of about 150 to 500m. Socio-economic information will be collected and analyzed for 5km or 1.5 hour walking distance (depending upon the settlement pattern) from the boundary of the Proposal area on all sides (immediate influence area as Low Impact Area). The IEE study shall make every effort to quantify and evaluate site-specific impacts to the extent applicable in all resources within the High Impact Area.

The census shall be conducted particularly to collect data regarding households whose land or houses will be located within the Proposal area and those that are to be acquired.

4.2.1 Physical Environment

Information on physical environment shall be collected through literature, maps, and walk through surveys. The use of appropriate and calibrated equipment is encouraged to quantify information.

4.2.2 Biological Environment

Information on biological environment such as vegetation shall be collected by standard ecological methodologies shall be adopted to quantify vegetation of the forest of all categories such as disturbed, medium disturbed and undisturbed. The number of trees to be cleared for implementation of the Proposal shall be recorded by counting method or sample plot method.

Wildlife distribution and occurrence in the Proposal area shall be collected through pug marks of the wild animal, jungle trek and observation method including discussion with the local people. Fishery information shall be generated by the capture-recapture method, and by asking the local people and site observation. The Proponent may use data sheets to collect the field level information.

4.2.3 Socio-Economic Environment

Socio-economic information shall be collected by using questionnaire, checklists, observation, interviewing with the local people, focus group discussion, Participatory Rural Appraisal (PRA), discussion with district level offices, Rural Municipality, and community groups. The Proponent shall collect information on damage to infrastructures, and community services through checklists and focus group discussion. Information on land acquisition and compensation issues shall be collected through intensive household surveys, at least in the direct impact zone.

4.3 Public Notice

While preparing the IEE report, a fifteen-day public notice will be published in the national daily newspaper to inform the stakeholders about the Proposal activities. Further, the notice will be affixed in the offices of concerned Rural Municipality, DCC, schools, hospitals, health posts and other public offices located near to the Proposal area requesting them to provide their written opinions, suggestions, and comments about the Proposal implementation activities.

4.4 Public Consultation and Recommendation Letters

During data collection for IEE study, public consultation will be carried out at appropriate places considering the views, suggestions, and comments on the Proposal implementation. Public consultation will also be carried out through focus group discussions with targeted groups such as women, key informants, and other groups.

Recommendation of affected Rural Municipality as per EPR will be collected and attached in the IEE report. Also, if the Proposal requires forest land of community forest to be cleared, the recommendations from these affected community forest shall be collected and attached in the IEE report.

If the Proposal requires land from government forest/community forest or trees are needed to be cut down from them, then comments and suggestions from District Forest Office and affected community forests should be attached in the IEE report along with their recommendation letters.

4.5 Data Analysis and Impact Prediction

After collecting baseline field data from the Proposal area on physical, biological, socio-economic and cultural environment, these will be analyzed and interpreted as per EPR 2077. The analysis shall be made on the nature, magnitude, duration, and extent of the impacts due to construction and operation activities of the Proposal. Mitigation measures and monitoring plans will be described for each impact.

Data collection for IEE study as aforementioned methods will first be compiled and classified to relevant components (Physical, Cultural, Socio-Economic, and Biological) for convenience and to avoid mismatches. Computer software with usage of expert's opinion will be primary tool to process data coming from physical, socio economic and cultural domain. While categorizing the impacts into significant and insignificant depending on the nature of impact, the following aspects shall be made to each of the impact:

4.5.1 Nature: Direct or Indirect

4.5.2 Magnitude: High, Moderate, Low

4.5.3 Extent: Site specific, Local and National

4.5.4 Timing: Short term, Medium-term, Long-term

4.5.5 Duration: Temporary, Permanent

4.5.6 Impact quality: Beneficial, Negative

Regarding Impacts, after determining baseline setting, Impacts prediction, analysis and Evaluation need to be carried out.

4.5.7 For Impact Identification the following methods could be utilized. Simple Formats of each are provided here:

4.5.7.1 Simple Matrix

Environmental components	**Project Activities**							
	Plant Construction	Pesticide and Fertilizer Use	Raw materials transport	Water Intake	Solid Waste	Effluent Discharge	Emissions	Employment
Surface Water Quality		X			X	X		X
Surface Water Hydrology				X				
Air Quality			X				X	
Fisheries		X				X		
Terrestrial Wildlife Habitat	X							
Terrestrial Wildlife	X							
Land Use Pattern								
Highways/Railways			X					
Water Supply						X		
Agriculture		X				X		X
Housing								
Health					X	X	X	
Socioeconomic								X

Fig: Simple Matrix example

4.5.7.2 Leopold Matrix

When an impact is anticipated, the matrix is marked with diagonal line in the interaction box. Secondly, interaction is defined by magnitude in the upper and importance in the lower section.

Proposed Action / Resources	Immigration of Labor	Dam Construction	Transmission Line	Reservoir Filling	Heavy Metal Discharge	Growth of Aquatic Weeds	Relocation of Inhabitants
Health	5 / 8	4 / 6		5 / 8	4 / 7	6 / 6	
Spawning of Fish		3 / 4		3 / 6	3 / 7	5 / 5	
Archeological Artefacts	4 / 6			8 / 8			
Tourism			7 / 6	7 / 6			
Downstream water pollution		7 / 7		7 / 8	2 / 4		
Social and Economic Aspects							8 / 7
Forestry		4 / 2					
Fishery		2 / 5			2 / 5		
Navigation				6 / 5			
Aquatic Plants				6 / 5			
Leopold Method	9 / 14	20 / 24	7 / 6	42 / 47	11 / 23	11 / 11	8 / 7

Fig: Leopold Matrix example

4.5.7.3 Network Diagrams

They begin with a list of project activities and generate a network of cause-condition-effects (i.e., chain of Events). Networks are usually in the form of flow charts or radiation diagrams, as illustrated below:

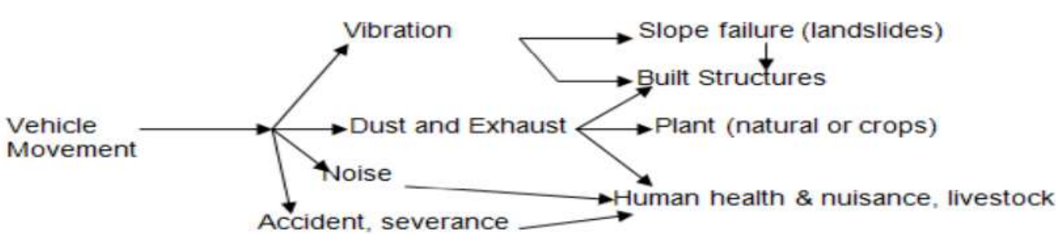


Fig: Simple Tree Network Diagram

4.5.7.4 Questionnaire

Historical/Cultural Resources

Describe any historical, archaeological, or cultural sites in the vicinity of the proposed project; note any sites included on the National Register of Historic Places. ☐ None

Would construction or operational activities planned under the proposed project disturb any historical, archaeological, or cultural sites? ☐ No planned construction ☐ No historic sites ☐ Yes (describe) ☐ No Impact (discuss)

Has the State Historic Preservation Office been contacted with regard to this project? ☐ No ☐ Yes (describe)

Would the proposed project interfere with visual resources (e.g., eliminate scenic views) or alter the present landscape?
☐ No ☐ Yes (describe)

Fig: Format of questionnaires

4.5.7.5 Overlay Mapping and Geographic Information Systems (GIS)

For identifying the spatial distribution of the impacts and in identifying where the cumulative impacts and impact interactions may occur because of the project.

4.5.7.6 Impact Prediction

Once all the important impacts have been identified, their potential sizes and characteristics can be predicted. Impact identification refers to the analysis of potential impacts in the future. For the impact prediction the methods that can be used are simply listed as:

4.5.7.6.1 Mathematical model (Deterministic and Stochastic)
4.5.7.6.2 Statistical Model
4.5.7.6.3 Geographic Model
4.5.7.6.4 Field and Laboratory Experimental Model
4.5.7.6.5 Physical Model
4.5.7.6.6 Expert Judgement
4.5.7.6.7 Case Studies

4.5.7.7 Impact Evaluation Techniques

Once the impacts are predicted, they are evaluated to determine its significance. Evaluation of impacts should be based on comparing the values against the set standards. They may be defined in legislation and procedures. Two major steps engage in it:

Step 1: Evaluation of the significance of ‘as predicted’ impacts to define the requirements for mitigation and other remedial impacts.

Step 2: Evaluation of the significance of the ‘residual’ impacts i.e., after mitigation measures are considered.

Importantly, this test is also the critical measure of whether the proposal is likely to cause significant impacts or not.

5. Existing environmental condition (Environmental Setting/ Baseline Condition)

The environmental study team also conducted a field visit to the proposed area in Manohara River before tor preparation of IEE study and following observations were made on the environment of the proposal.

5.1 Physical Environment

- Topography:
- Climate and Hydrology:
- Geology:
- Soil Erosion and Landslide:
- Land Use:
- Air and Water quality:
- Noise Quality:

5.2 Biological Environment

The proposed area belongs to the free zone. There is no national park, wildlife sanctuary, buffer zone or conservation area in or around the proposed area.

- Forest and Flora
- Forest type, community forest, plant/tree species
- Plant Species of Medicinal and Ethno-Botanical
- Values Fauna
- Fishes
- Rare, Endangered and Protected Species of Flora and Fauna

5.3 Socio-economic and Cultural Environment

- Population
- Ethnicity and Settlements
- Occupation

- Local Economy Literacy
- Health and Sanitation
- Public Facilities

6. Policies, Laws, and Manuals to be undertaken into account while preparing the report

The policy documents, acts, regulations, and guidelines as indicated below shall be reviewed and relevant provisions shall be referred to in the IEE report. A concrete recommendation shall be made in the IEE report as to whether the Proposal could be implemented after reviewing acts and regulations. Any legal provision that surfaces against proposal implementation will be investigated and put forth with recommendations, if felt necessary will be made in the IEE report. The following documents will be considered during IEE report preparation.

6.1 Plans, Policies and Strategies:

- Three Years Interim Plan, 2008
- Forestry Sector Master Plan, 1998
- Water Resource Strategy, 2002
- Nepal Biodiversity Strategy, 2002
- Nepal Environment Policy and Action Plan, 1993 and 1998
- Revised Forest Policy, 2000

6.2 Acts and Regulations

- Interim Constitution of Nepal, 2007
- Environment Protection Act, 2076
- Environment Protection Rules, 2077
- Plant Protection Act, 1972
- National Parks and Wildlife Conservation Act, 1973
- Soil and Watershed Conservation Act, 1982
- Forest Act, 1993
- Forest Regulation, 1995
- Labor Act, 1991
- Land Acquisition Act, 1977
- Local Self-Governance Act, 1998
- Local Self-Governance Regulation,1999

- Industrial Enterprises Act, 1993
- Export, Import Act, 1956
- Mines and Minerals Act, 1993
- Aquatic Animals Protection Act, 1993
- Water Resources Act, 1993
- Water Resources Regulations, 1993
- Solid Waste Management and Resource Mobilization Act, 1987 and Regulations, 1989
- Wildlife Reserve Rule, 1977

6.3 Guidelines and Manuals

- National Environmental Impact Assessment Guideline, 1993
- EIA Guideline for Forestry Sector, 1995
- Environmental Management Guidelines (Road), 1997
- Community Forest Guidelines, 2001
- A handbook on Licensing and Environmental Assessment Process for Hydropower process in Nepal, 2006
- Working Procedures of Forest, 2007
- Guidelines on Land-use for Other Purposes, 2063 B.S.

6.4 Conventions

- Convention on Biological Diversity, 1992
- CITES, 1973
- Convention No 169 concerning Indigenous and tribal people in Independent Countries

Any significant conflicts and issues related to the environment and others will be identified and recommendations will be based on the above-mentioned policies, rules, and regulations. The proponent will describe the pertinent regulations and standards governing environmental quality, health and safety, protection of the sensitive areas, protection of the endangered species, land use control etc. At district, Rural Municipality/Municipality, and ward level. The proponent shall also review the responsibilities of the local, district, and central level institutions and recommend steps to ensure necessary coordination during the Proposal implementation.

7. Preparation of the report

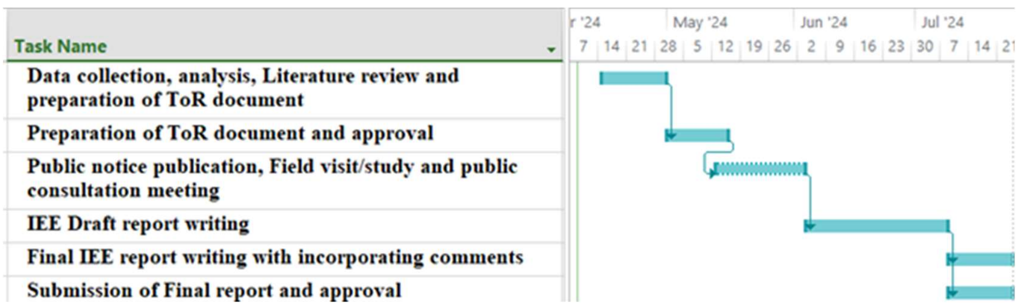
The IEE report shall be prepared in consonance with the contents given on Schedule-5 of the EPR 1997. The report shall document systematically the baseline data on physical, biological as well as socio-economic and cultural environment.

7.1 Time Schedule, Budget, and Study Team

The estimated study time schedule, cost and team composition is as follows.

7.1.1 Time Schedule

The estimated time schedule for completion of the IEE study is 1.5 months as given in Table 7.1. Here, the time schedule for approval of ToR and IEE report is not included.



Task Name
Data collection, analysis, Literature review and preparation of ToR document
Preparation of ToR document and approval
Public notice publication, Field visit/study and public consultation meeting
IEE Draft report writing
Final IEE report writing with incorporating comments
Submission of Final report and approval

Table 7.1: Time Schedule for IEE Study

7.1.2 Team Composition

Environmental Specialist - Team Leader

Geologist/Hydrologist - Team Member

Sociologist - Team Member

Civil Engineer - Team Member

GIS Expert - Team Member

Other Support Staffs

8. Specific impacts on the implementation of proposal on the environment

After the field visit, the study team will presume the environmental impacts to be affected by the implementation of the Proposal. These impacts will be categorized as site specific, local, and regional in terms of extent. During the IEE study, these issues will be dealt with thoroughly. The impacts will be characterized as low, medium, high in terms of magnitude and long term, medium term, and short term in terms of duration.

8.1 Physical Environment

➢ Impact due to site clearance

- Change in Topography, Geomorphology (riverbank and embankment stability, disturbance to fragile slope)
- Change in land use (loss or degradation of productive land)
- Impact due to spoil handling and disposal
- Impact on slope stability, sedimentation, soil erosion, bank cutting and flooding.
- Change in River hydrology and morphology (meandering and alteration of river flow, change/deepening in riverbed) other natural resources e.g., water spring, well etc.
- Impact on water quality in downstream and water use system
- Impact due to noise and vibration
- Impact due to stockpiling of construction materials
- Impact on air quality due to dust emission and vehicle movement
- Access road related issues including impact due to heavily loaded vehicles for transportation of construction materials.
- Decline in aesthetic value and visual impact.
- Any other issues likely to arise during IEE study.

8.2 Biological environment

- Impacts on vegetation due to clearance for the Proposal activities.
- Impacts on protected species of flora and fauna.
- Any other issues likely to arise during IEE study including biodiversity aspects.
- Impacts on aquatic habitat.
- Impacts on forests, forest products and vegetation of surrounding area and dependent wildlife due to increased human activities.
- Impacts on wildlife and birds due to destruction of its habitat.
- Impacts on wildlife, bird, and its habitat due to illegal hunting, poaching
- Impacts on protected species of flora and fauna.
- Any other issues likely to arise during IEE study.

8.3 Socio-economic and Cultural Environment

- Land acquisition, compensation issues and people’s displacement.
- Impact on occupational health, safety, and sanitation.
- Any other issues likely to arise during IEE study.
- Impact on access road, people’s movement, and traffic management due to operation activities

- Loss or damage to private land and properties such as houses, farm sheds, and other structures, crops, and fodder/fruit trees
- Impact on occupational health, safety and sanitation and solid waste management
- Loss or degradation of farmland and impact on agriculture productivity and livestock
- Pressure on existing infrastructure facilities like health post, education, communication, water supply and sanitation facilities, irrigation system, foot trail, trail bridges etc.
- Social conflict
- Change in social structure, traditional and cultural practices, rituals of the rural people due to exposition to outside workforce.
- Impact on law and order in the Proposal area due to large work force.
- Impact due to proposal site infrastructure development
- Impact on religious, cultural, and historical sites (including cremation site)
- Impact on gender and vulnerable groups (including extension/evacuation of squatter settlement)
- Migration of laborers
- Impact on livelihood of local people (fishermen etc.)
- Impact on nearby human settlements
- Impact on use of other natural resources
- Conflict in revenue/income sharing mechanism.
- Other public concern issues
- Any other issues likely to arise during IEE study.

8.4 Beneficial impacts

- Employment opportunity for skilled, semi-skilled and unskilled local people
- Boost up of local economy.
- Impact on gender and vulnerable groups.
- Business opportunity to local people.
- Revenue generation to local bodies.
- Community development and Potential improvement of public facilities
- Benefit to the local people due to the Proposal implementation.

9. Alternatives for the implementation of the proposal on the environment

During IEE study of the Proposal, an alternative analysis of the Proposal implementation shall be performed considering various aspects like proposal site, operation procedure, technology, and schedule. Alternative analysis shall be performed on the following aspects:

- No proposal options.
- Proposal scale alternatives
- Alternative Proposal sites
- Alternative technology and operation
- Alternative resources, procedure, and schedule

10. Mitigation and Enhancement Measures

Preventive, corrective, and compensatory mitigation measures as identified and anticipated in the study will be included in the IEE report. Each of the identified adverse impacts will be evaluated in detail and cost-effective mitigation measures will be suggested to minimize the adverse impacts. Social enhancement measures shall be proposed to enhance beneficial impacts due to implementation of the Proposal. The mitigation measures will be categorized in terms of physical, biological, socio-economic, and cultural environment during proposal construction and operation phase. Public Suggestions, comments shall be strictly followed to propose mitigation measures regarding impacts on forest and vegetation. The detail IEE report shall include the following:

- Mitigation and enhancement measures during construction and operation phases for each environment; agencies responsible for implementation and construction
- Mitigation and enhancement cost
- Organizational set up to conduct the mitigation and enhancement measures, and
- Preparation of Environment Impact Mitigation Matrix

The following considerations shall be made while proposing the cost for environmental protection measures.

- Appropriate compensation for the loss of assets.
- Appropriate mitigation measures to minimize the adverse impacts on biophysical, socioeconomic, and cultural environment.
- Health and sanitation measures.

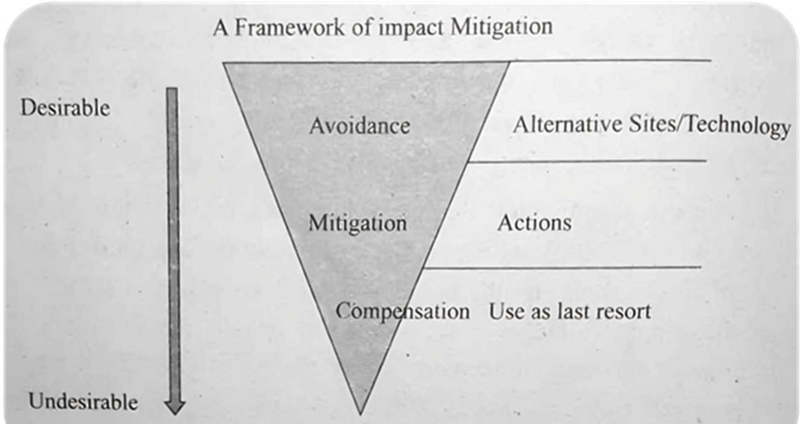


Fig. 6.1: *Main Elements of Mitigation*

Summary of Cost Benefit assessment shall be given which shall include the followings:

- Cost for Environment Mitigation Measures
- Cost for Enhancement Measures,
- Cost for other Social Support Programs,
- Cost for Environmental Monitoring,
- Total Proposal Income and revenue generation, and
- Percentage of total Environmental Cost to the Total Proposal Income and revenue generation

11. Matters to be monitored while implementing the proposal

"Environmental Monitoring Plan" shall be categorized in terms of baseline impact, and compliance monitoring plans. Impact and Compliance Monitoring plan shall be further categorized in terms of construction and operation phases. The Environmental Monitoring Plan will be prepared in terms of physical, biological, socio-economic, and cultural environment. The proposed plan will be worked out about the cost of impact and cost of monitoring activities to conduct the proposed activities at least for 2 years of proposal operation period. Estimated budget for EMP implementation in IEE report of the proposal shall be reported as presented in table 11.1.

Table 11.1: Sample to show summary of cost of EMP implementation in IEE report

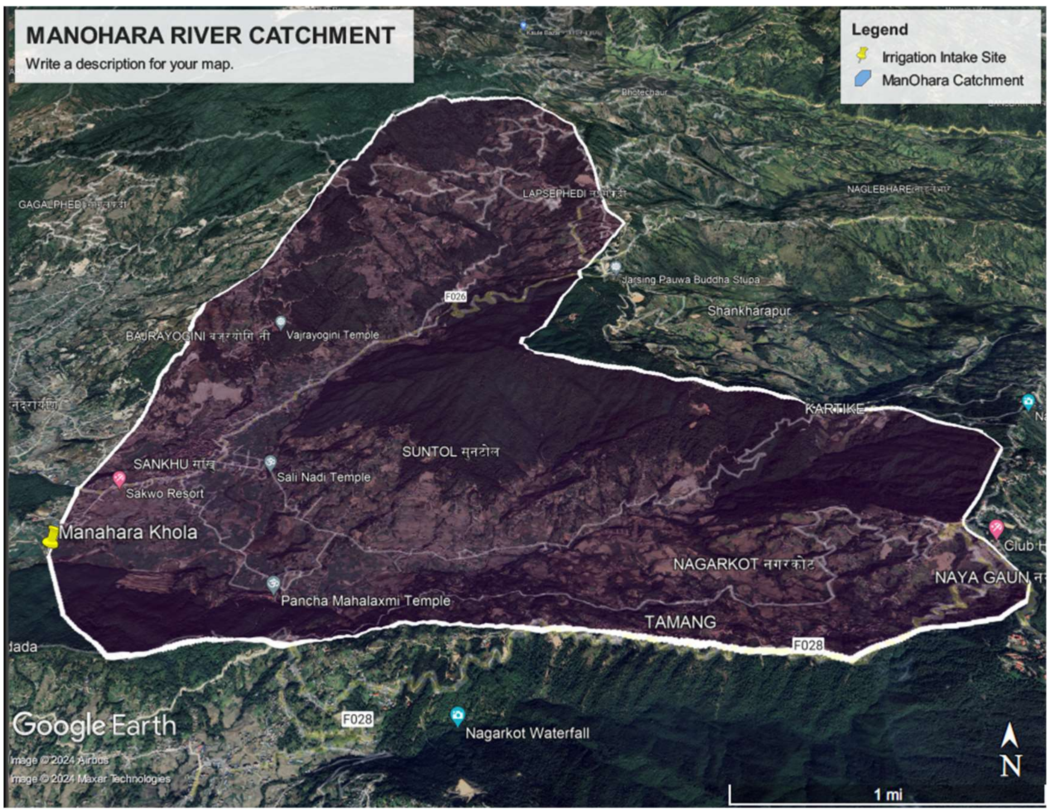
SN	Activities	Budget Allocation (NRS)	Remarks
1	Environmental Protection		
2	Social Enhancement Measures in the Proposed Area		
3	Environmental Monitoring		
	Total		

12. Other necessary matters

- Report format and deliverables: References, annexes, maps, photos, tables, charts, and questionnaire as relevant and applicable shall be included in the report.

ANNEX – I

Proposed Location Map



8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Based on the pre-feasibility study of the Manohara Irrigation project, following are the conclusions:

- In a 45.8 ha command area, irrigation water requirement research was conducted to determine the canal discharge by using an appropriate cropping pattern, such as monsoon paddy, summer and winter vegetables, potato, and maize. To meet the need, research and calculation of river discharge were carried out. Additionally, the maximum amount of irrigation needed for a specific half-month resulted in a canal discharge of 0.241 m^3/s.
- Cost of Project was estimated using unit rate method which was estimated to 17.6 crore Nepalese Rupees and annual revenue generated from design system is 2.537 crore Nepalese Rupees. For the span of 20 years as evaluation period, the B/C ratio is 1.116 > 1. The payback period for the project is 15.9 years. Thus, from all analysis criteria it is concluded that this project is financially feasible.

8.2 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

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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

A.1.1: Gumbel's 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's method.

Table A.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.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.3: Mean and S.D. Calculation

SN	Extreme (Xi)	$(X-Xi)^2$	SN	Extreme (Xi)	$(X-Xi)^2$
1	44	1256.097	21	91	133.600
2	90	111.483	22	98	344.419
3	46	1118.331	23	65.5	194.364
4	46	1118.331	24	105	653.239
5	44	1256.097	25	92	157.717
6	40.8	1493.163	26	82.5	9.355
7	40.8	1493.163	27	94	211.951

8	126.8	2242.831	28	72.2	52.439
9	90	111.483	29	100.4	439.260
10	80	0.312	30	72	55.375
11	67.5	142.599	31	48.2	976.029
12	60	377.970	32	179.5	10011.711
13	102	508.888	33	88.4	80.255
14	85	30.897	34	96	274.185
15	80.5	1.120	35	95.6	261.098
16	80	0.312	36	80	0.312
17	95.5	257.877	37	80.5	1.120
18	65	208.556	38	72	55.375
19	82	6.546	39	64.4	226.246
20	92	157.717	40	60	377.970
			41	62	304.205
Σxi =	3257.1		$\Sigma(X\text{-}Xi)^2$ =	26714.000	
n =	41				
Yn =	0.5436		Sn =	1.1413	
x̄ =	79.441		σ =	25.843	

Table A.4: Gumbel Analysis

SN	Extreme Rainfall (xi)	Rainfall (Desc. order)	Year (m)	T = (N+1)/m	yT	K	xT
1	44	179.5	1	42.000	3.726	2.788	151.493
2	90	126.8	2	21.000	3.020	2.170	135.520
3	46	105	3	14.000	2.602	1.804	126.056
4	46	102	4	10.500	2.302	1.540	119.252
5	44	100.4	5	8.400	2.066	1.334	113.903
6	40.8	98	6	7.000	1.870	1.162	109.472
7	40.8	96	7	6.000	1.702	1.015	105.671
8	126.8	95.6	8	5.250	1.554	0.886	102.330
9	90	95.5	9	4.667	1.422	0.770	99.338
10	80	94	10	4.200	1.302	0.665	96.619
11	67.5	92	11	3.818	1.192	0.568	94.118
12	60	92	12	3.500	1.089	0.478	91.797
13	102	91	13	3.231	0.993	0.394	89.623
14	85	90	14	3.000	0.903	0.315	87.573
15	80.5	90	15	2.800	0.817	0.239	85.628
16	80	88.4	16	2.625	0.735	0.168	83.772

17	95.5	85	17	2.471	0.656	0.099	81.992
18	65	82.5	18	2.333	0.581	0.032	80.277
19	82	82	19	2.211	0.507	-0.032	78.617
20	92	80.5	20	2.100	0.436	-0.094	77.005
21	91	80.5	21	2.000	0.367	-0.155	75.432
22	98	80	22	1.909	0.298	-0.215	73.891
23	65.5	80	23	1.826	0.232	-0.273	72.378
24	105	80	24	1.750	0.166	-0.331	70.885
25	92	72.2	25	1.680	0.100	-0.388	69.406
26	82.5	72	26	1.615	0.036	-0.445	67.937
27	94	72	27	1.556	-0.029	-0.502	66.472
28	72.2	67.5	28	1.500	-0.094	-0.559	65.003
29	100.4	65.5	29	1.448	-0.159	-0.616	63.525
30	72	65	30	1.400	-0.225	-0.674	62.030
31	48.2	64.4	31	1.355	-0.293	-0.733	60.509
32	179.5	62	32	1.313	-0.361	-0.793	58.953
33	88.4	60	33	1.273	-0.432	-0.855	57.349
34	96	60	34	1.235	-0.506	-0.919	55.681
35	95.6	48.2	35	1.200	-0.583	-0.987	53.927
36	80	46	36	1.167	-0.666	-1.060	52.058
37	80.5	46	37	1.135	-0.755	-1.138	50.030
38	72	44	38	1.105	-0.855	-1.225	47.773
39	64.4	44	39	1.077	-0.970	-1.327	45.159
40	60	40.8	40	1.050	-1.113	-1.452	41.923
41	62	40.8	41	1.024	-1.318	-1.632	37.278

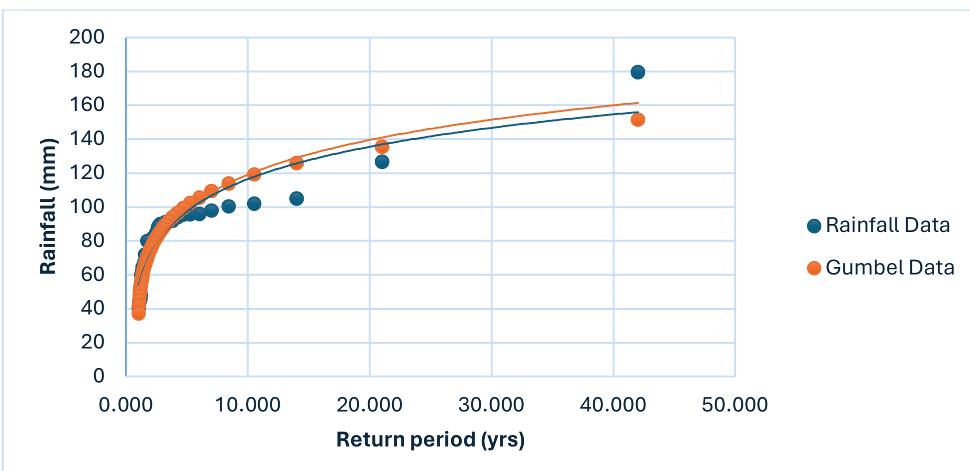

Fig A.1: Graph for Gumbel's Method Analysis

Table A.5: Correlation Calculation for Gumbel's Method

T	yT	K	xT
10	2.250	1.495	118.088301
20	2.970	2.126	134.387578
33	3.481	2.574	145.957531
50	3.902	2.943	155.48532
100	4.600	3.554	171.295108
200	5.296	4.164	187.04721
300	5.702	4.520	196.247211
500	6.214	4.968	207.829116

A.1.2: Log Normal Method

Table A.6: Log Normal Analysis

SN	Extreme Rainfall (xi)	zi = log(xi)	$(zi\text{-}z)^2$	$(zi\text{-}z)^3$	T = (N+1)/m	P = 1/T
1	44	1.643	0.05531	-0.01301	42.000	0.024
2	90	1.954	0.00572	0.00043	21.000	0.048
3	46	1.663	0.04660	-0.01006	14.000	0.071
4	46	1.663	0.04660	-0.01006	10.500	0.095
5	44	1.643	0.05531	-0.01301	8.400	0.119
6	40.8	1.611	0.07181	-0.01924	7.000	0.143
7	40.8	1.611	0.07181	-0.01924	6.000	0.167
8	126.8	2.103	0.05039	0.01131	5.250	0.190
9	90	1.954	0.00572	0.00043	4.667	0.214
10	80	1.903	0.00060	0.00001	4.200	0.238
11	67.5	1.829	0.00243	-0.00012	3.818	0.262
12	60	1.778	0.01010	-0.00101	3.500	0.286
13	102	2.009	0.01689	0.00220	3.231	0.310
14	85	1.929	0.00258	0.00013	3.000	0.333
15	80.5	1.906	0.00074	0.00002	2.800	0.357
16	80	1.903	0.00060	0.00001	2.625	0.381
17	95.5	1.980	0.01028	0.00104	2.471	0.405
18	65	1.813	0.00432	-0.00028	2.333	0.429
19	82	1.914	0.00124	0.00004	2.211	0.452
20	92	1.964	0.00725	0.00062	2.100	0.476

21	91	1.959	0.00647	0.00052	2.000	0.500
22	98	1.991	0.01268	0.00143	1.909	0.524
23	65.5	1.816	0.00389	-0.00024	1.826	0.548
24	105	2.021	0.02032	0.00290	1.750	0.571
25	92	1.964	0.00725	0.00062	1.680	0.595
26	82.5	1.916	0.00143	0.00005	1.615	0.619
27	94	1.973	0.00893	0.00084	1.556	0.643
28	72.2	1.859	0.00040	-0.00001	1.500	0.667
29	100.4	2.002	0.01515	0.00187	1.448	0.690
30	72	1.857	0.00045	-0.00001	1.400	0.714
31	48.2	1.683	0.03825	-0.00748	1.355	0.738
32	179.5	2.254	0.14095	0.05292	1.313	0.762
33	88.4	1.946	0.00460	0.00031	1.273	0.786
34	96	1.982	0.01074	0.00111	1.235	0.810
35	95.6	1.980	0.01037	0.00106	1.200	0.833
36	80	1.903	0.00060	0.00001	1.167	0.857
37	80.5	1.906	0.00074	0.00002	1.135	0.881
38	72	1.857	0.00045	-0.00001	1.105	0.905
39	64.4	1.809	0.00486	-0.00034	1.077	0.929
40	60	1.778	0.01010	-0.00101	1.050	0.952
41	62	1.792	0.00744	-0.00064	1.024	0.976
Sum	**3257.1**	**77.024**	**0.77237**	**-0.01587**		
Mean (z)=	1.879	**Std Dev (s)=**	0.139	**Cs=**	0.000	

Table A.7: Log Normal correlation Analysis

Recurrence Intervals	kz	kzs	x = z + kzs	antilog(x)
2	0.000	0.000000	1.879	75.619
10	1.282	0.178144	2.057	113.966
25	1.751	0.243315	2.122	132.418
50	2.054	0.285419	2.164	145.899
100	2.326	0.323215	2.202	159.165
200	2.576	0.357955	2.237	172.420
1000	3.090	0.429379	2.308	203.241

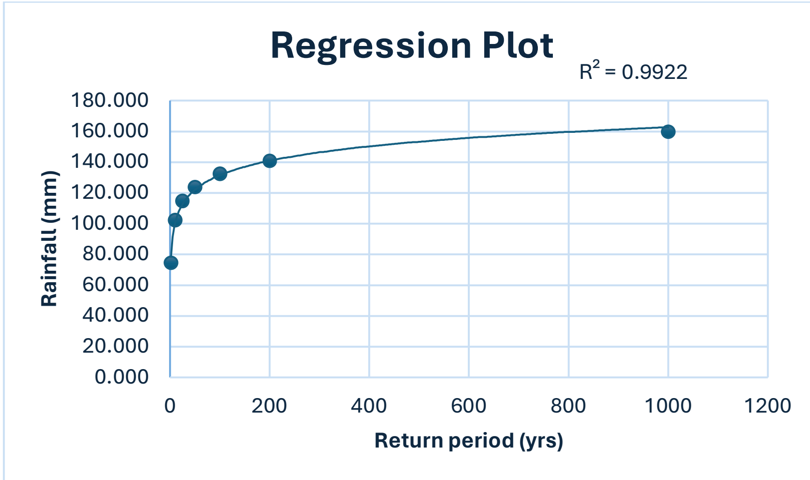

Figure A.2: Graph of Log Normal values for different return periods

A.1.3: Log Pearson III Method

Table A.8: Log Pearson III Analysis

SN	Extreme Rainfall (xi)	zi = log(xi)	$(zi-z)^2$	$(zi-z)^3$	T = (N+1)/m	P = 1/T
1	44	1.643	0.05531	-0.01301	42.000	0.024
2	90	1.954	0.00572	0.00043	21.000	0.048
3	46	1.663	0.04660	-0.01006	14.000	0.071
4	46	1.663	0.04660	-0.01006	10.500	0.095
5	44	1.643	0.05531	-0.01301	8.400	0.119
6	40.8	1.611	0.07181	-0.01924	7.000	0.143
7	40.8	1.611	0.07181	-0.01924	6.000	0.167
8	126.8	2.103	0.05039	0.01131	5.250	0.190
9	90	1.954	0.00572	0.00043	4.667	0.214
10	80	1.903	0.00060	0.00001	4.200	0.238
11	67.5	1.829	0.00243	-0.00012	3.818	0.262
12	60	1.778	0.01010	-0.00101	3.500	0.286
13	102	2.009	0.01689	0.00220	3.231	0.310
14	85	1.929	0.00258	0.00013	3.000	0.333
15	80.5	1.906	0.00074	0.00002	2.800	0.357
16	80	1.903	0.00060	0.00001	2.625	0.381
17	95.5	1.980	0.01028	0.00104	2.471	0.405

18	65	1.813	0.00432	-0.00028	2.333	0.429
19	82	1.914	0.00124	0.00004	2.211	0.452
20	92	1.964	0.00725	0.00062	2.100	0.476
21	91	1.959	0.00647	0.00052	2.000	0.500
22	98	1.991	0.01268	0.00143	1.909	0.524
23	65.5	1.816	0.00389	-0.00024	1.826	0.548
24	105	2.021	0.02032	0.00290	1.750	0.571
25	92	1.964	0.00725	0.00062	1.680	0.595
26	82.5	1.916	0.00143	0.00005	1.615	0.619
27	94	1.973	0.00893	0.00084	1.556	0.643
28	72.2	1.859	0.00040	-0.00001	1.500	0.667
29	100.4	2.002	0.01515	0.00187	1.448	0.690
30	72	1.857	0.00045	-0.00001	1.400	0.714
31	48.2	1.683	0.03825	-0.00748	1.355	0.738
32	179.5	2.254	0.14095	0.05292	1.313	0.762
33	88.4	1.946	0.00460	0.00031	1.273	0.786
34	96	1.982	0.01074	0.00111	1.235	0.810
35	95.6	1.980	0.01037	0.00106	1.200	0.833
36	80	1.903	0.00060	0.00001	1.167	0.857
37	80.5	1.906	0.00074	0.00002	1.135	0.881
38	72	1.857	0.00045	-0.00001	1.105	0.905
39	64.4	1.809	0.00486	-0.00034	1.077	0.929
40	60	1.778	0.01010	-0.00101	1.050	0.952
41	62	1.792	0.00744	-0.00064	1.024	0.976
Sum	**3257.1**	**77.024**	**0.77237**	**-0.01587**		
Mean (z)=	1.879	**Std Dev (s)=**	0.139	**Cs=**	-0.155	

Table A.9: Log Pearson III correlation calculation

Recurrence Intervals	kz	kzs	x = z + kzs	antilog(x)
2	0.026	0.003585	1.882	76.246
10	1.263	0.175559	2.054	113.290
25	1.696	0.235700	2.114	130.116
50	1.970	0.273712	2.152	142.018
100	2.211	0.307277	2.186	153.430
200	2.430	0.337709	2.216	164.566
1000	2.873	0.399225	2.278	189.608

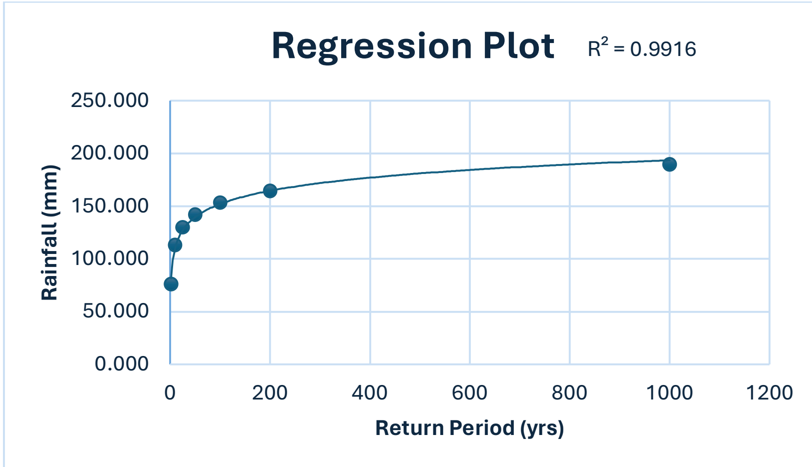

Fig A.3: Log Pearson III rainfall values for different return periods

A.1.4: Probability distribution of values

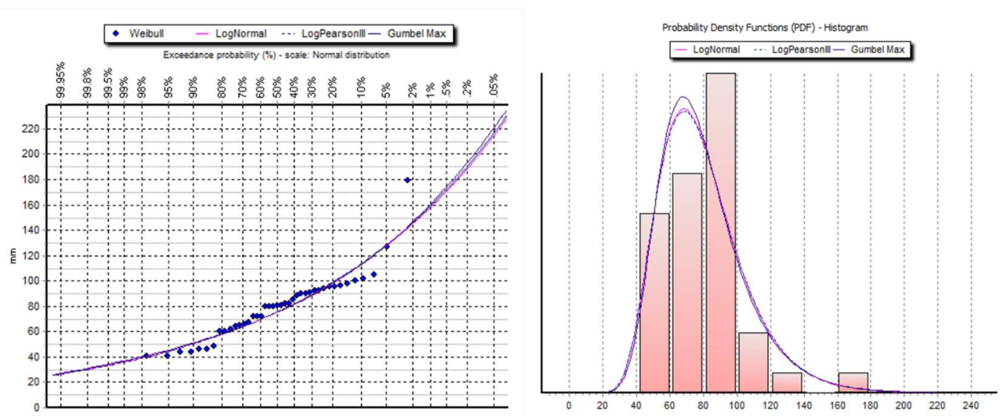

Fig A.4: Exceedance probability chart and PDF histogram for Sankhu

A.1.5: χ-Square and K-S test

Table A.10: χ-Square and K-S test for Sankhu station

Kolmogorov-Smirnov test for: All data	a = 1%	a = 5%	a = 10%	Attained a	Dmax
LogNormal	ACCEPT	ACCEPT	ACCEPT	34.70350%	0.14312
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	36.22080%	0.14128
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	28.93110%	0.15062

X-Square test for: All data	a = 1%	a = 5%	a = 10%	Attained a	Pearson
LogNormal	ACCEPT	ACCEPT	ACCEPT	2.68060%	9.19512
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	1.00764%	9.19512
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	2.68060%	9.19512

Since the r^2 value for Log Normal is higher for Sankhu (0.9973), we select the Log Normal values.

A.2 Nagarkot Station

Frequency analysis of maximum rainfall in Nagarkot station is also done in the same way as above, i.e. Gumbel's, Log Pearson III, and Log Normal methods. Then, the best fit curve is selected in the second step.

A.2.1: χ-Square and K-S test

Table A.11: χ-Square and K-S test for Nagarkot station

Kolmogorov-Smirnov test for: All data	a = 5%	a = 10%	Attained a	Dmax
LogNormal	ACCEPT	ACCEPT	66.42630%	0.11088
Log Pearson III	ACCEPT	ACCEPT	98.42860%	0.06891
EV1-Max (Gumbel)	ACCEPT	ACCEPT	88.13320%	0.09124

X-Square test for: All data	a = 5%	a = 10%	Attained a	Pearson
LogNormal	ACCEPT	ACCEPT	16.47910%	5.09756
Log Pearson III	ACCEPT	ACCEPT	29.17940%	2.46341
EV1-Max (Gumbel)	ACCEPT	ACCEPT	23.87160%	4.21951

Since the r^2 value for Log Normal is higher for Nagarkot (0.9904), we select the Log Normal values.

A.2.2: Presentation of maximum daily rainfall

Table A.12: Maximum daily rainfall for Nagarkot station

Recurrence Intervals	kz	kzs	x = z + kzs	antilog(x)
2	-0.106	-0.011359	1.861	72.638
10	1.330	0.142568	2.015	103.536
25	1.950	0.209072	2.082	120.669
50	2.379	0.254971	2.127	134.120
100	2.783	0.298341	2.171	148.205
200	3.169	0.339719	2.212	163.020
1000	4.019	0.430845	2.303	201.079

A.3 Changunarayan Station

Frequency analysis of maximum rainfall in Changunarayan station is also done in the same way as above, i.e. Gumbel's, Log Pearson III, and Log Normal methods. Then, the best fit curve is selected in the second step.

A.3.1: χ-Square and K-S test

Table A.13: χ-Square and K-S test for Changunarayan station

Kolmogorov-Smirnov test for: All data	a = 1%	a = 5%	a = 10%	Attained a	Dmax
LogNormal	ACCEPT	ACCEPT	ACCEPT	99.22360%	0.06615
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	88.76830%	0.09045
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	98.47780%	0.07038

X-Square test for: All data	a = 5%	a = 10%	Attained a	Pearson
LogNormal	ACCEPT	ACCEPT	16.70480%	3.57895
Log Pearson III	ACCEPT	ACCEPT	40.81410%	0.68421
EV1-Max (Gumbel)	ACCEPT	ACCEPT	21.73350%	3.05263

Since the r^2 value for Log Pearson III is higher for Changunarayan (0.997), we select the Log Pearson III values.

A.3.2: Presentation of maximum daily rainfall

Table A.14: Maximum daily rainfall for Changunarayan station

Recurrence Intervals	**kz**	**kzs**	**x = z + kzs**	**antilog(x)**
2	0.000	0.00000	1.951	89.239
10	1.282	0.12717	2.078	119.597
25	1.751	0.17369	2.124	133.120
50	2.054	0.20374	2.154	142.659
100	2.326	0.23072	2.181	151.803
200	2.576	0.25552	2.206	160.723
1000	3.090	0.30651	2.257	180.744

APPENDIX B

METEOROLOGICAL DATA FOR POTENTIAL EVAPOTRANSPIRATION

TABLE OF CONTENTS

LIST OF TABLES

B.1: Temperature and Wind

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

Day	Tmax	Tmin	Wind (m/s)
1	12.8	2.5	5.400
2	12.7	2.6	2.050
3	12.7	2.8	1.850
4	13.0	2.9	1.700
5	12.5	3.0	1.500
6	12.9	2.3	1.200
7	13.3	2.4	2.150
8	12.9	2.9	1.350
9	12.6	2.5	1.900
10	12.8	2.6	2.250
11	13.0	2.6	4.700
12	13.2	3.0	3.400
13	12.4	3.1	2.550
14	12.3	2.6	1.650
15	12.0	2.6	1.800
16	12.2	2.6	1.150
17	12.3	2.7	2.150
18	12.7	2.8	1.250
19	12.4	2.9	3.000
20	12.9	2.8	2.700
21	12.7	2.8	1.300
22	12.4	2.8	2.500
23	12.3	2.9	1.900
24	12.4	2.6	1.450
25	12.9	2.6	1.600
26	12.7	2.9	2.300
27	13.3	3.1	5.150
28	12.9	3.6	4.500
29	12.9	3.5	1.650
30	13.4	3.3	4.900
31	13.0	3.2	3.300
Mean	12.7	2.8	2.460

Similarly, the temperature and wind velocity were calculated for all months.

B.2: Humidity

Year	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
1976										80.070	89.540	91.157	91.384	92.080	92.678	91.140	90.207	91.923			89.293	89.793	68.803	55.537
1977	75.750	69.383	71.073	63.250	49.406	51.033	71.140	67.540	78.741	77.720	79.200	93.423	95.466	95.579	93.947	94.627	92.007	96.003	94.250	84.845	89.753	88.357	89.791	83.347
1978	77.216	71.013	62.237	82.327	72.343	63.519	56.890	72.977	85.500	84.860	88.777	94.010	95.169	95.640	92.775	91.027	96.023	93.357	93.209	87.477	90.853	87.090	59.984	80.223
1979	79.947	79.683	82.100	67.681	53.522	48.210	58.853	70.790	67.206	62.873	74.620	91.637	92.806	97.157	94.659	98.980	95.970	90.140	92.184	87.673	82.183	88.387	83.613	86.593
1980	75.531	89.067	80.627	80.561	62.809	56.327	48.237	55.500	71.525	74.913	91.823	95.493	95.497	95.207	98.425	94.990	96.270	96.337	87.188	83.480	75.823	82.703	72.791	86.470
1981	76.925	80.047	75.730	69.600	61.275	74.337	68.677	76.357	83.497	82.860	78.950	94.090	97.569	97.387	94.081	98.820	97.240	93.190	85.613	86.360	83.933	69.947	74.641	83.460
1982	74.147	66.970	88.373	52.804	73.356	65.917	62.150	70.203	60.831	59.350	88.710	90.773	94.991	95.118	94.616	94.697	92.627	95.157	81.934	86.227	88.753	89.440	83.453	85.353
1983	75.816	82.510	70.370	61.477	61.934	52.213	48.717	73.783	82.816	80.660	79.060	74.957	96.597	94.420	94.669	95.867	96.370	95.333	89.138	80.990	88.917	86.730	67.309	82.040
1984	70.159	80.907	75.387	63.486	72.944	64.177	49.970	66.363	91.897	88.367	95.220	95.993	97.275	96.803	91.309	96.820	96.667	95.753	90.884	85.313	72.887	79.720	86.116	53.533
1985	66.463	83.557	79.427	81.735	64.425	54.307	71.470	75.727	70.981	76.443	83.513	92.883	96.791	97.087	92.347	96.347	94.527	96.567	93.263	87.517	86.190	89.893	78.516	86.083
1986	87.656	83.777	82.177	83.162	74.803	63.760	85.103	81.413	85.206	68.247	77.813	95.783	95.488	96.037	91.472	95.943	97.737	94.200	92.000	86.943	85.220	88.897	75.847	70.763
1987	73.750	81.903	82.737	64.073	77.153	79.410	58.207	71.230	70.875	60.233	85.810	91.923	96.166	98.497	97.569	93.793	94.238	91.550	79.056	83.460	71.739	74.380	77.891	55.550
1988	79.425	69.130	69.573	66.787	65.253	62.770	61.377	91.430	88.797	91.773	86.875	91.100	94.656	91.990	94.847	96.050	92.160	86.880	84.288	78.373	64.900	66.662	83.281	76.993
1989	77.181	78.207	48.560	63.385	51.806	65.427	50.833	37.587					96.947	95.536	92.031	90.560	95.587	97.073	86.338	76.747	74.967	79.280	65.869	83.933
1990	72.056	74.157	80.287	83.062	87.225	72.571	70.047	69.673	82.850	84.933	90.380	93.907	96.733	96.320	91.169	88.767	94.120	93.133	93.581	82.353	77.647	76.993	75.369	76.880
1991	88.973	78.940	67.360	64.615	69.238	77.680	62.160	54.667	71.944	87.193	91.780	91.493	96.800	90.050	96.144	91.793	97.340	95.120	92.619	77.513	69.100	81.307	62.400	69.350
1992	69.538	88.860	86.807	72.064	54.050	46.279	38.043	50.767	75.363	73.564	66.820	94.727	93.988	92.907	91.150	95.907	95.673	93.373	94.019	83.133	71.460	78.407	76.550	79.107

Year	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
1993			94.350	84.233	85.094	77.560	60.287	78.540	81.406	80.760	90.743	92.287	95.944	95.913	95.413	97.740	93.140	92.380	90.238	85.573	81.000	74.227	82.225	61.413
1994	68.694	46.800	77.233	76.862	71.875	74.633	50.380	48.640	56.988	91.500	86.727	94.640	95.056	97.160	94.119	93.347	96.067	94.407	93.613	86.000	75.580	83.727	72.038	80.253
1995	79.656	89.073	93.327	85.231	75.663	63.800	50.367	54.047	67.425	67.087	96.807	97.313	97.925	95.227	95.481	94.080	91.460	94.247	90.625	71.067	73.933	81.970	86.606	86.048
1996	85.263	70.603	84.463	73.361	84.566	58.573	45.730	71.999	75.749	73.885	95.626	97.914	97.766	97.123	97.477	95.590	94.773	93.800	87.666	86.690	77.520	83.130	74.753	76.783
1997	70.894	88.247	79.270	72.158	68.680	61.927	81.587	71.623	69.034	84.743	79.060	89.240	97.513	95.450	96.219	96.300	97.193	96.013	86.716	92.380	87.170	83.627	87.225	86.547
1998	85.678	83.957	89.707	83.069	84.756	84.750	74.103	86.697	81.016	86.060	76.090	98.210	97.453	98.037	99.441	99.593	95.727	97.653	97.338	96.027	76.483	85.300	74.366	61.423
1999	65.906	67.338	54.353	62.488	52.913	44.713	57.110	60.493	80.028	91.643	95.887	96.957	95.594	95.657	96.709	98.773	95.327	94.053	92.172	80.237	80.180	82.347	81.363	83.487
2000	84.688	76.257	75.413	63.179	58.303	75.143	89.127	79.293	80.156	93.447	94.953	98.470	97.956	97.400	97.497	97.490	96.227	92.817	88.866	87.260	88.300	79.460	76.491	79.406
2001	80.908	76.637	63.477	79.446	64.316	56.043	49.587	62.210	86.406	88.767	92.560	97.597	96.591	97.857	94.588	97.697	96.923	94.330	90.222	78.170	83.093	81.683	86.147	75.643
2002	66.699	74.443	68.073	70.846	70.369	68.173	78.939	74.063	89.931	82.883	92.997	93.250	95.484	95.283	94.144	94.357	91.150	92.650	91.756	82.730	80.053	80.790	75.422	84.160
2003	67.031	77.973	73.045	87.215	72.897	81.053	72.710	87.192	73.673	79.584	91.619	95.497	95.902	93.810	95.263	96.577	94.747	95.820	94.578	96.233	88.482	85.977	83.925	75.197
2004	76.725	85.550	75.770	81.404	70.556	73.483	78.110	81.490	81.766	93.343	95.437	94.947	98.369	98.510	95.950	96.303	96.747	96.103	90.563	85.500	83.253	85.077	82.781	84.680

Year	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
2005	85.628	85.497	80.187	78.115	80.931	85.817	72.593	75.543	85.363	83.840	87.730	95.780	96.416	94.560	96.794	96.747	91.753	91.183	87.922	86.233	84.700	86.780	73.291	70.103
2006	66.625	67.727	85.037	80.692	77.728	66.470	66.763	83.083	89.716	92.847	91.347	91.362	97.259	94.020	93.306	96.060	95.137	93.967	91.534	84.317	83.053	84.540	79.934	83.643
2007	76.381	84.727	92.033	88.469	83.681	77.347	74.413	80.977	84.197	74.840	93.800	92.210	94.975	98.147	94.600	96.193	97.857	92.687	93.400	90.283	88.910	83.980	84.200	73.097
2008	58.559	78.633	69.630	73.961	80.431	55.210	63.867	53.520	69.353	82.600	89.467	91.989	94.503	95.497	92.428	96.477	92.017	91.130	87.334	79.630	75.970	79.000	69.313	76.973
2009	73.047	65.897	67.443	69.754	71.613	80.247	77.100	69.620	74.806	86.417	82.217	84.503	93.563	91.347	96.528	95.997	95.097	91.317	90.594	83.883	70.773	82.183	81.081	85.723
RH Mean	75.404	77.421	76.534	73.653	69.876	66.148	63.777	69.850	77.970	80.858	87.332	93.197	95.959	95.553	94.701	95.454	94.885	93.813	90.142	84.564	80.649	82.405	77.452	77.053
RH mean mthly	76.412		75.094		68.012		66.813		79.414		90.265		95.756		95.078		94.349		87.353		81.527		77.253	

Table B.2: Mean monthly relative humidity calculation

Humidity data were obtained from DHM.

B.3: Radiation and Sunshine hours

Latitude = 27.72°

Table B.3: Radiation (Ra) values for latitudes in Northern hemisphere

Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0	14.5	15	15.2	14.7	13.9	13.4	13.5	14.2	14.9	15	14.6	14.3
10	12.8	13.9	14.8	15.2	15	14.8	14.8	15	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	16.5	16.2	15.3	13.5	11.3	9.1	7.9
40	6	8.3	11	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

Table B.4: Sunshine hours (N) values for latitudes in Northern hemisphere

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	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
40	9.6	10.7	11.9	13.2	14.4	15	14.7	13.8	12.5	11.2	10	9.4
50	8.6	10.1	11.8	13.8	15.4	16.4	16	14.5	12.7	10.8	9.1	8.1

Ra and N are interpolated at our latitude 27.72° for each month.

Table B.5: Interpolated Ra and N values

Radiation												
Latitude	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
27.72	9.024	10.910	12.974	14.891	15.932	16.340	16.086	15.300	13.705	11.665	9.579	8.447
Sunshine Hours												
Latitude	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
27.72	10.560	11.191	12.000	12.832	13.563	13.918	13.740	13.109	12.377	11.546	10.737	10.360

Table B.6: Monthly values required to calculate Et_o in tabulated form

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Tmax	12.727	14.640	18.873	22.283	22.789	22.973	22.407	22.529	21.864	20.335	17.332	13.958
Tmin	2.822	4.231	7.403	10.594	12.379	14.624	15.410	15.252	14.148	10.808	7.091	4.102
Rhmean	76.412	75.094	68.012	66.813	79.414	90.265	95.756	95.078	94.349	87.353	81.527	77.253
U	2.460	2.624	2.666	3.473	2.832	2.245	1.756	1.506	1.230	1.140	0.672	0.658
N	10.560	11.191	12.000	12.832	13.563	13.918	13.740	13.109	12.377	11.546	10.737	10.360
Ra	9.024	10.910	12.974	14.891	15.932	16.340	16.086	15.300	13.705	11.665	9.579	8.447

B.4: ET_o calculation table

We have:

Altitude = 2163 masl

Latitude = 27.72°

We proceed to calculate the evapotranspiration for the month of January. The values found out in the process are presented as follows.

B.4.1: Basic climate data

Table B.7: Required climate data for ET_0 calculations

Mean Daily Temperature:	7.775	°C
Mean Relative Humidity:	76.412	%
Max Relative Humidity:	79.609	%
Mean actual vapor pressure:	8.066	Kpa
Actual daily sunshine hours:	5.944	hrs
Measured 24 hr windrun:	212.516	km/d
Height of anemometer:	>2	m
Max Possible Sunshine Hours:	10.560	hrs
Extra Terrestrial Radiation:	9.024	W/m^2

B.4.2: Sample ET_0 calculation for January

Table B.8: ET_0 calculation for January

Wind Conversion Factor (ucf):	0.840	
Converted 24 hr windrun above ground:	178.514	km/d
Mean Atmospheric Pressure (PMB):	763.174	bars
Mean Absolute Temperature (Tkmean):	280.775	K
Saturation Vapor Pressure (ea):	10.556	Kpa
Mean Actual Vapor Pressure (ed):	8.066	KPa

f(u):	0.752
G:	0.504
D:	0.926
w:	0.648
R_s:	4.796
R_{ns}:	3.597
f(T):	12.430
f(e_d):	0.215
f(n/N):	0.607
R_{n1}:	1.621
R_n:	1.975

Selection of c

Rhmax	79.609	%
Rs	4.796	
U24	178.514	km/d
c	0.93	

$$\therefore ET_0 = c\big((w * R_n) + (1 - w) * f(u) * (e_a - e_d)\big) = 1.804\ mm/day$$

The ET_0 values for other months can also be calculated in the same way. The table below shows all the potential evapotranspiration and half monthly evapotranspiration values from January to December.

Table B.9: ET_0 monthly and half-monthly values

Month	ET_0	Half monthly ET_0 values		Month	ET_0	Half monthly ET_0 values	
		(1/2)	(2/2)			(1/2)	(2/2)
Jan	1.804	1.737	1.986	Jul	3.213	3.379	3.211
Feb	2.531	2.350	2.781	Aug	3.203	3.206	3.164
Mar	3.531	3.281	3.828	Sep	3.047	3.086	3.020
Apr	4.717	4.421	4.671	Oct	2.939	2.966	2.725
May	4.531	4.577	4.367	Nov	2.082	2.296	1.945
Jun	3.874	4.038	3.709	Dec	1.535	1.671	1.602

APPENDIX C

CROP WATER AND IRRIGATION WATER REQUIREMENTS

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C.1: Rainfall data tabulation of Kathmandu Airport

Table C.1: 80% reliable rainfall data calculation

SN	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	1968	30.1	8.5	45.3	25.5	109.6	305.7	379.5	228.2	86.9	160.4	0	0
2	1969	8.6	1.4	47.6	27.4	86.9	166.1	299.7	323.9	175.3	40.3	2	0
3	1970	29.1	27.6	26.6	34.4	93.6	193.7	494.3	229.7	163.9	58.2	11.2	0
4	1971	3	6.3	28.4	180.8	109.7	608.1	204.6	252.6	36.4	81.2	0.2	0
5	1972	1.4	25.5	80.4	23.8	56.6	157.3	480.9	155.3	174.5	86.1	19.6	0
6	1973	23.7	32.4	48.5	25.3	81.1	340.4	456	336.5	321.1	119.3	15.5	0
7	1974	16.9	5.8	23.3	30.9	108	74.8	339.6	364.2	204.6	45.6	0	11.4
8	1975	30.6	25.4	8	36.1	69.1	138.5	436.1	379	267.5	34.2	0	0
9	1976	30.2	14.5	0	68.6	153.4	387.4	335	307.3	169.9	24.3	0	0
10	1977	11.5	12.1	17.1	103.9	90.1	265.6	322.7	338.3	78.9	29.1	14.4	13.6
11	1978	4.7	11.1	69.4	41.7	143.3	298.9	323.6	392.5	159.8	108.6	0.2	2.2
12	1979	5.6	39.3	0.7	42.1	37.3	258.1	447.3	320.3	99.1	35.7	5.6	65.3
13	1980	1	17.7	45.7	10.1	124.4	349.3	296.1	238.5	183.5	69	0	5.6
14	1981	14.5	0	60.4	100.9	216.2	140.7	304	266.9	225.1	0	42	0
15	1982	14.2	21.9	35.5	48.8	39.7	200.5	238.2	384.3	155.4	9	18.3	3.4
16	1983	18.2	4	30.2	78.7	110.1	81.4	499.9	194.2	287.7	129.9	0	15.3
17	1984	13.9	17.4	13.5	60.1	96	275	250.1	301.9	260.2	18.4	0.1	7.4
18	1985	9.7	3.2	4	24.8	132.5	160.8	418.3	434.4	375.6	167.2	0	54.6
19	1986	0	22.5	15.8	93.4	96.9	315.6	380.8	218.6	221.3	79.5	0	49.4
20	1987	3.2	43.3	35.9	34.4	57.6	116.4	498.8	256.3	171.2	159.3	0	18.8
21	1988	0.6	19.1	68	42.3	152.9	239.5	397.3	278.7	134.4	17.6	11.7	78.9
22	1989	47.4	10.7	12.1	4	148.7	135.5	328	206	196.5	42.4	0	0.7
23	1990	0	42.2	59.5	116.2	108.3	364.7	345.6	308.5	188.2	78.7	0	2.8
24	1991	20.7	11.4	45.2	106.3	145.3	114.4	190.3	280.9	127.9	0.4	0.2	24.9
25	1992	6.4	17.2	0.2	44.5	69.9	232.7	223.6	219.9	209.1	51	15.5	3.1
26	1993	10.2	15.4	42	86.8	184.6	204.4	296.3	293.8	156.2	14.5	1.8	0
27	1994	27.3	19.4	13.9	8.3	141.8	413.6	254.1	445.6	243.2	0	12	0
28	1995	3.3	28.8	39.8	3.2	60.5	590.5	336.1	404.1	100.6	38.3	61.6	7
29	1996	70.8	15.2	7.1	47.4	57.7	337.8	318.5	485.5	207.3	52.4	0	0
30	1997	16.4	5.5	13.9	100.1	90.3	245.4	511	370.5	70.9	12	4.9	87.4
31	1998	0.1	28.2	70.6	75.9	282	247.7	440.2	376.3	193.6	44.2	12	0
32	1999	4.2	4.2	0	6	106.5	315.6	485.2	393.5	266.9	152.2	0	1.2
33	2000	1.3	5.3	20.9	61.9	209.9	266.5	336.3	384.7	119.4	0.6	0	0.2
34	2001	6.8	15.7	8.4	34.6	179.9	250.4	498.8	460.3	145.5	20.5	0	0
35	2002	33.8	29.9	93	93.9	158.8	227.4	544.8	499.9	148	15	26.5	0
36	2003	19.5	68.4	85.9	38	37.7	222.3	591.5	347	293.4	17.7	0	18.6
37	2004	26.9	0	32.3	164.1	168.8	183	459.5	219.4	199.1	120.5	36	0
38	2005	55.1	17	50.1	34.8	40.6	222.9	253.5	309.3	126.5	126.1	0	0
39	2006	0	0	30.9	132.8	145.5	216.2	337	248.4	217.5	43.9	1.5	17.5
40	2007	0	72.8	36.3	77.9	90.7	263	227.3	223.7	332.5	18.5	3.2	0
41	2008	4.9	0	35.9	43.7	99.9	237.7	255.4	240.8	291.3	10.3	0	0
42	2009	0	0	28.4	21.3	132	125	326.3	382.9	113.4	71.5	1	3.6
43	2010	1.9	23.3	35.7	45.3	148	141.7	354.9	486.3	217.1	24.5	0	0
44	2011	6.2	54.9	16.4	56.8	167.4	306	437.8	265.4	318	13	12.9	0
TOTAL		633.9	844.5	1482.8	2537.8	5139.8	10938.2	16154.8	14054.3	8434.4	2441.1	329.9	492.9
MEAN		14.407	19.193	33.700	57.677	116.814	248.595	367.155	319.416	191.691	55.480	7.498	11.202
SD		15.922	17.232	24.190	41.318	52.494	112.783	101.083	87.309	76.381	48.998	13.014	21.774
80% REL		1.007	4.690	13.341	22.904	72.635	153.677	282.083	245.937	127.408	14.243	-3.455	-7.123
CORRECTED 80% REL		1.007	4.690	13.341	22.904	72.635	153.677	282.083	245.937	127.408	14.243	0.000	0.000

C.2: Irrigation Water Requirements

C.2.1: Pattern – 1 (85% land area)

Table C.2: IWR for 85% land area

	POTATO					MAIZE							land prep	MONSOON RICE							POTATO			
Month	Jan	Jan	Feb	Feb	Mar	Mar	Apr	Apr	May	May	Jun	Jun	Jul	Jul	Aug	Aug	Sep	Sep	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/d)	1.737	1.986	2.350	2.781	3.281	3.828	4.421	4.671	4.577	4.367	4.038	3.709	3.379	3.211	3.206	3.164	3.086	3.020	2.966	2.725	2.296	1.945	1.671	1.602
ET0 (mm)	26.917	30.779	32.893	38.939	50.858	59.329	66.312	70.060	70.948	67.681	60.576	55.636	52.368	49.768	49.692	49.048	46.295	45.303	45.973	42.231	34.439	29.173	25.906	24.830
80% Rain(mm)	2.065	3.979	6.430	8.918	11.441	15.644	21.525	30.929	43.856	64.264	92.152	122.355	154.871	165.723	154.909	133.383	101.145	69.031	37.040	15.783	5.261	0.000	0.000	0.688
Crop coefficient	1.13	1.13	1.08	0.94	0.77	0.45	0.6	0.8	1.05	1.05	1.05	0.8	0	1.1	1.1	1.1	1.1	1.05	0.95	0.95	0.42	0.55	0.79	1.01
Etcrop (mm)	30.416	34.781	35.525	36.602	39.161	26.698	39.787	56.048	74.496	71.066	63.605	44.509	0.000	54.745	54.661	53.953	50.924	47.568	43.674	40.120	14.465	16.045	20.466	25.078
Land Prep (mm)													55	50	50									
Percolation (mm)													77.5	77.5	77.5	77.5	75	75	77.5	77.5				
Field Req (mm)	30.416	34.781	35.525	36.602	39.161	26.698	39.787	56.048	74.496	71.066	63.605	44.509	132.500	182.245	182.161	131.453	125.924	122.568	121.174	117.620	14.465	16.045	20.466	25.078
Eff. Rainfall	0.000	0.000	4.501	6.243	8.009	10.951	15.067	21.650	30.699	44.985	64.507	104.002	131.641	140.864	131.673	113.376	85.973	48.321	25.928	11.048	3.683	0.000	0.000	0.000
I-net corrected	30.416	34.781	31.023	30.360	31.152	15.747	24.720	34.398	43.796	26.081	0.000	0.000	0.859	41.381	50.488	18.077	39.951	74.246	95.247	106.571	10.782	16.045	20.466	25.078
I-net	30.416	34.781	31.023	30.360	31.152	15.747	24.720	34.398	43.796	26.081	-0.902	-59.493	0.859	41.381	50.488	18.077	39.951	74.246	95.247	106.571	10.782	16.045	20.466	25.078
E-field	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
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	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	0.8
I-gross (mm)	72.418	82.811	73.865	72.285	74.171	37.494	58.856	81.899	104.277	62.097	0.000	0.000	2.046	98.525	120.210	43.041	95.121	176.777	226.778	253.742	25.671	38.203	48.728	59.709
I-gross (l/s/ha)	0.541	0.618	0.611	0.598	0.554	0.280	0.454	0.632	0.779	0.464	0.000	0.000	0.015	0.736	0.898	0.321	0.734	1.364	1.693	1.895	0.198	0.295	0.364	0.446

C.2.2: Pattern – II (15% land area)

Table C.3: IWR for 15% land area

	Winter Vegetable												Summer Vegetable									Winter Vegetable		
Month	Jan	Jan	Feb	Feb	Mar	Mar	Apr	Apr	May	May	Jun	Jun	Jul	Jul	Aug	Aug	Sep	Sep	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/d)	1.73655	1.98576	2.34952	2.78132	3.28116	3.82765	4.42078	4.67067	4.57731	4.36655	4.03839	3.70906	3.37856	3.21085	3.20592	3.16442	3.08632	3.02019	2.966	2.72459	2.29596	1.94488	1.67135	1.6019
Et0 (mm)	26.9165	30.7792	32.8933	38.9385	50.858	59.3286	66.3117	70.06	70.9483	67.6815	60.5758	55.6359	52.3676	49.7681	49.6918	49.0485	46.2949	45.3028	45.973	42.2312	34.4395	29.1732	25.9059	24.8295
80% Rain(mm)	2.06492	3.97896	6.43042	8.91787	11.4413	15.6436	21.5249	30.929	43.8561	64.2638	92.1523	122.355	154.871	165.723	154.909	133.383	101.145	69.0306	37.0397	15.7831	5.26105	0	0	0.68831
Crop coefficient	0.86	0.95	0.89	0.94	0	0	0	0	0	0	0	0	0.34	0.54	0.93	2.05	1.05	1.04	0.91	0	0	0.28	0.34	0.54
Etcrop (mm)	23.1482	29.2403	29.275	36.6022	0	0	0	0	0	0	0	0	17.805	26.8748	46.2134	100.549	48.6096	47.1149	41.8355	0	0	8.1685	8.808	13.4079
Land Prep (mm)													0	0	0									
Percolation (mm)													0	0	0	0	0	0	0	0				
Field Req (mm)	23.1482	29.2403	29.275	36.6022	0	0	0	0	0	0	0	0	17.805	26.8748	46.2134	100.549	48.6096	47.1149	41.8355	0	0	8.1685	8.808	13.4079
Eff. Rainfall	0	0	4.5013	6.24251	8.0089	10.9505	15.0674	21.6503	30.6992	44.9847	64.5066	104.002	131.641	140.864	131.673	113.376	85.9734	48.3214	25.9278	11.0482	3.68273	0	0	0
I-net corrected	23.1482	29.2403	24.7737	30.3597	0	0	0	0	0	0	0	0	0	0	0	0	0	0	15.9077	0	0	8.1685	8.808	13.4079
I-net	23.1482	29.2403	24.7737	30.3597	-8.0089	-10.951	-15.067	-21.65	-30.699	-44.985	-64.507	-104	-113.84	-113.99	-85.459	-12.827	-37.364	-1.2065	15.9077	-11.0482	-3.6827	8.1685	8.808	13.4079
E-field	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
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	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	0.8

APPENDIX D

FLOOD CALCULATIONS

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D.1: Rational Method

River = Manohara River

length of longest channel = 10.559 km

height difference = 1043 m

time of concentration (t_c) = 0.993 hrs

slope = 0.09878

Table D.1: Maximum discharge for different return periods by Rational Method

Return Period	2	5	10	20	33	50	100	200	300	500	1000
Max. Daily Rainfall (mm)	70.066	89.121	103.533	117.822	128.360	136.997	151.414	165.828	174.254	184.883	199.295
Time/Intensity	RI2	RI5	RI10	RI20	RI33	RI50	RI100	RI200	RI300	RI500	RI1000
0.25	61.207	77.852	90.441	102.923	112.130	119.674	132.269	144.860	152.220	161.505	174.095
0.5	38.558	49.044	56.975	64.838	70.638	75.390	83.324	91.256	95.893	101.742	109.674
0.75	29.425	37.427	43.480	49.481	53.907	57.534	63.589	69.642	73.180	77.644	83.697
1	24.290	30.896	35.892	40.846	44.499	47.493	52.491	57.488	60.409	64.094	69.090
2	15.302	19.463	22.611	25.731	28.033	29.919	33.068	36.215	38.056	40.377	43.524
4	9.640	12.261	14.244	16.210	17.660	18.848	20.831	22.814	23.974	25.436	27.419
6	7.356	9.357	10.870	12.370	13.477	14.384	15.897	17.411	18.295	19.411	20.924
8	6.073	7.724	8.973	10.212	11.125	11.873	13.123	14.372	15.102	16.024	17.273
12	4.634	5.895	6.848	7.793	8.490	9.061	10.015	10.968	11.525	12.228	13.182
16	3.826	4.866	5.653	6.433	7.008	7.480	8.267	9.054	9.514	10.094	10.881
20	3.297	4.193	4.871	5.544	6.040	6.446	7.124	7.803	8.199	8.699	9.377
24	2.919	3.713	4.314	4.909	5.348	5.708	6.309	6.910	7.261	7.703	8.304
0.993	24.404	31.040	36.060	41.037	44.707	47.715	52.737	57.757	60.692	64.394	69.414
Q, m^3/s	61.483	78.203	90.850	103.388	112.636	120.215	132.866	145.514	152.908	162.234	174.882

D.2: Hydest WECS 1990

River = Manohara River

Location = Changunarayan

Drainage Basin Area = 30.233 km^2

Area of basin below 5000 m elevation = 30.233 km^2

Area of basin below 3000 m elevation = 30.233 km^2

Table D.2: Flood flow statistics by Hydest WECS 1990 Method

Return Period (yrs)	**Flood Discharge (m^3/s)**	
	Daily	**Instantaneous**
2	22	39
5	34	68
10	43	91
20	53	116
50	66	153
100	76	183
200	87	216
500	102	265
1000	115	305
5000	146	413
10000	161	465

D.3: DHM 2004

River = Manohara River

Location = Changunarayan

Basin Area = 30.233 km^2

Area of basin below 5000 m elevation = 30.233 km^2

Area of basin below 3000 m elevation = 30.233 km^2

Average basin elevation = 1872.5 masl

Annual wetness index = 1600 mm

Table D.3: Flood flow statistics by DHM 2004 Method

Return Period (yrs)	Flood Discharge (m^3/s)	
	Daily	Instantaneous
2	22	43
5	34	80
10	43	111
20	53	145
50	66	197
100	76	241
200	87	290
500	102	363
1000	115	425
5000	146	593
10000	161	677

D.4: MHSP 1997

Total drainage area = 30.233 km^2

Mean precipitation = 82.04 mm

Monsoon wetness index = 1600

Region = Central

Table D.4: Flood flow estimation by MHSP 1997 method

Return Period	Flow (m^3/s)
5	45.13
20	76.43
50	101.52
100	123.89
1000	224.05
10000	379.22

D.5: Modified Dicken's

Total basin area = 30.233 km^2

Perpetual snow area = 0 km^2

P = 19.846

Table D.5: Flood flow estimation by Modified Dicken's method

T	2	10	33	50	100	300	500	1000
C_T	4.33	7.24	9.39	10.14	11.4	13.38	14.3	15.56
Q_T	56	93	121	131	147	173	184	201

D.6: PCJ 1996

River = Manohara River

Location = Changunarayan

Basin Area = 30.233 km^2

Area of basin below 5000 m elevation = 30.233 km^2

Area of basin below 3000 m elevation = 30.233 km^2

Table D.6: Flood flow estimation by PCJ 1996 method

Rainfall Station	Return Period in Years				
	10	33	50	100	300
	Hourly Intensity in mm/min				
Sankhu (1035)	0.71	0.84	0.87	0.93	1.01
	Since the hourly intensity for Nagarkot and Changunarayan Stations are not registered, we convert the maximum daily rainfall obtained from Rational Method to PCJ hourly intensity in mm/min.				
Average {(0.38*rainfall)}/60	0.656	0.813	0.868	0.959	1.104
K_t	0.98	0.98	0.98	0.98	0.98
a_p	0.64	0.80	0.85	0.94	1.08
O_p	0.5	0.75	0.85	0.95	1
ϕ	0.272	0.272	0.272	0.272	0.272
K_f	1.000	1.000	1.000	1.000	1.000
F	30.24	30.24	30.24	30.24	30.24
Q	44.07	81.93	99.14	122.42	148.35

D.7: Discharge Calculation by Flowmeter

D.7.1: Calculation in section 1 (upstream of intake)

Table D.7: Flowmeter calculation table for upstream section

Chainage(cm)	Depth(cm)	Speed(ft/sec)	Speed(m/s)	Area(m^2)
40	0.5	0.2	0.061	0.009
55	11.5			
70	13	2.9	0.884	0.082
85	13			
100	13			
115	14.8			
130	19			
145	20.5	3.7	1.128	0.136
160	23.5			
175	20.5			
190	24			
205	32			
220	35	4.5	1.372	0.195
235	32.5			
250	30			
265	32.5			
280	31.5			
295	29	4.2	1.280	0.171
310	27			
325	28			
340	30			
355	30.5			
370	28	3.4	1.036	0.160
385	31			
400	26.5			
415	21.5			
430	16.5			
445	10	0.2	0.061	0.028
460	9			
475	0			

Discharge (Q_1) = 0.879 m^3/s

D.7.2: Calculation in section 2 (downstream of intake)

Table D.8: Flowmeter calculation table for downstream section

<table>
<tr><th>Chainage(cm)</th><th>Depth(cm)</th><th>Speed(ft/sec)</th><th>Speed(m/s)</th><th>Area(m²)</th></tr>
<tr><td>200</td><td>0</td><td rowspan="5">0.800</td><td rowspan="5">0.350</td><td rowspan="5">0.021</td></tr>
<tr><td>230</td><td>3</td></tr>
<tr><td>260</td><td>2.5</td></tr>
<tr><td>290</td><td>1</td></tr>
<tr><td>320</td><td>4</td></tr>
<tr><td>350</td><td>9</td><td rowspan="5">2.100</td><td rowspan="5">0.710</td><td rowspan="5">0.106</td></tr>
<tr><td>380</td><td>9</td></tr>
<tr><td>410</td><td>10</td></tr>
<tr><td>440</td><td>8</td></tr>
<tr><td>470</td><td>13</td></tr>
<tr><td>500</td><td>10.5</td><td rowspan="3">2.700</td><td rowspan="3">0.823</td><td rowspan="3">0.091</td></tr>
<tr><td>530</td><td>12</td></tr>
<tr><td>560</td><td>11</td></tr>
<tr><td>590</td><td>18.5</td><td rowspan="5">4.100</td><td rowspan="5">1.240</td><td rowspan="5">0.257</td></tr>
<tr><td>620</td><td>24</td></tr>
<tr><td>650</td><td>22</td></tr>
<tr><td>680</td><td>20</td></tr>
<tr><td>710</td><td>27</td></tr>
<tr><td>740</td><td>27</td><td rowspan="3">3.400</td><td rowspan="3">1.100</td><td rowspan="3">0.228</td></tr>
<tr><td>770</td><td>30</td></tr>
<tr><td>800</td><td>27</td></tr>
<tr><td>830</td><td>30</td><td rowspan="3">3.100</td><td rowspan="3">0.970</td><td rowspan="3">0.158</td></tr>
<tr><td>860</td><td>20</td></tr>
<tr><td>890</td><td>9</td></tr>
<tr><td>920</td><td>3</td><td rowspan="2">0.400</td><td rowspan="2">0.130</td><td rowspan="2">0.005</td></tr>
<tr><td>950</td><td>0</td></tr>
</table>

Discharge (Q_1) = 0.881 m^3/s

APPENDIX E

DESIGN OF HEADWORKS

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E.1: Design of Weir

E.1.1: Data

Design flood discharge = 132.87 m^3/s

Riverbed level = 1303.05 m

HFL before construction of weir = 1305.23 m

Permissible afflux = 1 m

Pond level = 1305.15 m

Canal FSL = 1304.65 m

Discharge concentration factor = 20%

Bed retrogression = 0.5 m

Lacey's silt factor = 1.2

Weir crest raised to Pond level.

∴ No Pond level flow through weir.

Let, Crest level of weir bay = 1305.15 m

Crest width = 1 m

E.1.2: Check for discharge capacity

Lacey's wetter perimeter = 54.75 m

1.2P = 65.70 m (take 65 m waterway where under sluice is 5 m wide, weir is 57 m, and a divide wall of 3 m separates them)

q = 132.87/65 = 2.044 cumecs/m

scour depth, R = $1.35\left(\frac{q^2}{1.2}\right)^{1/3}$ = 2.046 m

Velocity = 0.999 m/s

velocity head = 0.051 m

H on under sluice = 3.231 m

H on weir = 1.131 m

Q = Q1 + Q2 = 159.052 m^3/s > 132.87 m^3/s (Design is proper)

E.1.3: Determination of discharge intensity (q) and Head loss (HL) for high flood condition:

Table E.1: q and HL for HFL conditions of weir

a	Without flow concentration and retrogression	
	Head on weir (H)	1.131 m
	$q_a = 1.7H^{1.5}$	2.044 cumecs/m
	u/s TEL	1306.281 m
	d/s TEL	1305.281 m
	HL	1 m
b	With 20% concentration and 0.5 m retrogression	
	$q_b = 1.2q_a$	2.453 cumecs/m
	Head over the crest = $(q_b/1.7)^{0.667}$	1.277 m
	u/s TEL	1306.427 m
	d/s TEL	1304.781 m
	HL	1.646 m

E.1.4: Hydraulic Jump Calculation

Table E.2: Hydraulic jump calculation of weir

SN	Item	High Flood Flow	
		no concentration and retrogression	20% concentration, 0.5 m retrogression
1	discharge intensity (q)	2.044	2.453
2	u/s water level	1306.230	1306.230
3	d/s water level	1305.230	1304.730
4	u/s TEL	1306.281	1306.427
5	d/s TEL	1305.281	1304.781
6	Head Loss (HL)	1.000	1.646
7	d/s specific energy (Ef_2)	1.623	1.951
8	jump level = d/s TEL - Ef_2	1303.658	1302.829
9	u/s specific energy (Ef_1)	2.623	3.623
10	pre-jump depth (y1)	0.303	0.304
11	post-jump depth (y2)	1.532	1.863
12	length of floor = 5(y2-y1)	6.145	7.795
13	Initial Froude Number (F1)	3.914	4.673

d/s floor length = 8 m (take greater than 7.795 m)

d/s floor level = 1302.8 m

glacis slope = 3:1

u/s slope = 1:1

crest width = 1 m

u/s slope length = (1305.15 - 1303.05) * 1 = 2.1 m

d/s glacis length = (1305.15 – 1302.8) * 3 = 7.05 m

E.1.5: Depth of Sheet pile

q = 2.036 cumecs/m

scour depth, R = $1.35\left(\frac{q^2}{1.2}\right)^{1/3}$ = 2.041 m (take 2.1 m)

RL of bottom of d/s cutoff = 1304.73 – 1.5*2.1 = 1301.58 m

Since the depth of cutoff is very less in this case, we increase the depth up to 3 m from d/s floor.

RL of bottom of d/s cutoff = 1302.8 – 3 = 1299.8 m

RL of bottom of u/s cutoff = 1306.23 – 1.25*2.1 = 1303.605 m

Increasing the depth of the sheet pile up to 2 m from the u/s floor, we get:

RL of bottom of u/s cutoff = 1303.05 – 2 = 1301.05 m

E.1.6: Total Floor Length

G_E = 1/6

Maximum Static Head (H) = 1305.15 – 1302.8 = 2.35 m

d/s cutoff depth (d) = 3 m

λ = 2.238

α = 3.329

Now, we get the floor length values as:

b = α.d = 9.988 m

balancing u/s floor length = 9.988 – 7.05 – 8 = -8.162 ($-^{ve}$)

Since the u/s floor length is negative, we provide a minimum required length of 2 m.

Total floor length = 20.15 m

E.1.7: Uplift Pressure Calculation

$\lambda_1 = 5.562$

$\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\left(\frac{0-\lambda_1}{\lambda_1}\right) = 100\%$

$\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\left(\frac{1-\lambda_1}{\lambda_1}\right) = 80.615\%$

$\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\left(\frac{2-\lambda_1}{\lambda_1}\right) = 72.215\%$

$\lambda_2 = 3.895$

$\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-2}{\lambda_2}\right) = 33.825\%$

$\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-1}{\lambda_2}\right) = 23.327\%$

$\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-0}{\lambda_2}\right) = 0\%$

Table E.3: Corrected uplift pressure percentages of weir

Point	Pressure	Correction for			Corrected Pressure
		Thickness	MI	Slope	
E1	100	0	0	0	100
D1	80.615	0	0	0	80.615
C1	72.125	4.245	1.064	0	77.434
E2	33.825	-5.249	-0.191	0	28.385
D2	23.327	0	0	0	23.327
C2	0	0	0	0	0

Table E.4: HGL calculation through uplift pressures of weir

Condition of flow	u/s water level	d/s water level	H	HG line above Datum					
				u/s pile			d/s pile		
				ϕE1	ϕD1	ϕC1	ϕE2	ϕD2	ϕC2
No flow	1305.15	1302.8	2.35	2.350	1.894	1.820	0.667	0.548	0.000
				1305.150	1304.694	1304.620	1303.467	1303.348	1302.800
HFL with conc and ret	1306.23	1304.73	1.5	1.500	1.209	1.162	0.426	0.350	0.000
				1306.230	1305.939	1305.892	1305.156	1305.080	1304.730

E.1.8: Pre-jump Profile

Table E.5: Pre-jump Profile of weir

distance	glacis level	HFL	
		Ef_1	y_1
0	1305.15	1.2770231	-
3	1304.15	2.2770231	0.405
6	1303.15	3.2770231	0.322
6.962	1302.829	3.598	0.305

E.1.9: Post-jump Profile

Table E.6: Post-jump Profile of weir

x	HFL		
	x/(y2-y1)	y/(y2-y1)	y
2	1.283	0.35	0.546
4	2.566	0.65	1.013
6	3.849	0.88	1.372
8	5.131	1	1.559

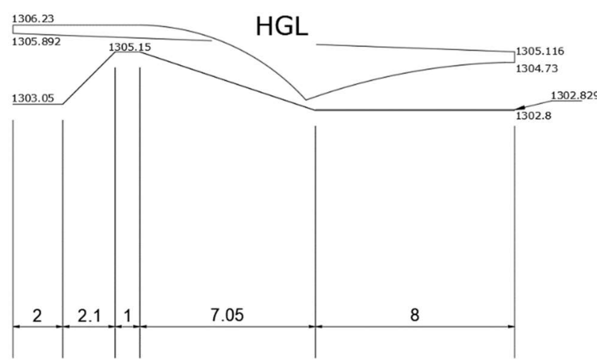


Fig E.1: Jump formation in weir at High Flood Condition

E.1.10: Thickness of Floor

Table E.7: Floor thickness at distances from crest of weir

SN	Distance from Crest	No flow condition			High Flood Condition				controlling head	thickness
		Level of floor	HGL	UBH	water level	HGL	UBH	2H/3		
1	0	1305.15	1304.505	-0.645	-	1305.818	-	-	-	-
2	3	1304.15	1304.334	0.184	1304.555	1305.709	1.154	0.769	1.154	1.012
3	6	1303.15	1304.162	1.012	1303.472	1305.599	2.127	1.418	2.127	1.866
4	6.962	1302.829	1304.107	1.278	1303.134	1305.564	2.430	1.620	2.430	2.131
5	8.962	1302.8	1303.993	1.193	1303.346	1305.491	2.146	1.430	2.146	1.882
6	10.962	1302.8	1303.878	1.078	1303.813	1305.418	1.605	1.070	1.605	1.408
7	12.962	1302.8	1303.764	0.964	1304.172	1305.345	1.173	0.782	1.173	1.029
8	14.962	1302.8	1303.649	0.849	1304.359	1305.272	0.913	0.609	0.913	0.801

E.1.11: Protection Works

R = 2.1 m

D/S protection:

D = 2*R – y2 = 2*2.1 – (1304.730 – 1302.8) = 2.27 m

Length of Launching apron = L = 1.5D = 1.5*2.27 = 3.405 m (take 4 m launching apron)

Thickness of apron = (2.25*D)/L = 1.277 m (take a thickness of 1.5 m)

U/S protection:

D = 1.5*R – y1 = 1.5*2.1 – (1306.23 – 1303.05) = -0.03 m ($-^{ve}$)

Take a launching apron length of 3 m of thickness 1.5 m.

E.2: Design of Under sluice

E.2.1: Data

Discharge concentration factor = 20%

Bed retrogression = 0.5 m

Lacey's silt factor = 1.2

Let, Crest level of under sluice = riverbed level = 1303.05 m

d/s HFL = 1303.125 m

E.2.2: Check for flow at Pond level condition

H on under sluice = 2.1 m

Q = 23.694 m^3/s

q = 4.739 cumecs/m

scour depth, R = 3.584 m

V = 1.322 m/s

Velocity head = 0.089 m

E.2.3: Determination of q and HL for different flow conditions:

Table E.8: q and HL for HFL conditions of under sluice

a	Without flow concentration and retrogression	
	Head on under sluice (H)	3.231 m
	$q_a = 1.7H^{1.5}$	9.873 cumecs/m
	u/s TEL	1306.281 m
	d/s TEL	1305.281 m
	HL	1 m
b	With 20% concentration and 0.5 m retrogression	
	$q_b = 1.2q_a$	11.847 cumecs/m
	Head over the crest = $(q_b/1.7)^{0.667}$	3.648 m
	u/s TEL	1306.698 m
	d/s TEL	1304.781 m
	HL	1.918 m

Table E.9: q and HL for pond level conditions of under sluice

a	Without flow concentration and retrogression	
	Head on under sluice (H)	2.1 m
	$q_a = 1.7H^{1.5}$	5.173 cumecs/m
	u/s TEL	1305.239 m
	d/s TEL	1303.214 m
	HL	2.025 m
b	With 20% concentration and 0.5 m retrogression	
	$q_b = 1.2q_a$	6.208 cumecs/m
	Head over the crest = $(q_b/1.7)^{0.667}$	2.371 m
	u/s TEL	1305.421 m
	d/s TEL	1302.714 m
	HL	2.707 m

E.2.4: Hydraulic Jump Calculation

Table E.10: Hydraulic jump calculation of under sluice

SN	Item	High Flood Flow		Pond Level Flow	
		no concentration and retrogression	20% concentration, 0.5 m retrogression	no concentration and retrogression	20% concentration, 0.5 m retrogression
1	discharge intensity (q)	9.873	11.847	5.173	6.208
2	u/s water level	1306.230	1306.230	1305.150	1305.150
3	d/s water level	1305.230	1304.730	1303.125	1302.625
4	u/s TEL	1306.281	1306.698	1305.239	1305.421
5	d/s TEL	1305.281	1304.781	1303.214	1302.714
6	Head Loss (HL)	1.000	1.918	2.025	2.707
7	d/s specific energy (Ef_2)	4.015	4.863	3.053	3.539
8	jump level = d/s TEL - Ef_2	1301.266	1299.917	1300.161	1299.175
9	u/s specific energy (Ef_1)	5.142	6.777	5.075	6.233
10	pre-jump depth (y1)	1.110	1.125	0.549	0.590
11	post-jump depth (y2)	3.640	4.512	2.890	3.366
12	length of floor = 5(y2-y1)	12.650	16.935	11.705	13.880
13	Initial Froude Number (F1)	2.695	3.170	4.061	4.374

d/s floor length = 17 m

d/s floor level = 1299.1 m

glacis slope = 3:1

d/s glacis length = (1303.05 – 1299.1) * 3 = 11.85 m

E.2.5: Depth of Sheet pile

q = 8.597 cumecs/m

scour depth, R = 1.35 $\left(\frac{q^2}{1.2}\right)^{1/3}$ = 5.331 m (take 5.4 m)

RL of bottom of d/s cutoff = 1304.73 – 1.5*5.4 = 1296.63 m

Since the depth of cutoff is very less in this case (2.47 m), we double the depth up to 4.94 m from d/s floor.

RL of bottom of d/s cutoff = 1299.1 – 4.94 = 1294.16 m

RL of bottom of u/s cutoff = 1306.23 – 1.25*5.4 = 1299.48 m

Depth of u/s cutoff = 3.57 m

E.2.6: Total Floor Length

G_E = 1/6

Maximum Static Head (H) = 1305.15 – 1299.1 = 6.05 m

d/s cutoff depth (d) = 4.94 m

λ = 5.471

α = 9.891

Now, we get the floor length values as:

b = α.d = 48.864 m

balancing u/s floor length = 48.864 – 17 – 11.85 = 20.014 m

E.2.7: Uplift Pressure Calculation

λ_1 = 7.362

$\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\left(\frac{0-\lambda_1}{\lambda_1}\right)$ = 100%

$\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\left(\frac{1-\lambda_1}{\lambda_1}\right)$ = 83.215%

$\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\left(\frac{2-\lambda_1}{\lambda_1}\right)$ = 75.97%

λ_2 = 5.471

$\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-2}{\lambda_2}\right)$ = 28.123%

$\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-1}{\lambda_2}\right)$ = 19.552%

$\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-0}{\lambda_2}\right)$ = 0%

Table E.11: Corrected uplift pressure percentages of under sluice

Point	**Pressure**	**Correction for**			**Corrected Pressure**
		Thickness	**MI**	**Slope**	
E1	100	0	0	0	100
D1	83.215	0	0	0	83.215
C1	75.970	2.029	1.660	0	79.660
E2	28.123	-2.603	-0.412	0	25.109
D2	19.552	0	0	0	19.552
C2	0.000	0	0	0	0

Table E.12: HGL calculation through uplift pressures of under sluice

Condition of flow	u/s water level	d/s water level	H	HG line above Datum					
				u/s pile			d/s pile		
				ϕE1	ϕD1	ϕC1	ϕE2	ϕD2	ϕC2
No flow	1305.15	1299.1	6.05	6.05	5.035	4.819	1.519	1.183	0
				1305.15	1304.135	1303.919	1300.619	1300.283	1299.1
HFL with conc and ret	1306.23	1304.73	1.5	1.5	1.248	1.195	0.377	0.293	0
				1306.230	1305.978	1305.925	1305.107	1305.023	1304.730
PL with conc and ret	1305.15	1302.625	2.525	2.525	2.101	2.011	0.634	0.494	0
				1305.15	1304.726	1304.636	1303.259	1303.119	1302.625

E.2.8: Pre-jump Profile

Table E.13: Pre-jump Profile of under sluice

distance	glacis level	HFL		PL	
		Ef1	y1	Ef1	y1
0	1303.05	3.648	-	2.371	-
3	1302.05	4.648	1.5	3.371	0.89
6	1301.05	5.648	1.28	4.371	0.73
9	1300.05	6.648	1.14	5.371	0.65
9.398	1299.917	6.781	1.12	5.504	0.64
11.626	1299.175	7.524	1.05	6.247	0.59

E.2.9: Post-jump Profile

Table E.14: Post-jump Profile of under sluice

x/y1	HFL			PL		
	y/y1	y	x	y/y1	y	x
1.25	1.3	1.4625	1.40625	1.3	0.767	0.7375
2.5	1.9	2.1375	2.8125	1.9	1.121	1.475
5	2.8	3.15	5.625	2.8	1.652	2.95
10	3.5	3.9375	11.25	3.8	2.242	5.9
15	4	4.5	16.875	4.6	2.714	8.85

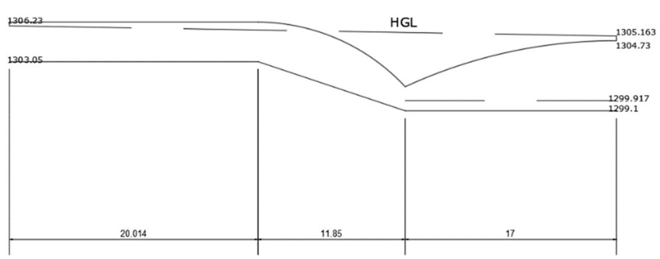


Fig E.2: Jump formation in under sluice at High Flood Condition

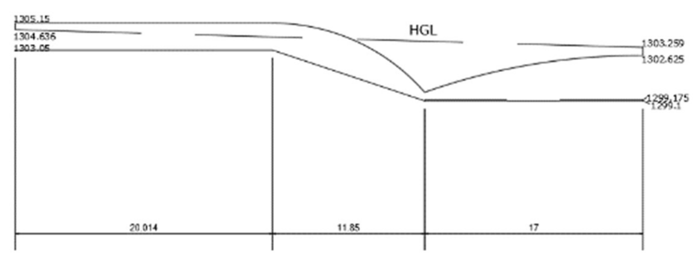


Fig E.3: Jump formation in under sluice at pond level condition

E.2.10: Thickness of Floor

Table E.15: Floor thickness at distances from crest of under sluice

SN	Distance from Crest	No flow condition			High Flood Condition				Pond Level Condition				controlling head	thickness
		floor level	HGL	UBH	water level	HGL	UBH	2H/3	water level	HGL	UBH	2H/3		
1	0	1303.050	1302.568	-0.482	-	1305.590	-	-	-	1304.072	-	-	-	-
2	3	1302.050	1302.365	0.315	1303.550	1305.540	1.990	1.326	1302.940	1303.988	1.048	0.698	1.990	1.745
3	6	1301.050	1302.162	1.112	1302.330	1305.489	3.159	2.106	1301.780	1303.903	2.123	1.415	3.159	2.771
4	9	1300.050	1301.960	1.910	1301.190	1305.439	4.249	2.833	1300.700	1303.819	3.119	2.079	4.249	3.727
5	9.398	1299.917	1301.933	2.015	1301.037	1305.432	4.395	2.930	1300.557	1303.807	3.250	2.167	4.395	3.855
6	10.804	1299.449	1301.838	2.389	1300.911	1305.409	4.498	2.998					4.498	3.945
7	12.210	1299.100	1301.743	2.643	1301.238	1305.385	4.148	2.765					4.148	3.638
8	15.023	1299.100	1301.553	2.453	1302.250	1305.338	3.088	2.059					3.088	2.709
9	20.648	1299.100	1301.173	2.073	1303.038	1305.244	2.206	1.471					2.206	1.936
10	26.273	1299.100	1300.793	1.693	1303.600	1305.150	1.550	1.033					1.693	1.485
11	11.626	1299.175	1301.782	2.608	1300.225	1305.395	5.170	3.447	1299.765	1303.745	3.980	2.653	5.170	4.535
12	12.363	1299.100	1301.733	2.633					1299.867	1303.724	3.857	2.571	3.857	3.383
13	13.101	1299.100	1301.683	2.583					1300.221	1303.703	3.482	2.321	3.482	3.054
14	14.576	1299.100	1301.583	2.483					1300.752	1303.661	2.909	1.940	2.909	2.552
15	17.526	1299.100	1301.384	2.284					1301.342	1303.578	2.236	1.491	2.284	2.003
16	20.476	1299.100	1301.185	2.085					1301.814	1303.495	1.681	1.121	2.085	1.829

E.2.11: Protection Works

D/S protection:

D = 2*R – y2 = 5.17 m

Length of Launching apron = L = 1.5D = 7.755 m (take 9 m)

Thickness of apron = (2.25*D)/L = 1.293 m (take 1.5 m)

U/S protection:

D = 1.5*R – y1 = 4.92 m

Thickness = 1.5 m

Length of launching apron = (2.25*D)/t = 7.38 m (take 7.5 m)

Take a launching apron length of 3 m of thickness 1.5 m.

E.3: Design of Canal Head Regulator

E.3.1: Regulator width calculation

Design canal discharge = 0.24 m^3/s

Bed level = 1303.05 m

u/s HFL = 1306.23 m

Pond level = 1305.15 m

Crest level = 1303.55 m

Canal FSL = 1304.65 m

Cd_1 = 0.577

Cd_2 = 0.8

h = 1.1 m

h_1 = 0.5 m

therefore, by formula of submerged weir,

B = 0.063 m (take a minimum of 0.5 m width)

E.3.2: Determination of opening (x) during High Flood Condition

$x = \frac{Q}{0.62B\sqrt{(2g(HFL-FSL)}}$ = 0.139 m

Velocity (v) = 3.452 m/s

loss = $0.5\frac{v^2}{2g}$ = 0.304 m

TEL just u/s of regulator = 1306.281 m

TEL just d/s of regulator = 1306.281 – 0.304 = 1305.977 m

Head loss = 1305.977 – 1304.65 = 1.327 m

E.3.3: Hydraulic Jump Calculation

Table E.16: Hydraulic jump calculation of Canal head regulator

SN	Item	High Flood Flow	Pond Level Flow
1	discharge intensity (q)	0.480	0.480
2	u/s water level	1306.230	1305.150
3	d/s water level	1304.650	1304.650
4	u/s TEL	1305.977	1305.150
5	d/s TEL	1304.650	1304.650
6	Head Loss (HL)	1.327	0.500
7	d/s specific energy (Ef_2)	0.764	0.646
8	jump level = d/s TEL - Ef_2	1303.886	1304.004
9	u/s specific energy (Ef_1)	2.058	1.151
10	pre-jump depth (y1)	0.077	0.106
11	post-jump depth (y2)	0.743	0.615
12	length of floor = 5(y2-y1)	3.330	2.545
13	Initial Froude Number (F1)	7.173	4.441

d/s floor length = 4 m

d/s floor level = 1303.7 m

Since the floor level downstream is found to be higher than crest level, we adjust the downstream floor level to be equal to upstream riverbed level, i.e. 1303.05 m.

Assume a glacis length of 1.2 m.

E.3.4: Depth of Sheet pile

q = 0.48 cumecs/m

scour depth, R = $1.35\left(\frac{q^2}{1.2}\right)^{1/3}$ = 0.779 m (take 1 m)

RL of bottom of d/s cutoff = 1300.7 m

RL of bottom of u/s cutoff = 1299.48 m

d/s cutoff depth = 3 m

u/s cutoff depth = 3.57 m

E.3.5: Total Floor Length

$G_E = 1/6$

Maximum Static Head (H) = 2.53 m

d/s cutoff depth (d) = 3 m

$\lambda = 1.802$

$\alpha = 2.403$

Now, we get the floor length values as:

$b = \alpha.d = 7.210$ m

balancing u/s floor length = 7.21 – 4 – 1.2 = 3.8 m

E.3.6: Uplift Pressure Calculation

$\lambda_1 = 1.856$

$\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\left(\frac{0-\lambda_1}{\lambda_1}\right) = 100\%$

$\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\left(\frac{1-\lambda_1}{\lambda_1}\right) = 65.259\%$

$\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\left(\frac{2-\lambda_1}{\lambda_1}\right) = 47.529\%$

$\lambda_2 = 2.081$

$\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-2}{\lambda_2}\right) = 48.759\%$

$\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-1}{\lambda_2}\right) = 32.612\%$

$\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda_2-0}{\lambda_2}\right) = 0\%$

Table E.17: Corrected uplift pressure percentages of Canal head regulator

Point	Pressure	Correction for			Corrected Pressure
		Thickness	MI	Slope	
E1	100	0	0	0	100
D1	65.259	0	0	0	65.259
C1	47.529	4.966	3.511	0	56.006
E2	48.759	-8.073	-5.365	0	35.320
D2	32.612	0	0	0	32.612
C2	0	0	0	0	0

Table E.18: HGL calculation through uplift pressures of Canal head regulator

Condition of flow	u/s water level	d/s water level	H	HG line above Datum					
				u/s pile			d/s pile		
				ϕE1	ϕD1	ϕC1	ϕE2	ϕD2	ϕC2
No flow	1306.23	1303.7	2.53	2.53	1.651	1.417	0.894	0.825	0
				1306.23	1305.351	1305.117	1304.594	1304.525	1303.7
HFL with conc and ret	1306.23	1304.65	1.58	1.6	1.031	0.885	0.558	0.515	0
				1306.230	1305.681	1305.535	1305.208	1305.165	1304.650
PL with conc and ret	1305.15	1304.65	0.5	0.500	0.326	0.280	0.177	0.163	0
				1305.15	1304.976	1304.930	1304.827	1304.813	1304.650

E.3.7: Floor thickness and Protection Works

Length of d/s floor = 4 m

Maximum unbalanced head at d/s end will occur at No flow condition.

UBH = 0.894 m

Thickness = UBH/(G-1) = 0.894/1.24 = 0.721 m

UBH at 3 m from d/s end = $0.894 + \frac{1.417-0.894}{9} * 3 = 1.068$ m

Thickness = 0.861 m

Since the difference in thickness is not significant, we apply a uniform thickness of 0.9 m throughout the d/s floor.

Now, since the scour depth, u/s floor length and d/s floor length are very less as compared to the under sluice, we use a nominal length of launching apron in the d/s end.

Length of launching apron = 2 m

Thickness = 1.5 m

E.4: Design of Settling basin

E.4.1: Data

Canal Discharge = 0.24 m^3/s

Particle diameter = 0.5 mm

E.4.2: Design from Nomogram (Refer Fig 5.1, 5.2, 5.3)

Table E.19: Design calculations for settling basin

SN	Description	Design value calculation			
		Item	Value	Unit	Remarks
i	Dimensions	Q/A_s	0.017	m/s	for particle 0.5 mm
		A_s	14.118	m^2	
		L:B	10:1		assumed
		B	1.188	m	adopt 1.2 m
		L	12	m	
ii	bottom depth	Critical bottom velocity	0.24	m/s	
		depth	0.833	m	adopt 0.9 m
iii	scour depth	scour velocity	1.92	m/s	
		scour flow	0.24	m^3/s	
		scour depth	0.10417	m	adopt 0.2 m
iv	scour bed slope	V	1.92	m/s	
		n	0.015		
		R	0.36	m	
		slope (S_{sc})	1:310		from Manning's equation
v	flushing gate	depth (Y_g)	0.25	m	taking 1 m width; using $Q = 1.7bY_g^{1.5}$
		depth below sill	0.1	m	Y_g+0.05-Y_s
vi		approach slope	1:1.5		
vii		starting chainage	339.51	m	
viii		transition start	340	m	
ix		transition length	0.75	m	
x		RL	1301.32	m	

Therefore, the overall dimensions of the settling basin are 12 m x 1.2 m x 1.1 m.

APPENDIX F

DESIGN OF CANALS, DROPS, AQUEDUCTS AND OUTLETS

TABLE OF CONTENTS

LIST OF TABLES

F.1: Canal Cut & Fill Volume

Table F.1: Cut & Fill Volume calculation of Primary Canal

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
0+050	0.79	0	0	0	0	0	0
0+100	0.71	0	50	37.62	0	37.62	0
0+150	0.71	0	50	35.49	0	73.11	0
0+200	0.56	0	50	31.75	0	104.86	0
0+250	0.67	0	50	30.87	0	135.73	0
0+300	0.28	0	50	23.84	0	159.57	0
0+350	0	0.16	50	6.98	3.93	166.55	3.93
0+400	0.42	0	50	10.56	3.93	177.11	7.86
0+450	0.02	0	50	11.01	0	188.12	7.86
0+500	0	0.31	50	0.46	7.71	188.58	15.57
0+550	0.1	0	50	2.52	7.71	191.1	23.28
0+600	0.34	0	50	10.92	0	202.02	23.28
0+650	0.02	0	50	8.98	0	211	23.28
0+700	0.03	0	50	1.46	0	212.46	23.28
0+750	0.08	0	50	2.84	0	215.3	23.28
0+800	0.1	0	50	4.4	0	219.7	23.28
0+850	0.14	0	50	5.95	0	225.65	23.28
0+900	0.11	0	50	6.19	0	231.84	23.28
0+950	0.09	0	50	4.81	0	236.65	23.28
1+000	0.17	0	50	6.27	0	242.92	23.28
1+050	0.18	0	50	8.71	0	251.63	23.28
1+100	0.2	0	50	9.55	0	261.18	23.28
1+150	0.22	0	50	10.36	0	271.54	23.28
1+200	0.19	0	50	10.15	0	281.69	23.28
1+250	0.26	0	50	11.17	0	292.86	23.28
1+300	0.46	0	50	17.81	0	310.67	23.28
1+350	0.42	0	50	21.89	0	332.56	23.28
1+400	0.28	0	50	17.49	0	350.05	23.28
1+450	0.15	0	50	10.77	0	360.82	23.28
1+500	0.17	0	50	7.93	0	368.75	23.28
1+550	0.11	0	50	7.02	0	375.77	23.28
1+600	0.01	0	50	3.21	0	378.98	23.28
1+650	0.29	0	50	7.62	0	386.6	23.28
1+700	0.29	0	50	14.64	0	401.24	23.28
1+750	0.07	0	50	9.14	0	410.38	23.28
1+800	0.15	0	50	5.59	0	415.97	23.28
1+850	0.18	0	50	8.41	0	424.38	23.28
1+900	0	0.01	50	4.58	0.32	428.96	23.6
1+950	0.1	0	50	10.06	0.32	439.02	23.92
2+000	0.16	0	50	21.51	0	460.53	23.92
2+050	0.34	0	50	20.07	0	480.6	23.92
2+100	0.2	0	50	13.66	0	494.26	23.92
2+150	0.18	0	50	9.53	0	503.79	23.92
2+200	0.28	0	50	11.59	0	515.38	23.92
2+250	0.37	0	50	16.31	0	531.69	23.92
2+300	0.39	0	50	19	0	550.69	23.92

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
2+350	0.36	0	50	18.77	0	569.46	23.92
2+400	0.35	0	50	17.76	0	587.22	23.92
2+450	0.41	0	50	18.9	0	606.12	23.92
2+500	0.1	0	50	12.61	0	618.73	23.92
2+550	0.01	0	50	2.7	0	621.43	23.92
2+600	0.19	0	50	4.94	0	626.37	23.92
2+650	0.15	0	50	8.39	0	634.76	23.92
					Excess Cut Volume	610.84	

Table F.2: Cut & Fill Volume calculation of Secondary Canal-I

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
0+030	0.02	0	0	0	0	0	0
0+060	0.04	0	30	0.9	0	0.9	0
0+090	0.02	0	30	0.93	0	1.83	0
0+120	0.04	0	30	0.96	0	2.79	0
					Excess Cut Volume	2.79	

Table F.3: Cut & Fill Volume calculation of Secondary Canal-II

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
0+030	0.08	0	0	0	0	0	0
0+060	0	0.11	30	1.2	1.61	1.2	1.61
0+090	0	0.1	30	0	3.15	1.2	4.76
0+120	0.07	0	30	1.03	1.54	2.23	6.3
0+150	0.1	0	30	2.54	0	4.77	6.3
0+180	0.07	0	30	2.59	0	7.36	6.3
0+210	0.05	0	30	1.86	0	9.22	6.3
0+240	0.04	0	30	1.45	0	10.67	6.3
0+270	0.02	0	30	0.93	0	11.6	6.3
0+300	0	0	30	0.27	0.05	11.87	6.35
0+330	0.03	0	30	0.47	0.05	12.34	6.4
0+360	0.05	0	30	1.27	0	13.61	6.4
					Excess Cut Volume	7.21	

Table F.4: Cut & Fill Volume calculation of Secondary Canal-III

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
0+030	0.07	0	0	0	0	0	0
0+060	0	0.01	30	0.99	0.18	0.99	0.18
0+090	0.03	0	30	0.38	0.18	1.37	0.36
0+120	0.14	0	30	2.46	0	3.83	0.36
0+150	0.14	0	30	4.1	0	7.93	0.36
0+180	0.13	0	30	3.98	0	11.91	0.36
0+210	0.13	0	30	3.83	0	15.74	0.36
0+240	0.12	0	30	3.61	0	19.35	0.36
0+270	0.09	0	30	3.11	0	22.46	0.36
0+300	0.06	0	30	2.31	0	24.77	0.36
0+330	0.09	0	30	2.34	0	27.11	0.36
					Excess Cut Volume	26.75	

Table F.5: Cut & Fill Volume calculation of Secondary Canal-IV

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m^3)	Cumulative Fill (m^3)
0+30	0.43	0	0	0	0	0	0
0+60	0.64	0	30	16	0	16	0
0+90	0.74	0	30	20.62	0	36.62	0
0+120	0.65	0	30	20.81	0	57.43	0
0+150	0.56	0	30	18.2	0	75.63	0
0+180	0.48	0	30	15.57	0	91.2	0
0+210	0.54	0	30	15.27	0	106.47	0
0+240	0.64	0	30	17.65	0	124.12	0
0+270	0.46	0	30	16.48	0	140.6	0
					Excess Cut Volume	140.6	

F.2: Design of Outlet

Table F.6: Summary of design of outlets

Canal Outlet	Area (m^2)	Q (lps)	Q (m^3/s)	Depth (m)	Chainage (m)	RL of Bed (masl)	u/s FSL	Field ht	d/s FSL	Head (m)	Area (m^2)	Dia (cm)	Invert Level
0--1	54846.27	19.692	0.02	0.5	2+380	1293.533	1294.033	1293.295	1293.445	0.588	0.009	12	1293.385
1--1	6406.9	2.3	0.002	0.12	0+030	1300.043	1300.163	1299.92	1300.07	0.093	0.003	6	1300.04
1--2	6529.2	2.344	0.002	0.12	0+030	1300.043	1300.163	1299.92	1300.07	0.093	0.003	6	1299.883
1--3	22492	8.076	0.008	0.12	0+090	1298.726	1298.846	1298.618	1298.768	0.078	0.011	12	1298.506
2--1	10600	3.806	0.004	0.2	0+020	1296.991	1297.191	1296.865	1297.015	0.176	0.003	8	1296.891
2--2	17041.2	6.119	0.006	0.2	0+015	1297.001	1297.201	1296.382	1296.532	0.669	0.003	6	1296.921
2--3	22162	7.957	0.008	0.2	0+140	1295.785	1295.985	1295.772	1295.922	0.063	0.012	12	1295.862
2--4	21177.7	7.604	0.008	0.2	0+140	1295.785	1295.985	1295.772	1295.922	0.063	0.011	12	1295.862
2--5	44428.2	15.952	0.016	0.2	0+260	1295.541	1295.741	1295.186	1295.336	0.405	0.009	12	1295.401
3--1	33263	11.943	0.012	0.21	0+060	1295.495	1295.705	1294.931	1295.081	0.624	0.006	10	1295.385
3--2	30765	11.046	0.011	0.21	0+090	1293.817	1294.027	1293.424	1293.574	0.453	0.006	10	1293.707
3--3	60795.3	21.828	0.022	0.21	0+080	1293.837	1294.047	1293.808	1293.958	0.089	0.027	20	1293.627
3--4	45465.5	16.324	0.016	0.21	0+359	1291.075	1291.285	1291.032	1291.182	0.103	0.019	16	1290.905
4--1	21043.8	7.556	0.008	0.15	0+003	1293.322	1293.472	1293.153	1293.303	0.169	0.007	12	1293.132
4--2	17613.55	6.324	0.006	0.15	0+003	1293.322	1293.472	1293.153	1293.303	0.169	0.006	12	1293.132
4--3	15784.49	5.667	0.006	0.15	0+050	1291.381	1291.531	1290.895	1291.045	0.486	0.003	8	1291.231
4--4	13244.6	4.755	0.005	0.15	0+030	1291.422	1291.572	1291.385	1291.535	0.037	0.009	12	1291.232
4--5	14370.34	5.1596418	0.0051596	0.15	0+275	1289.718	1289.868	1289.701	1289.851	0.017	0.014	14	1289.508

F.3: Design of Sarda drops

F.3.1: At primary canal chainage 1+254.69 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 1.526 m
  U/S FSL = 1300.026 m
  D/S FSL = 1298.5 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1299.526 m
  D/S bed level = 1298 m
  Bed width = 0.2 m

- Design of crest:

Table F.7: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1300.05 m	u/s FSL + H_a
RL of Crest	1299.326 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	1.326 m	
Top width	0.633 m	$0.55d^{0.5}$

Table F.8: Crest design (Trial 2)

Top width (B_t)	0.63 m	assumed
H	0.741 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1300.05 m	u/s FSL + H_a
RL of Crest	1299.309 m	u/s Tel - H
Ht of crest above u/s floor	0.217 m	
Ht of crest above d/s floor (d)	1.309 m	
Top width	0.629 m	$0.55d^{0.5}$

Adopt, top width = 0.65 m
Thickness at base (B_1) = 1.03 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5}$ = 5.32 m
  x = $0.25 * (H.Hl)^{2/3}$ = 0.27 m
  RL of cistern bed = RL of d/s bed – x = 1297.729 m

- Design of impervious floor:

Table F.9: Design values of impervious floor

Seepage head, H_s =d	1.31 m
Bligh's coefficient	6
Required creep length	7.856 m
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.75 m
Vertical length of creep = 2(d1+d2)	2.3 m
Required length of horizontal impervious floor	5.556 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	2.926 m
Provide,	
d/s impervious floor length	7 m
u/s impervious floor length	2 m

- Uplift pressure and thickness
  Total creep length = 12.33 m
  Provide minimum thickness of 0.30 m for u/s floor.
  Max unbalanced head under d/s toe of crest wall = 1.02 m
  Required Thickness = h/(G-1) = 0.83 m
  Provide 0.85 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.3.2: At primary canal chainage 1+343.49 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 1.53 m
  U/S FSL = 1298.323 m
  D/S FSL = 1296.793 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1297.823 m
  D/S bed level = 1296.293 m
  Bed width = 0.2 m

- Design of crest:

Table F.10: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1298.347 m	u/s FSL + H_a
RL of Crest	1297.623 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	1.33 m	
Top width	0.634 m	$0.55d^{0.5}$

Table F.11: Crest design (Trial 2)

Top width (B_t)	0.63 m	assumed
H	0.741 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1298.347 m	u/s FSL + H_a
RL of Crest	1297.606 m	u/s Tel - H
Ht of crest above u/s floor	0.217 m	
Ht of crest above d/s floor (d)	1.313 m	
Top width	0.63 m	$0.55d^{0.5}$

Adopt, top width = 0.65 m
Thickness at base (B_1) = 1.03 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5} = 5.32$ m
  $x = 0.25 * (H.Hl)^{2/3} = 0.27$ m
  RL of cistern bed = RL of d/s bed – x = 1296.021 m

- Design of impervious floor:

Table F.12: Design values of impervious floor

Seepage head, H_s =d	1.31 m
Bligh's coefficient	6
Required creep length	7.880 m
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.6 m
Vertical length of creep = 2(d1+d2)	2 m
Required length of horizontal impervious floor	5.88 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	2.92 m
Provide,	
d/s impervious floor length	7 m
u/s impervious floor length	2 m

- Uplift pressure and thickness
  Total creep length = 12.03 m
  Provide minimum thickness of 0.30 m for u/s floor.
  Max unbalanced head under d/s toe of crest wall = 1.05 m
  Required Thickness = h/(G-1) = 0.84 m
  Provide 0.85 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.3.3: At primary canal chainage 1+942.4 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 1.3 m
  U/S FSL = 1295.598 m
  D/S FSL = 1294.298 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1295.098 m
  D/S bed level = 1293.798 m
  Bed width = 0.2 m

- Design of crest:

Table F.13: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1295.622 m	u/s FSL + H_a
RL of Crest	1294.898 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	1.1 m	
Top width	0.577 m	$0.55d^{0.5}$

Table F.14: Crest design (Trial 2)

Top width (B_t)	0.58 m	assumed
H	0.735 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1295.622 m	u/s FSL + H_a
RL of Crest	1294.887 m	u/s Tel - H
Ht of crest above u/s floor	0.211	
Ht of crest above d/s floor (d)	1.089 m	
Top width	0.574 m	$0.55d^{0.5}$

Adopt, top width = 0.6 m
Thickness at base (B_1) = 0.91 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5}$ = 4.886 m
  x = $0.25 * (H.Hl)^{2/3}$ = 0.242 m
  RL of cistern bed = RL of d/s bed – x = 1292.556 m

- Design of impervious floor:

Table F.15: Design values of impervious floor

Seepage head, H_s =d	1.09 m
Bligh's coefficient	6
Required creep length	6.537 m
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.6 m
Vertical length of creep = 2(d1+d2)	2 m
Required length of horizontal impervious floor	4.537 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	2.7 m
Provide,	
d/s impervious floor length	6 m
u/s impervious floor length	2 m

- Uplift pressure and thickness
  Total creep length = 10.91 m
  Provide minimum thickness of 0.30 m for u/s floor.
  Max unbalanced head under d/s toe of crest wall = 0.85 m
  Required Thickness = h/(G-1) = 0.68 m
  Provide 0.7 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.3.4: At primary canal chainage 2+092.64 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 0.396 m
  U/S FSL = 1294.003 m
  D/S FSL = 1293.607 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1293.503 m
  D/S bed level = 1293.107 m
  Bed width = 0.2 m

- Design of crest:

Table F.16: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using Q = $1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1297.027 m	u/s FSL + H_a
RL of Crest	1293.303 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	0.196 m	
Top width	0.244 m	$0.55d^{0.5}$

Table F.17: Crest design (Trial 2)

Top width (B_t)	0.24 m	assumed
H	0.673 m	using Q = $1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1294.027 m	u/s FSL + H_a
RL of Crest	1293.355 m	u/s Tel - H
Ht of crest above u/s floor	0.148 m	
Ht of crest above d/s floor (d)	0.248 m	
Top width	0.274 m	$0.55d^{0.5}$

Adopt, top width = 0.3 m
Thickness at base (B_1) = 0.46 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5}$ = 2.58 m
  x = $0.25 * (H.Hl)^{2/3}$ = 0.1 m
  RL of cistern bed = RL of d/s bed – x = 1293.004 m

- Design of impervious floor:

Table F.18: Design values of impervious floor

Seepage head, H_s =d	0.25 m
Bligh's coefficient	6
Required creep length	1.485
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.6 m
Vertical length of creep = 2(d1+d2)	2 m
Required length of horizontal impervious floor	-0.515 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	1.796 m
Provide,	
d/s impervious floor length	4 m
u/s impervious floor length	2 m

- Uplift pressure and thickness
  Total creep length = 8.46 m
  Provide minimum thickness of 0.30 m for u/s floor.
  Max unbalanced head under d/s toe of crest wall = 0.2 m
  Required Thickness = h/(G-1) = 0.16 m
  Provide 0.18 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.3.5: At primary canal chainage 2+503.58 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 1.666 m
  U/S FSL = 1292.789 m
  D/S FSL = 1291.123 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1292.289 m
  D/S bed level = 1290.623 m
  Bed width = 0.2 m

- Design of crest:

Table F.19: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1292.813 m	u/s FSL + H_a
RL of Crest	1292.089 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	1.466 m	
Top width	0.666 m	$0.55d^{0.5}$

Table F.20: Crest design (Trial 2)

Top width (B_t)	0.66 m	assumed
H	0.744 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1292.813 m	u/s FSL + H_a
RL of Crest	1292.069 m	u/s Tel - H
Ht of crest above u/s floor	0.22 m	
Ht of crest above d/s floor (d)	1.446 m	
Top width	0.661 m	$0.55d^{0.5}$

Adopt, top width = 0.7 m
Thickness at base (B_1) = 1.1 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5}$ = 5.57 m
  x = $0.25 * (H.Hl)^{2/3}$ = 0.29 m
  RL of cistern bed = RL of d/s bed – x = 1290.334 m

- Design of impervious floor:

Table F.21: Design values of impervious floor

Seepage head, H_s =d	1.45 m
Bligh's coefficient	6
Required creep length	8.675 m
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.6 m
Vertical length of creep = 2(d1+d2)	2 m
Required length of horizontal impervious floor	6.675 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	3.066 m
Provide,	
d/s impervious floor length	7 m
u/s impervious floor length	2 m

- Uplift pressure and thickness
  Total creep length = 12.1 m
  Provide minimum thickness of 0.30 m for u/s floor.
  Max unbalanced head under d/s toe of crest wall = 1.14 m
  Required Thickness = h/(G-1) = 0.92 m
  Provide 0.95 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.3.6: At primary canal chainage 2+552.27 m

- Data:
  Fully supply discharge of canal = 0.2411 m^3/s
  Drop Height = 2.265 m
  U/S FSL = 1291.026 m
  D/S FSL = 1288.761 m
  U/S and D/S fully supply depth = 0.5 m
  U/S bed level = 1290.526 m
  D/S bed level = 1288.261 m
  Bed width = 0.2 m

- Design of crest:

Table F.22: Crest design (Trial 1)

Top width (B_t)	0.5 m	assumed
H	0.724 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1291.05 m	u/s FSL + H_a
RL of Crest	1290.326 m	u/s Tel - H
Ht of crest above u/s floor	0.2 m	
Ht of crest above d/s floor (d)	2.066 m	
Top width	0.79 m	$0.55d^{0.5}$

Table F.23: Crest design (Trial 2)

Top width (B_t)	0.79 m	assumed
H	0.758 m	using $Q = 1.84LH^{1.5}(H/BT)^{1/6}$
Approach Velocity (v_a)	0.689 m/s	
Approach Velocity Head (H_a)	0.024 m	
u/s TEL	1291.05 m	u/s FSL + H_a
RL of Crest	1290.292 m	u/s Tel - H
Ht of crest above u/s floor	0.234 m	
Ht of crest above d/s floor (d)	2.032 m	
Top width	0.784 m	$0.55d^{0.5}$

Adopt, top width = 0.8 m
Thickness at base (B_1) = 1.39 m

- Design of cistern:
  $L_c = 5(H.Hl)^{0.5}$ = 6.551 m
  x = $0.25 * (H.Hl)^{2/3}$ = 0.358 m
  RL of cistern bed = RL of d/s bed – x = 1287.902 m

➢ Design of impervious floor:

Table F.24: Design values of impervious floor

Seepage head, H_s =d	2.03 m
Bligh's coefficient	6
Required creep length	12.191 m
u/s cut off pile depth, d1=D1/3	0.167 m
d/s cut off pile depth, d2=D2/2	0.25 m
Provide,	
d1	0.4 m
d2	0.6 m
Vertical length of creep = 2(d1+d2)	2 m
Required length of horizontal impervious floor	10.191 m
Min. length of d/s impervoius floor = 2(D+L)+Hl	3.665 m
Provide,	
d/s impervious floor length	8 m
u/s impervious floor length	3 m

➢ Uplift pressure and thickness
Total creep length = 14.39 m
Provide minimum thickness of 0.30 m for u/s floor.
Max unbalanced head under d/s toe of crest wall = 1.51 m
Required Thickness = h/(G-1) = 1.21 m
Provide 1.25 m thick cement concrete floor over laid by 0.2 m brick pitching.

F.4: Design of Aqueduct

F.4.1: Data of Aqueduct 1
Full supply discharge = 0.241 m^3/s
FSL = 1297.9 m
Canal bed width = 0.2 m
Canal section slope = 1:01
Canal bed = 1296.132 m
Canal water depth = 0.5 m
HFDS = 2.83 m
HFL = 1295.73 m
Riverbed level = 1294.841 m
HFD = 0.889 m

Bed width = 0.2 m
Flume width = 0.15 m
Contraction 2:1 length = 0.05 m
Expansion 3:1 length = 0.075 m

F.4.2: Design values of Aqueduct 1

- At 4-4:
  trapezoidal area at 4-4 = 0.35 m^2
  velocity = 0.689 m/s
  velocity head = 0.024 m
  RL of bed at 4-4 = 1296.132 m
  TEL at 4-4 = 1297.924 m

- At 3-3:
  bed width = 0.15 m
  area = 0.075 m^2
  velocity = 3.213 m/s
  velocity head = 0.526 m
  expansion loss from 3-3 to 4-4 = 0.151 m
  TEL = 1298.075 m
  RL of water surface = 1297.549 m
  RL of bed = 1297.049 m

- At 2-2:
  Length = 8 m
  n = 0.016
  A = 0.075 m^2
  Perimeter = 1.15 m
  R = 0.065 m
  velocity in trough = 3.213 m/s
  head loss = $\frac{n^2v^2L}{R^{\frac{4}{3}}}$ = 0.806 m
  TEL at 2-2 = 1298.880 m
  water surface at 2-2 = 1298.354 m
  RL of bed at 2-2 = 1297.854 m

- At 1-1:
  contraction loss from 1-1 to 2-2 = 0.100 m
  TEL = 1298.981 m
  water surface = 1298.956 m
  bed RL = 1298.457 m

Adopt:

- 5 cm thick outer walls
- freeboard of 0.1 m with provision of escape throughout aqueduct
- 10 cm thick bottom slab

F.4.3: Data of Aqueduct 2
Full supply discharge = 0.241 m^3/s
FSL = 1296.79 m
Canal bed width = 0.2 m
Canal section slope = 1:01
Canal bed = 1296.13 m
Canal water depth = 0.66 m
HFDS = 2.83 m
HFL = 1295.35 m
Bed width = 0.2 m
Flume width = 0.15 m
Contraction 2:1 length = 0.05 m
Expansion 3:1 length = 0.075 m

F.4.4: Design values of Aqueduct 2

- At 4-4:
  trapezoidal area at 4-4 = 0.568 m^2
  velocity = 0.424 m/s
  velocity head = 0.009 m
  RL of bed at 4-4 = 1296.13 m
  TEL at 4-4 = 1296.799 m

- At 3-3:
  bed width = 0.15 m
  area = 0.099 m^2
  velocity = 2.434 m/s
  velocity head = 0.302 m
  expansion loss from 3-3 to 4-4 = 0.088 m
  TEL = 1296.887 m
  RL of water surface = 1296.585 m
  RL of bed = 1295.925 m

- At 2-2:
  Length = 18.35 m
  n = 0.016
  A = 0.099 m^2
  Perimeter = 1.47 m
  R = 0.067 m
  velocity in trough = 2.434 m/s
  head loss = $\frac{n^2 v^2 L}{R^{\frac{4}{3}}}$ = 1.016 m
  TEL at 2-2 = 1297.903 m
  water surface at 2-2 = 1297.601 m
  RL of bed at 2-2 = 1296.941 m

- At 1-1:
  contraction loss from 1-1 to 2-2 = 0.059 m

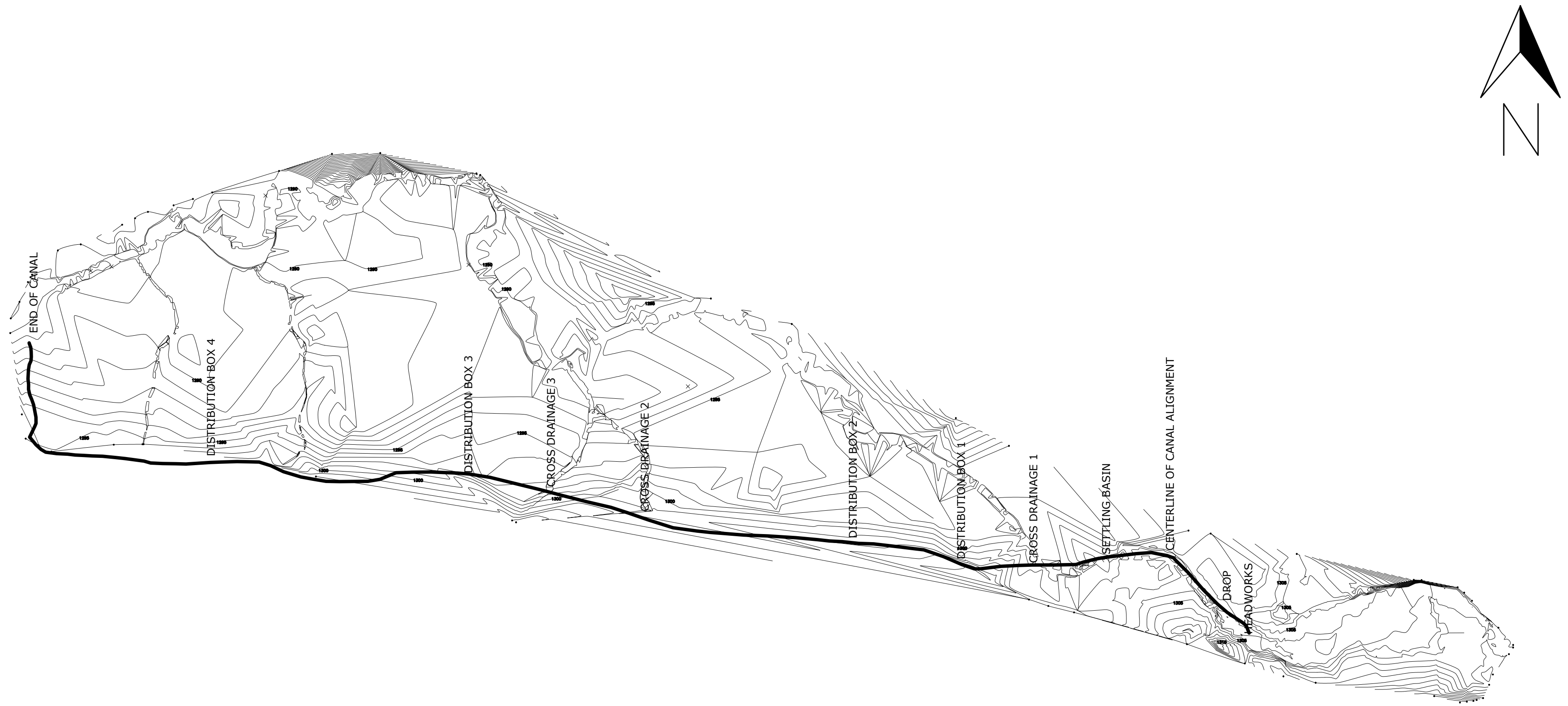
TEL = 1297.962 m
water surface = 1297.952 m
bed RL = 1297.292 m

Adopt:

- 5 cm thick outer walls
- freeboard of 0.1 m with provision of escape throughout aqueduct
- 10 cm thick bottom slab

APPENDIX G

DRAWINGS



TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
PULCHOWK CAMPUS

LAYOUT OF PROJECT

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Abhinav Kafle
Asian Chudali
Ayush Karki
Ayus Acharya

Supervisor:
Dr. Prem C. Jha

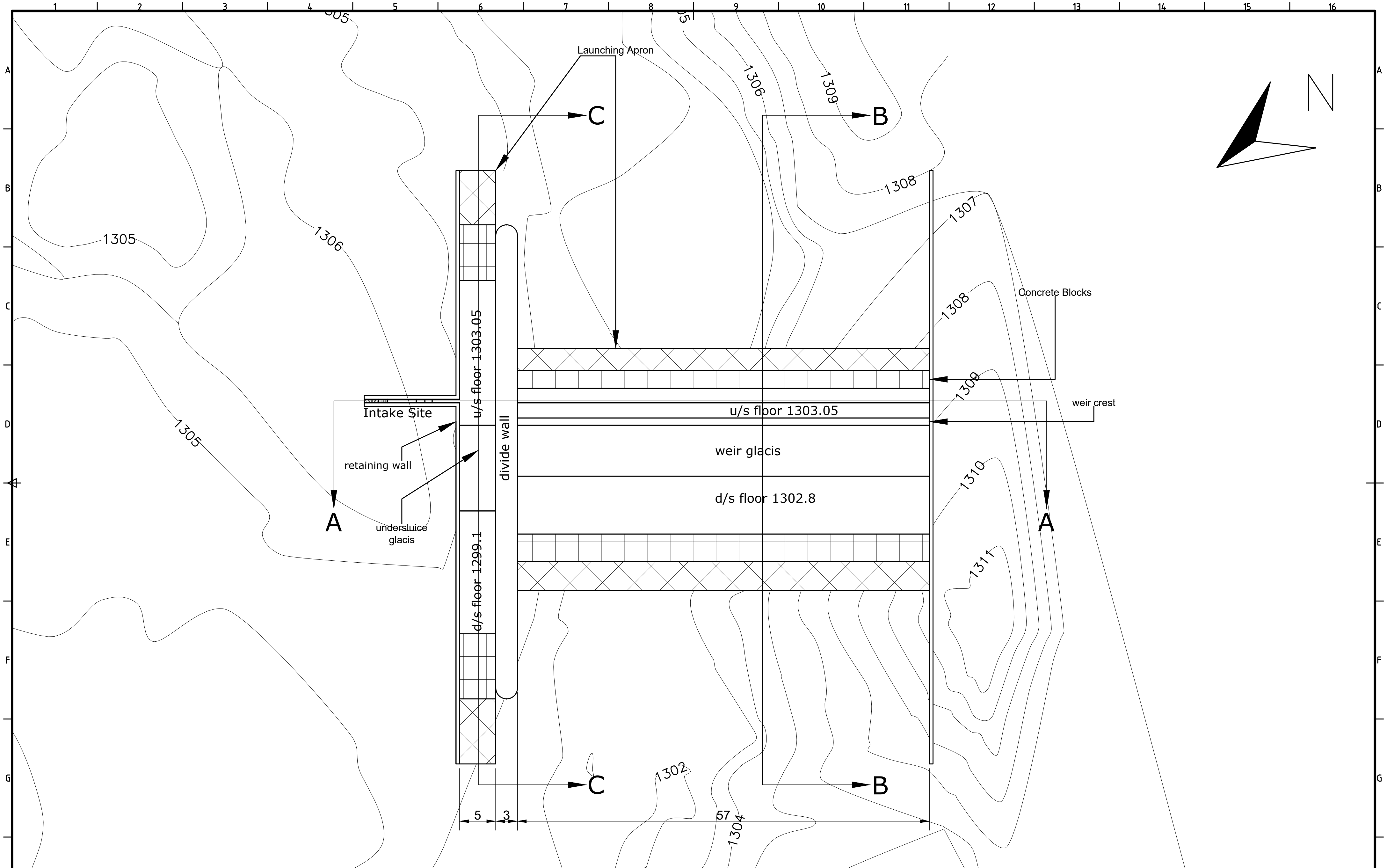
Examiners:
Dr. Vishnu P. Pandey
Dr. Pawan Bhattarai

Scale: 1:12500

Date: 2080/12/27

Pre-feasibility Study of Irrigation Canal at Manohara River, Changunarayan

1



TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
PULCHOWK CAMPUS

HEADWORK PLAN

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Ayush Karki
Ayus Acharya

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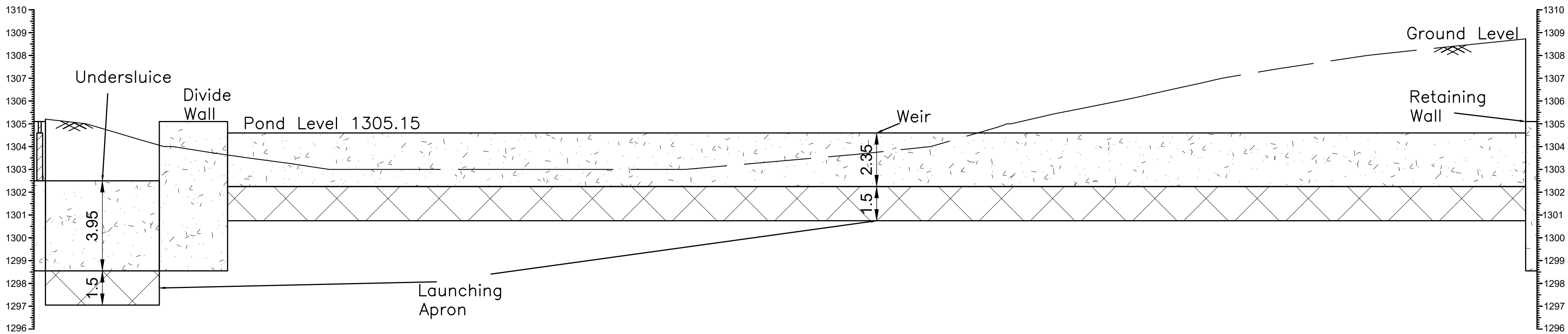
Scale:	1:666.7	Date:	2080/12/27

Pre-feasibility Study of Irrigation Canal at Manohara River, Changunarayan

2



SECTION AT A–A

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
PULCHOWK CAMPUS

WEIR AND UNDER SLUICE SECTION

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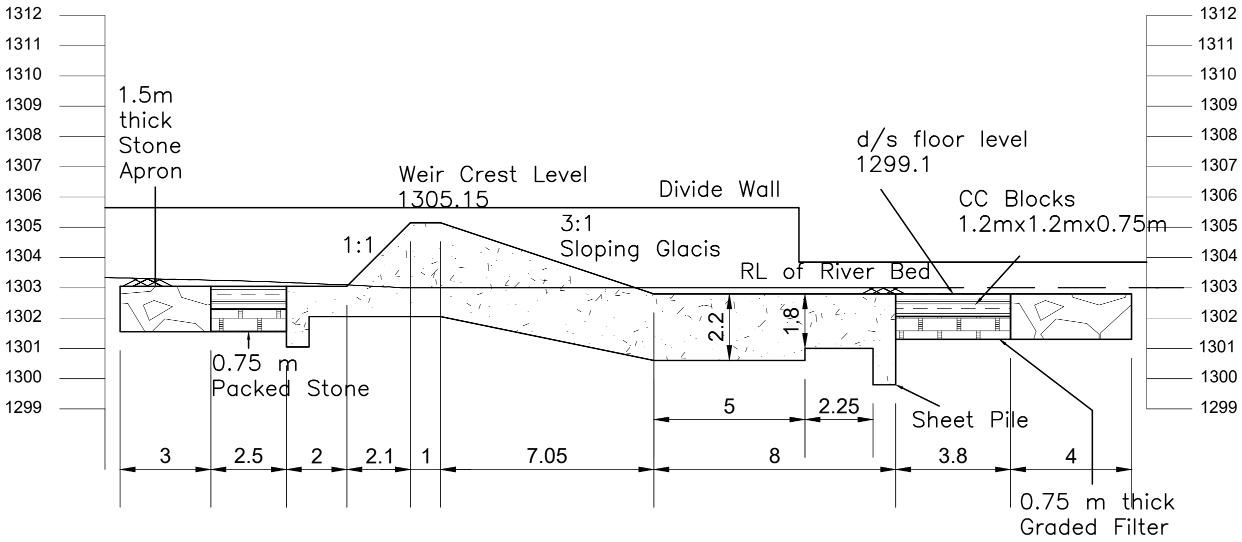
Scale: 1:250

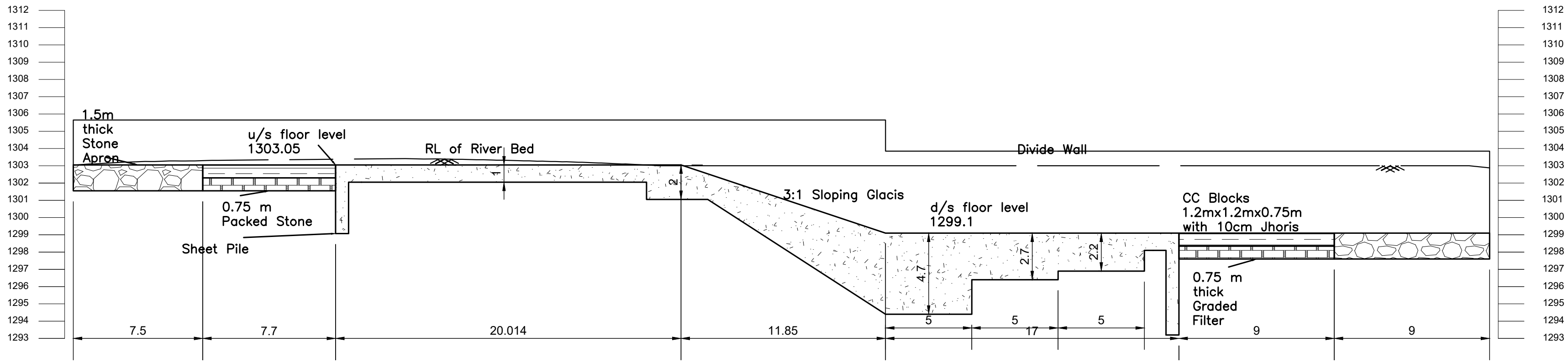
Date: 2080/12/27

Pre–feasibility Study of Irrigation Canal at Manohara River, Changunarayan

3



WEIR SECTION AT B–B (SCALE = 1:250)



UNDERSLUICE SECTION AT C–C (SCALE = 1:400)

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INSTITUTE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
PULCHOWK CAMPUS

WEIR AND UNDER SLUICE SECTION

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Supervisor:
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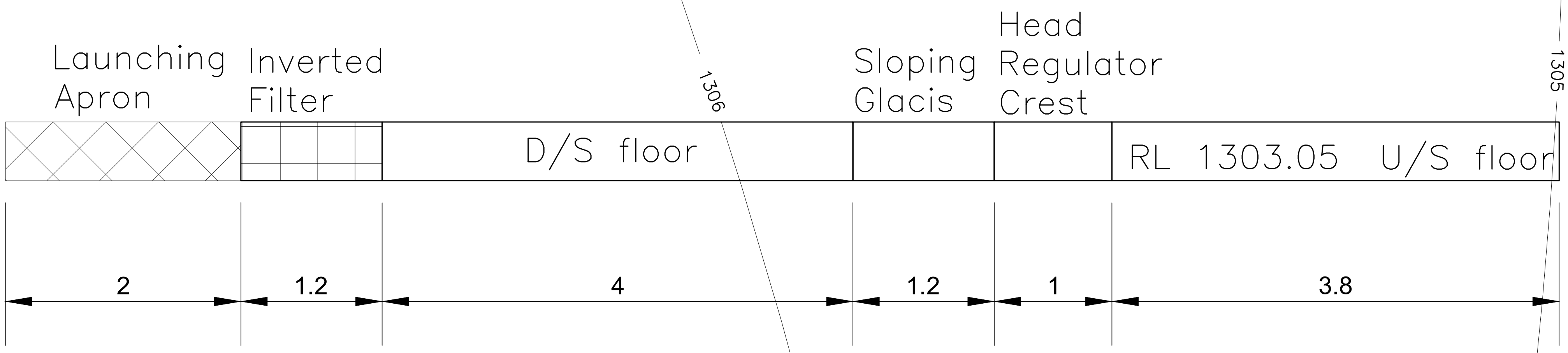
Examiners:
Dr. Vishnu P. Pandey
Dr. Pawan Bhattarai

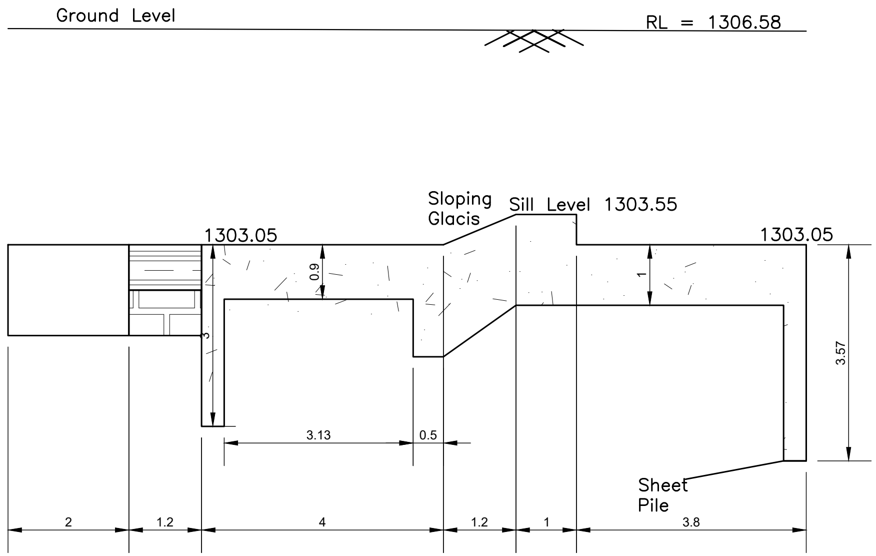
Scale: –

Date: 2080/12/27

Pre–feasibility Study of Irrigation Canal at Manohara River, Changunarayan



CANAL HEAD REGULATOR PLAN (SCALE = 1:50)



CANAL HEAD REGULATOR SECTION VIEW
(SCALE = 1:125)

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INSTITUTE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
PULCHOWK CAMPUS

CANAL HEAD REGULATOR
PLAN AND SECTION

Prepared By:
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Aashish Bhattarai
Abhinav Kafle
Asian Chudali
Ayush Karki
Ayus Acharya

Supervisor:
Dr. Prem C. Jha

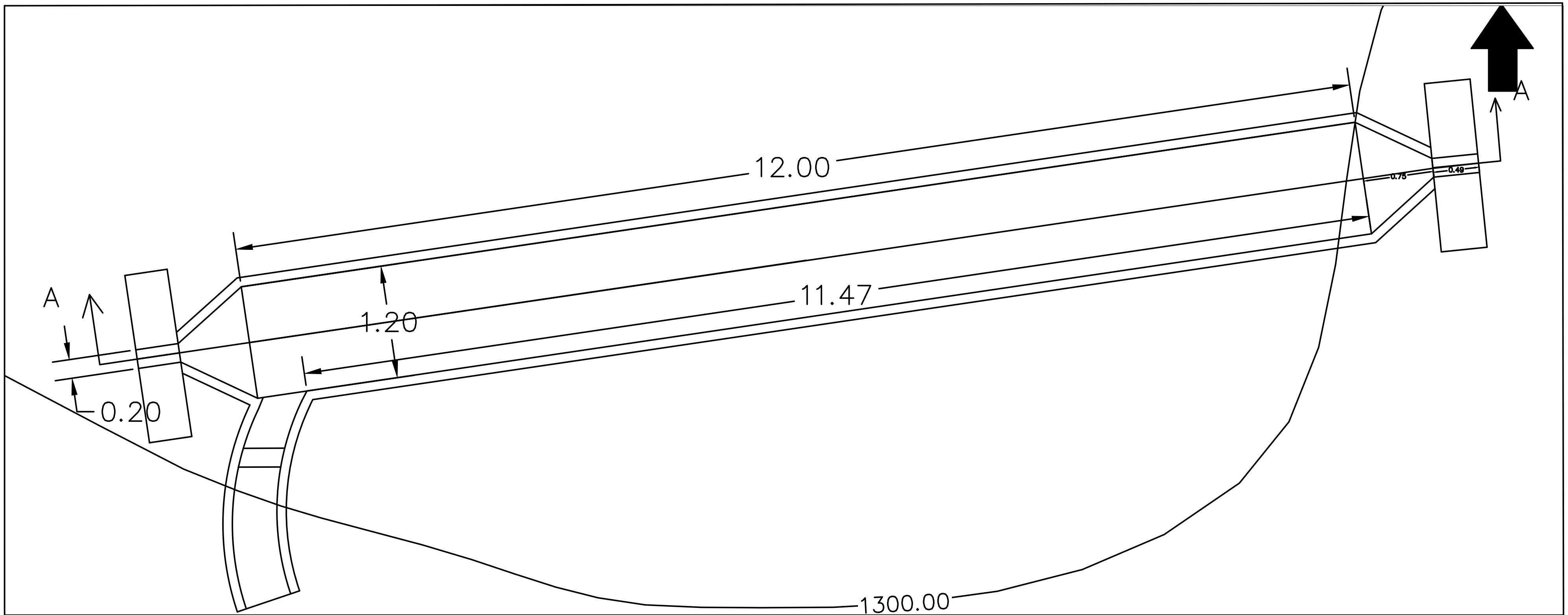
Examiners: Dr. Vishnu P. Pandey, Dr. Pawan Bhattarai

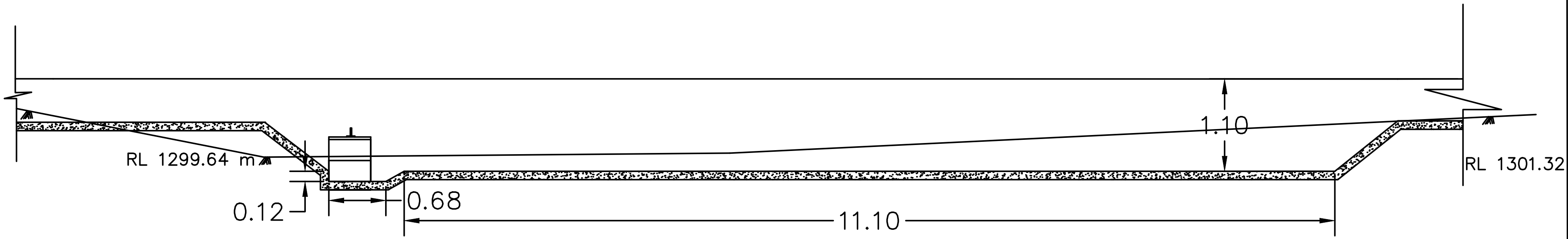
Scale: –

Date: 2080/12/27

Pre–feasibility Study of Irrigation Canal at Manohara River, Changunarayan

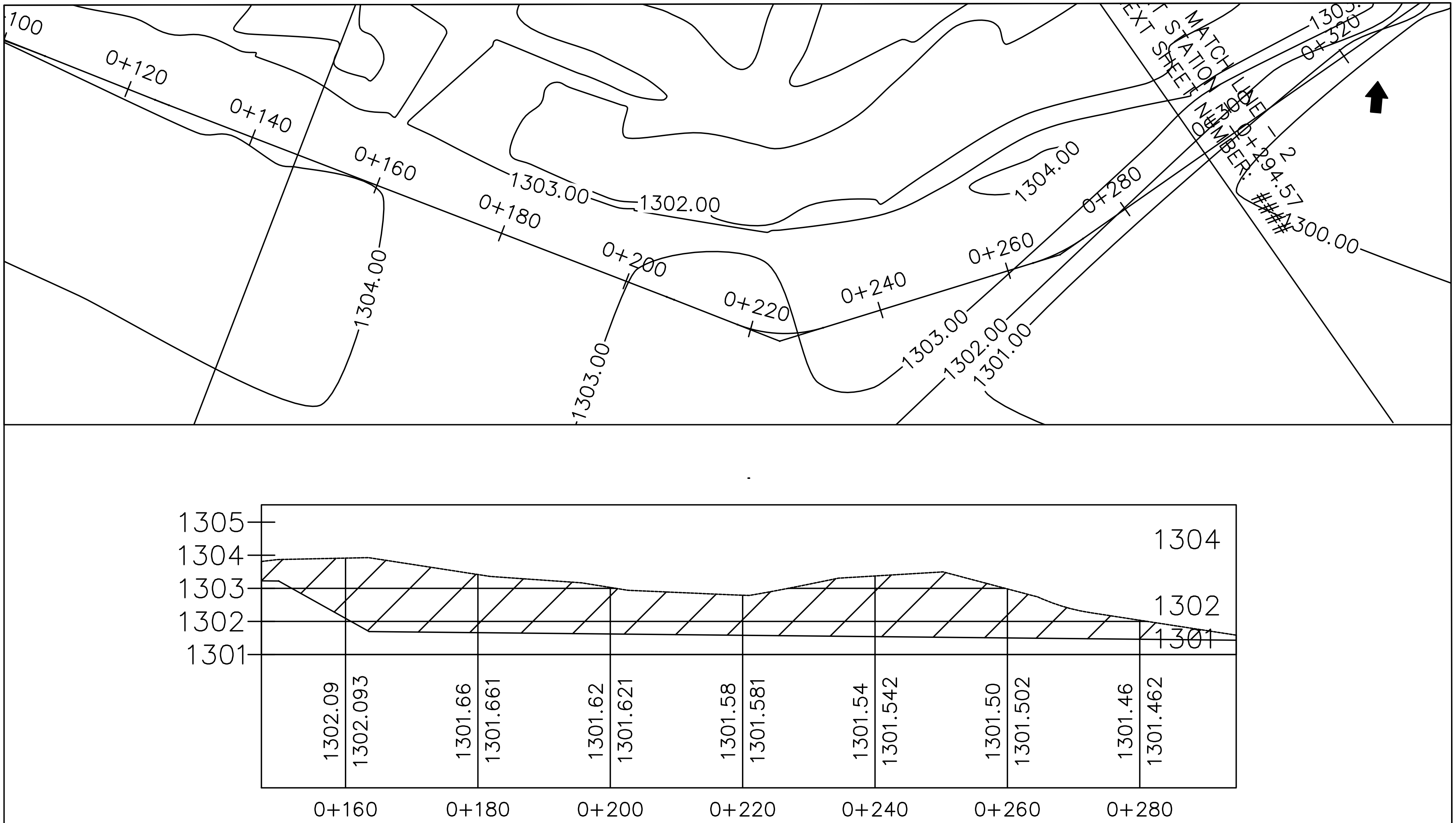
5



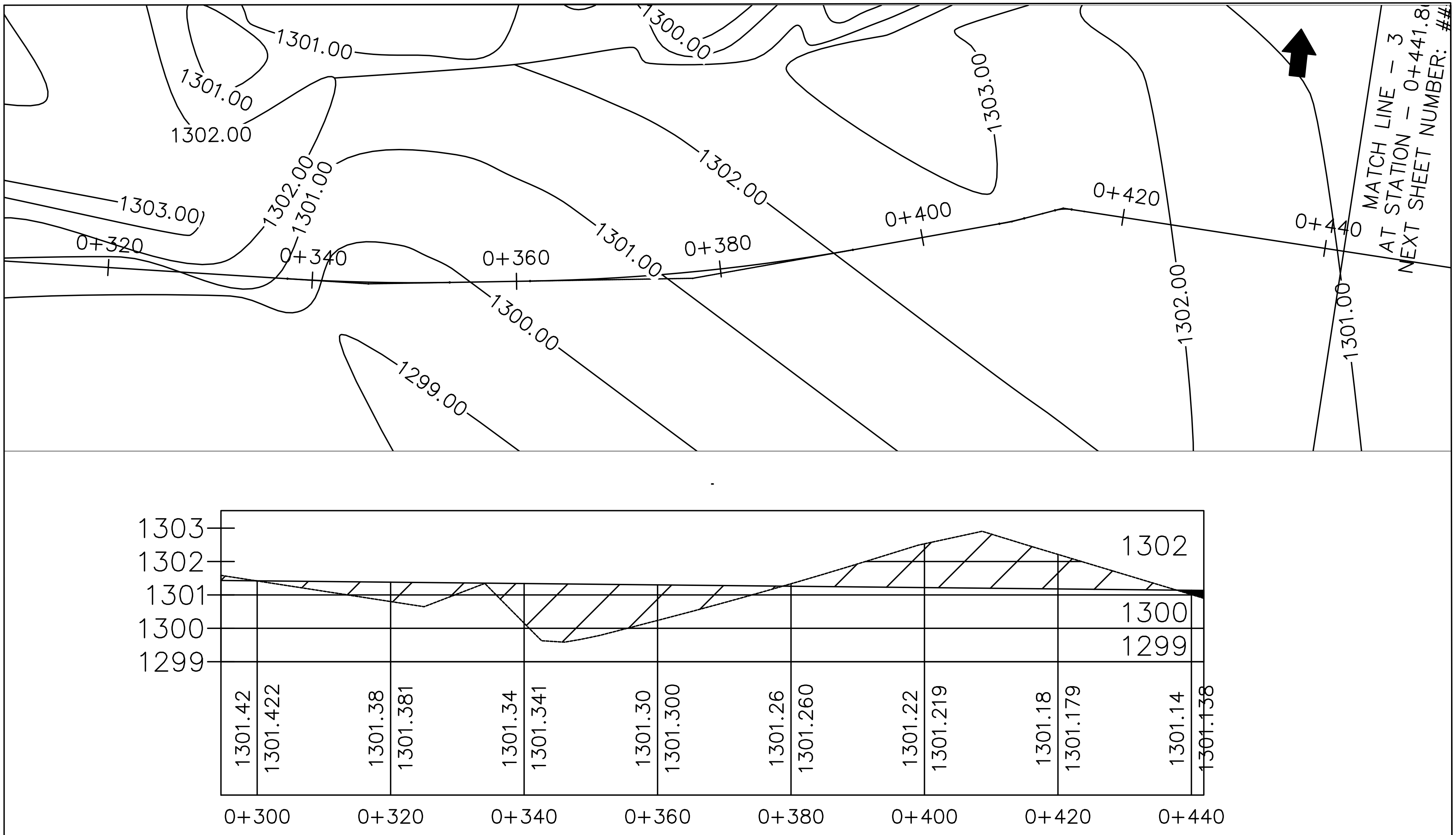


Section at A—A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	SETTLING BASIN	Prepared By: Aakriti Chalise Aashish Bhattarai Abhinav Kafle	Asian Chudali Ayus Acharya Ayush Karki	Supervisor: Dr. Prem C. Jha Scale: 1:100	Examiner : Dr. Vishnu P. Pandey Dr. Pawan Bhattarai Date: 2080/12/27	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	PRIMARY CANAL LONGITUDINAL PROFILE	Prepared By: Aakriti Chalise, Aashish Bhattarai, Abhinav Kafle, Asian Chudali, Ayus Acharya, Ayush Karki	Scale: 1:1500 Date: 2080/12/28	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan	7



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	PRIMARY CANAL LONGITUDINAL PROFILE	Prepared By: Aakriti Chalise, Aashish Bhattarai, Abhinav Kafle, Asian Chudali, Ayus Acharya, Ayush Karki	Scale: 1:1500 Date: 2080/12/28	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan	8

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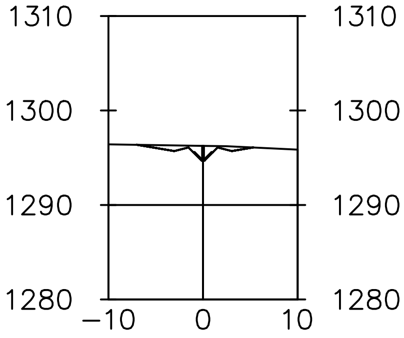

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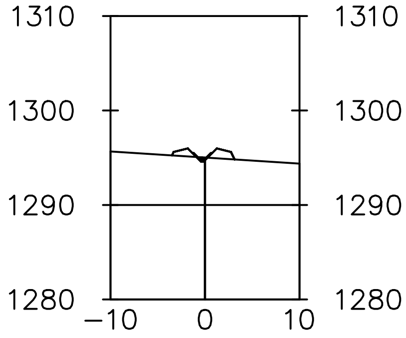

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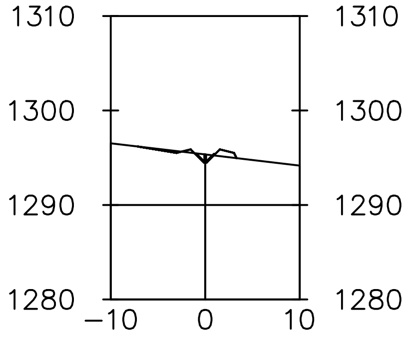

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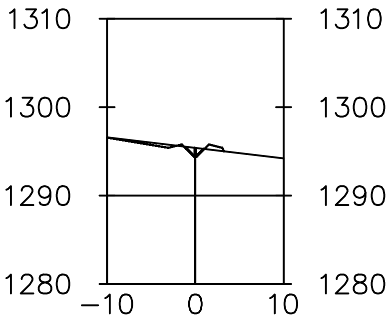

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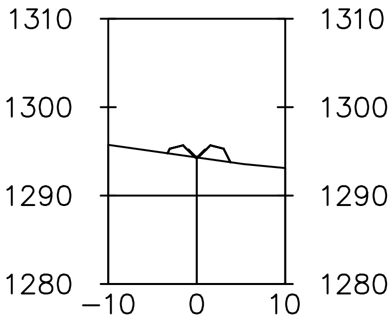

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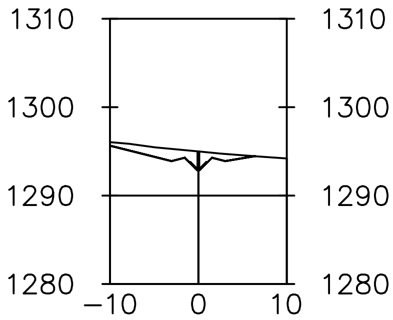
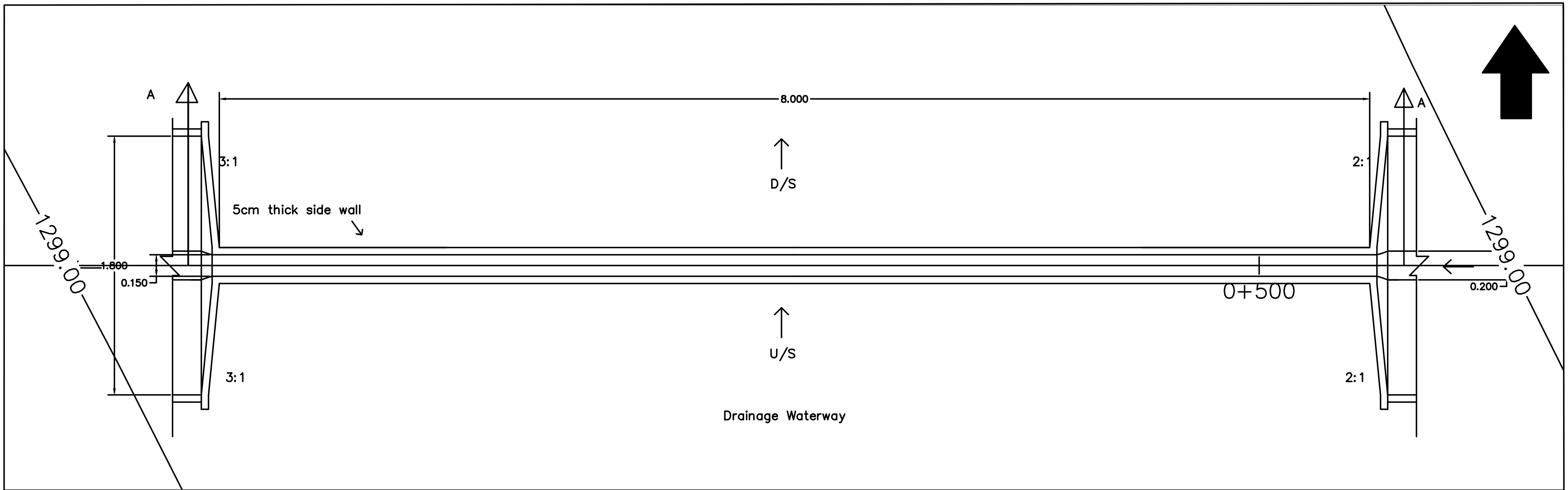
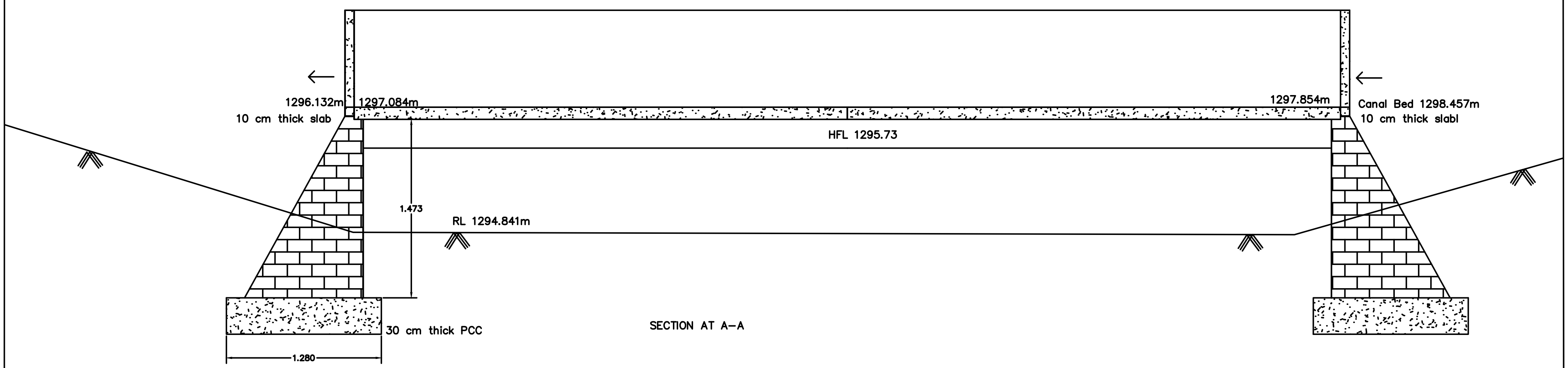


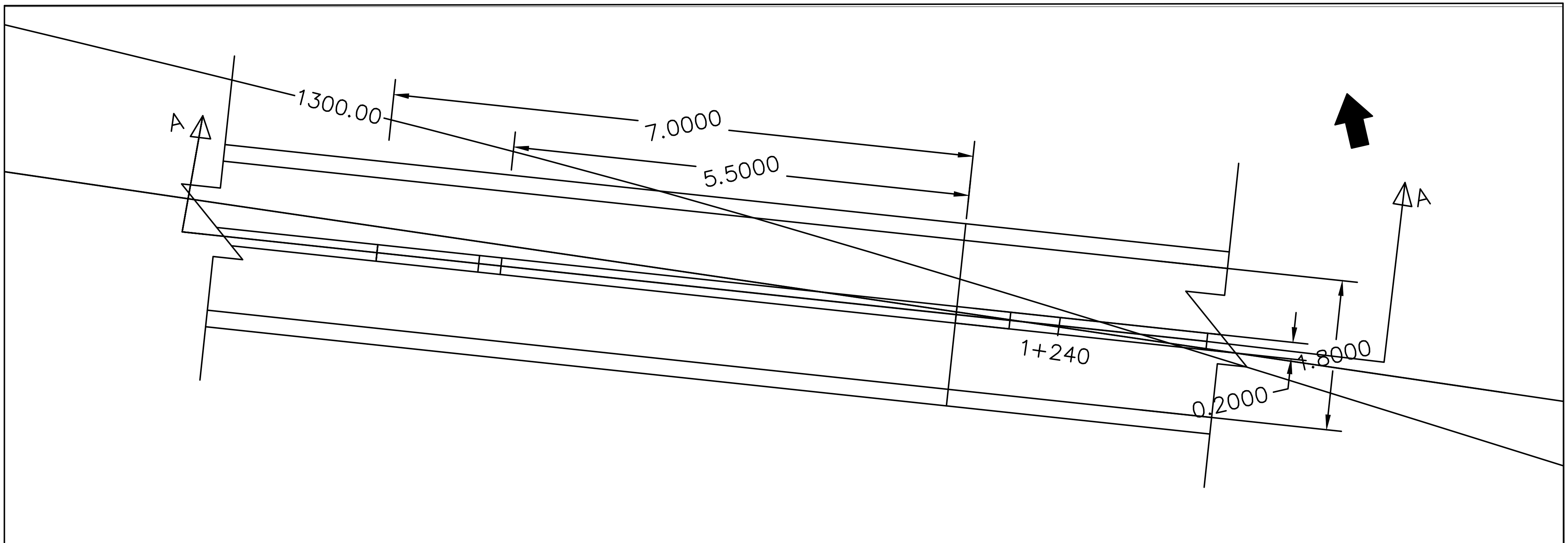
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	CANAL CROSS SECTIONAL PROFILE	Prepared By: Aakriti Chalise Aashish Bhattarai Abhinav Kafle	Asian Chudali Ayus Acharya Ayush Karki	Supervisor: Dr. Prem C. Jha Scale: 1:700	Examiner : Dr. Vishnu P. Pandey Dr. Pawan Bhattarai Date: 2080/12/27	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan

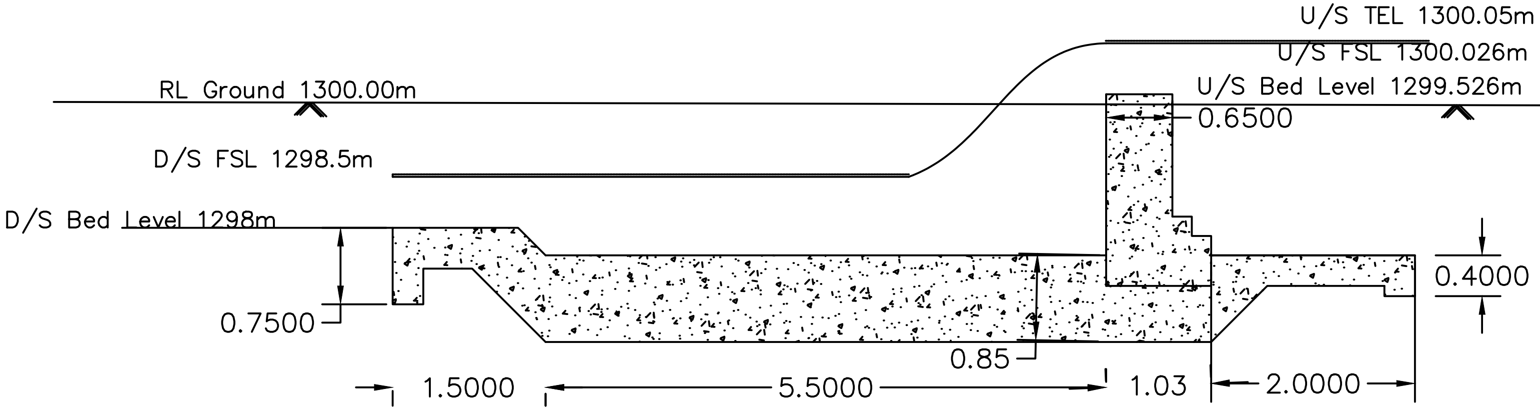




TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	AQUEDUCT	Prepared By: Aakriti Chalise Aashish Bhattarai Abhinav Kafle Asian Chudali Ayus Acharya Ayush Karki	Supervisor: Dr. Prem C. Jha Scale: 1:100	Examiner : Dr. Vishnu P. Pandey Dr. Pawan Bhattarai Date: 2080/12/27	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan

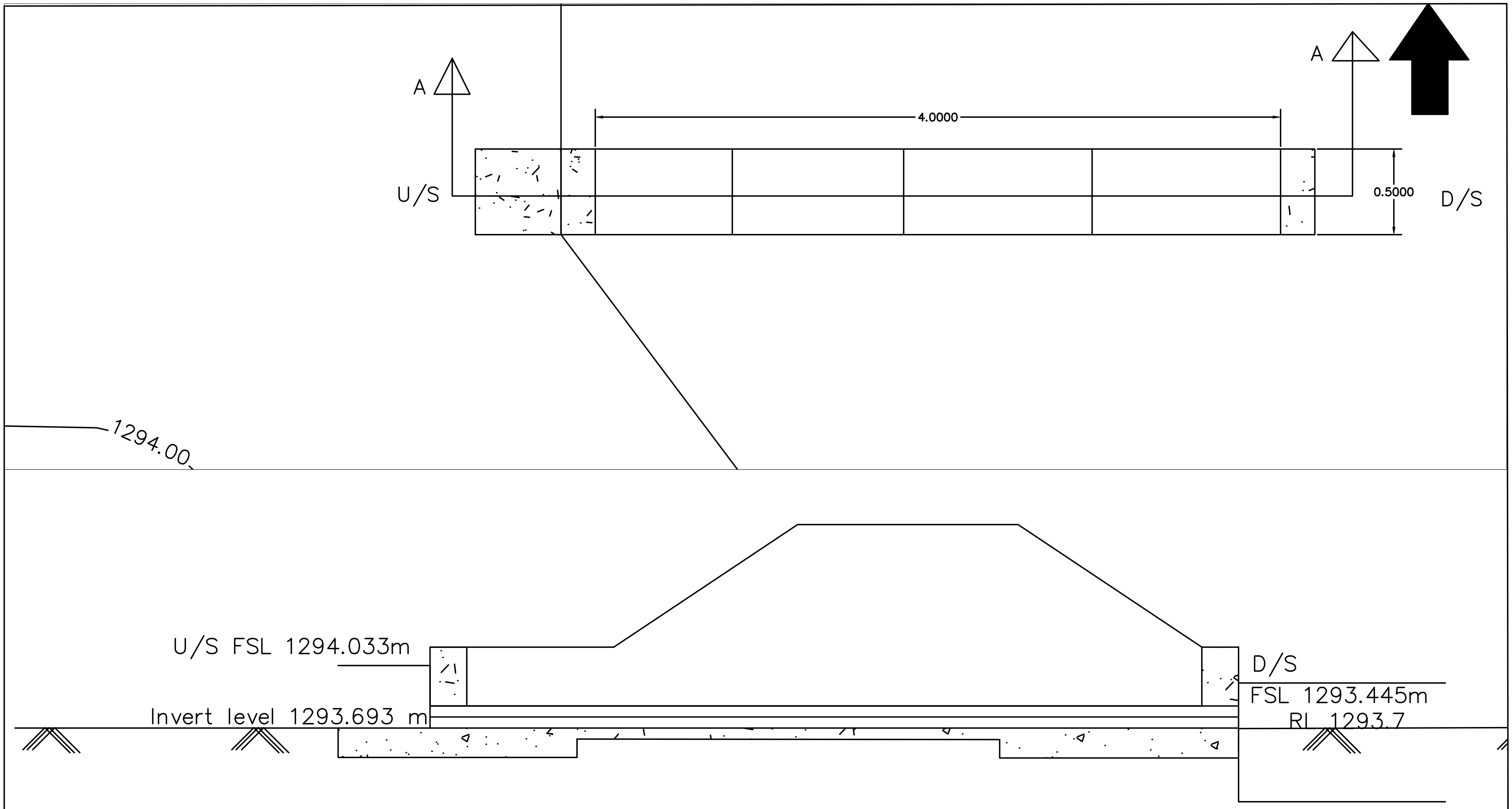
10





SECTION AT A–A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	DROP	Prepared By: Aakriti Chalise Aashish Bhattarai Abhinav Kafle Asian Chudali Ayus Acharya Ayush Karki	Supervisor: Dr. Prem C. Jha Scale: 1:50	Examiner : Dr. Vishnu P. Pandey Dr. Pawan Bhattarai Date: 2080/12/27	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan	11



SECTION AT A–A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING DEPARTMENT OF CIVIL ENGINEERING PULCHOWK CAMPUS	OUTLET	Prepared By: Aakriti Chalise Aashish Bhattarai Abhinav Kafle Asian Chudali Ayus Acharya Ayush Karki	Supervisor: Dr. Prem C. Jha Scale: 1:50	Examiner : Dr. Vishnu P. Pandey Dr. Pawan Bhattarai Date: 2080/12/27	Pre-Feasibility Study of Irrigation Canal at Manohara River, Chagunarayan	12

APPENDIX H

COST ESTIMATION AND ECONOMIC ANALYSIS

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LIST OF TABLES

LIST OF FIGURES

H.1: Quantity Estimation

H1.1: Earthwork estimation for weir

Since the terrain is uneven and the waterway of weir stretches up to 57 m, a cumulative cut and fill estimate is done assuming the right bank as the starting point and left end of the weir as the end of chainage.

Table H.1: Cut & Fill volume estimation for weir

Chainage	Cut area	Fill area	Distance	Cut Volume	Fill Volume	Cumulative Cut (m3)	Cumulative Fill (m3)
0+000	180.153	0	0	0	0	0	0
0+028.5	59.345	0	28.5	3412.8465	0	3412.8465	0
0+057	169.026	0	57	3254.28675	0	6667.13325	0

H1.2 Bill of Quantities

SN	Item of work	No	L	H	B	Qty	Unit
			WEIR				
1	Earthwork in excavation	1				6667.133	m^3
2	M20 PCC for RCC	1	42.388		57	2416.116	m^3
3	Sheet pile (u/s)	1	0.75	2	57	85.5	m^3
4	Sheet pile (d/s)	1	0.75	3	57	128.25	m^3
5	launching apron (u/s)	1	3	1.5	57	256.5	m^3
6	launching apron (d/s)	1	4	1.5	57	342	m^3
7	gravel	1	3.8	0.75	57	162.45	m^3
8	stone	1	2.5	0.75	57	106.875	m^3
9	concrete block	1	6.3	0.75	57	269.325	m^3
			UNDERSLUICE				
10	Earthwork in excavation	1	328.218		5	1641.09	m^3
11	M20 PCC for RCC	1	105.061		5	525.305	m^3
12	Sheet pile (u/s)	1	0.75	3.57	5	13.388	m^3
13	Sheet pile (d/s)	1	0.75	4.94	5	18.525	m^3
14	launching apron (u/s)	1	7.5	1.5	5	56.25	m^3
15	launching apron (d/s)	1	9	1.5	5	67.5	m^3
16	gravel	1	9	0.75	5	33.75	m^3
17	stone	1	7.7	0.75	5	28.875	m^3
18	concrete block	1	16.7	0.75	5	62.625	m^3
			CANAL HEAD REGULATOR				
19	Earthwork in excavation	1	64.251		0.5	32.126	m^3
20	M15 PCC for RCC	1	11.454		0.5	5.727	m^3
21	Sheet pile (u/s)	1	0.75	3.57	0.5	1.339	m^3
22	Sheet pile (d/s)	1	0.75	3	0.5	1.125	m^3

23	launching apron (d/s)	1	2	1.5	0.5	1.5	m^3
24	gravel	1	1.2	0.75	0.5	0.45	m^3
25	concrete block	1	1.2	0.75	0.5	0.45	m^3
26	Iron Gate	1	0.5	2.5		1.25	m^2
	DIVIDE WALL						
27	Earthwork in excavation	1	22.37		65.564	1466.667	m^3
28	M15 PCC for RCC	1	65.564	6.55	3	1288.333	m^3
	SETTLING BASIN						
29	Earthwork in excavation	1		5.695	1.2	6.834	m^3
30	Earthwork in filling	1		0.325	1.2	0.39	m^3
31	PCC M15						
	Floor	1	12	0.2	1.2	2.88	m^3
	Side Walls	2	12	1.1	0.2	5.28	m^3
32	Flush gate	1	0.6	0.25		0.15	m^2
	CANAL						
33	Net cut volume					788.19	m^3
34	Canal lining (M15 concrete)					942.9	m^3
	AQUEDUCT						
35	Earthwork in excavation	2	2.582		1.8	9.295	m^3
36	Earthwork in filling	2	0.752		1.8	2.707	m^3
37	First class b/w in 1:3 c-s mortar	4	0.8838		1.8	6.363	m^3
38	M20 PCC in abutment	4	1.28	0.3	1.8	2.765	m^3
39	M15 PCC in aqueduct channel						
	slab	2	8	0.1	0.25	0.400	m^3
	side walls	4	8	1	0.05	1.600	m^3
	DROP						
40	Earthwork in excavation	6	15.436		0.6	55.570	m^3
41	PCC M15	6	5.961		0.6	21.460	m^3
	OUTLET						
42	HDP pipes	lump sum				4.2	kg
43	Reinforcements					7.028	m3
						55169.800	kg

Table H.2: BOQ of Manohara Irrigation Project

Here, reinforcements are done on the footing and stabilizing part of the structures like sheet pile wall. The floors and walls of settling basin and aqueducts and all drops are reinforced too. Reinforcement volume is taken to be 2.5% of overall dry concrete volume.

H1.3: Justification of Lined Canal

Annual Benefit		
a. Seepage		
Seepage Loss	0.084	cum
Area of wetted perimeter for lined channel in	0.453	sqm
Seepage Loss in lined channel	0.029	cum
Saving in water loss	0.055	cum
Annual revenue saved	11086	Rs
b. Maintenance		
Annual maintenance cost of unlined channel	1500	Rs
Saving in maintenance cost	600	Rs
Total annual benefit	11686	Rs
Annual Cost		
Area of lining per km	452.548	sqm
Cost of lining	100	Rs/sqm
Interest rate	10	%
Capital recovery factor	5315.61	Rs
Benefit-Cost Ratio	2.198	

Therefore, it is justifiable to use a lined canal.

H.2: Unit Rate Analysis

H.2.1: Unit Rate Analysis of M15 PCC (1:2:4) for RCC per m^3

Table H.3: M15 PCC for RCC

A	Materials				
	Cement	6.4	bags	700	4480
	40 mm aggregates	0.52	cum	3461	1799.72
	20 mm aggregates	0.22	cum	3603	792.66
	10 mm aggregates	0.11	cum	3108	341.88
	Sand	0.445	cum	2508	1116.06
	Water	150	lits	0.3	45
				Sub-Total	8575.32
B	Labourers				
	Skilled	1	nos	1200	1200
	Unskilled	4	nos	900	3600
				Sub-Total	4800
C	Plants, equipment and tools = 3% of unskilled labourers				108
				Total	13483.32
D	Contractors overhead and profit				2022.498
				Grand Total	15505.818

H.2.2: Unit Rate Analysis of M20 PCC (1:1.5:3) for RCC per m^3

Table H.4: M20 PCC for RCC

A	Materials				
	Cement	8	bags	700	5600
	20 mm aggregates	0.57	cum	3603	2053.71
	10 mm aggregates	0.29	cum	3108	901.32
	Sand	0.425	cum	2508	1065.9
	Water	200	lits	0.3	60
				Sub-Total	9680.93
B	Labourers				
	Skilled	0.8	nos	1200	960
	Unskilled	7	nos	900	6300
				Sub-Total	7260
C	Plants, equipment and tools = 3% of unskilled labourers				189
				Total	17129.93
D	Contractors overhead and profit				2569.490
				Grand Total	19699.420

H.2.3: Unit Rate Analysis of first-class brickwork in 1:3 cement sand mortar per m^3

Table H.5: 1st class brickwork in 1:3 cement sand mortar

A	Materials				
	First Class Brick	530	nos	17.69	9375.7
	Cement	2.6	bags	700	1820
	Sand	0.27	cum	2508	677.16
	Water	150	lits	0.3	45
				Sub-Total	11917.86
B	Labourers				
	Skilled	1.5	nos	1200	1800
	Unskilled	2.2	nos	900	1980
				Sub-Total	3780
C	Plants, equipment and tools = 3% of unskilled labourers				59.4
				Total	15757.26
D	Contractors overhead and profit				2363.589
				Grand Total	18120.849

H.2.4: Unit Rate Analysis of stonework in 1:3 cement sand mortar per m^3

Table H.5: stonework in 1:3 cement sand mortar

A	Materials				
	Block Stone	1	cum	2720	2720
	Bond Stone	0.1	cum	4587	458.7
	Cement	3.88	bags	700	2716
	Sand	0.42	cum	2508	1053.36
	Water	120	lits	0.3	36
				Sub-Total	6984.06
B	Labourers				
	Skilled	1.5	nos	1200	1800
	Unskilled	5	nos	900	4500
				Sub-Total	6300
C	Plants, equipment and tools = 3% of unskilled labourers				135
				Total	13419.06
D	Contractors overhead and profit				2012.859
				Grand Total	15431.919

H.2.5: Unit Rate Analysis of reinforcement works per MT

Table H.5: Reinforcement works per MT

A	Materials (0.5% loss)				
	Reinforcement	1050	kg	99	103950
	Binding Wire	10	kg	106	1060
				Sub-Total	105010
B	Labourers				
	Skilled	12	nos	1200	14400
	Unskilled	12	nos	900	10800
				Sub-Total	25200
C	Plants, equipment and tools = 3% of unskilled labourers				324
				Total	130534
D	Contractors overhead and profit				19580.1
				Grand Total	150114.1

H.3: Cost Estimate

Table H.6: Total cost of the project

SN	Description	Quantity	Unit	Unit Rate	Amount
1	Earthwork in excavation	10666.905	m^3	746	7957511.13
2	Earthwork in filling	3.097	m^3	533	1650.701
3	M20 PCC	3189.849	m^3	19699.42	62838175.19
4	M15 PCC	2271.044	m^3	15505.818	35214394.93
5	Reinforcement works	55.171	MT	150114.1	8281945.011

6	Concrete Blocks (M20)	332.4	m^3	19699.42	6548087.208
7	Gravel	196.65	m^3	3603	708529.95
8	Stone	135.75	m^3	2720	369240
9	Iron Gate	1.25	m^2	8300	10375
10	Flush Gate	0.15	m^2	8300	1245
11	First class brickwork	6.363	m^3	18120.849	115302.9622
12	Stonework for Launching apron	723.75	m^3	15431.919	11168851.38
13	HDP pipes	4.2	kg	287	1205.4
	Sub Total (A)				133216513.9
	contingency				19982477.08
	work charge establishment				2664330.277
	Sub Total (B)				155863321.2
	VAT				20262231.76
	Grand Total				176125553

H.4: Economic Analysis

H.4.1: Payback Period Calculation

Table H.7: Cash flow of project

SN	Cost	PW of Cost	Revenue	PW of Revenue	Cumulative Cost	Cumulative Benefit	Net Benefit
1	88062776.487	80057069.534	0	0	80057069.534	0	-80057069.534
2	88062776.487	72779154.122	0	0	152836223.656	0	-152836223.656
3	176125.553	132325.735	25370143.200	19060964.087	152968549.390	19060964.087	-133907585.303
4	176125.553	120296.123	25370143.200	17328149.170	153088845.513	36389113.257	-116699732.256
5	176125.553	109360.111	25370143.200	15752862.882	153198205.624	52141976.139	-101056229.485
6	176125.553	99418.283	25370143.200	14320784.438	153297623.907	66462760.577	-86834863.330
7	176125.553	90380.257	25370143.200	13018894.944	153388004.165	79481655.521	-73906348.643
8	176125.553	82163.870	25370143.200	11835359.040	153470168.035	91317014.561	-62153153.474
9	176125.553	74694.428	25370143.200	10759417.309	153544862.462	102076431.870	-51468430.593
10	176125.553	67904.025	25370143.200	9781288.463	153612766.487	111857720.332	-41755046.155
11	176125.553	61730.932	25370143.200	8892080.421	153674497.419	120749800.753	-32924696.666
12	176125.553	56119.029	25370143.200	8083709.473	153730616.448	128833510.226	-24897106.222
13	176125.553	51017.299	25370143.200	7348826.794	153781633.747	136182337.020	-17599296.727
14	176125.553	46379.363	25370143.200	6680751.631	153828013.110	142863088.651	-10964924.459
15	176125.553	42163.057	25370143.200	6073410.573	153870176.167	148936499.224	-4933676.943
16	176125.553	38330.052	25370143.200	5521282.339	153908506.219	154457781.564	549275.345
17	176125.553	34845.502	25370143.200	5019347.581	153943351.721	159477129.145	5533777.424
18	176125.553	31677.729	25370143.200	4563043.256	153975029.450	164040172.401	10065142.951
19	176125.553	28797.935	25370143.200	4148221.142	154003827.385	168188393.543	14184566.158
20	176125.553	26179.941	25370143.200	3771110.129	154030007.326	171959503.671	17929496.345

H.4.2: Breakeven Analysis:

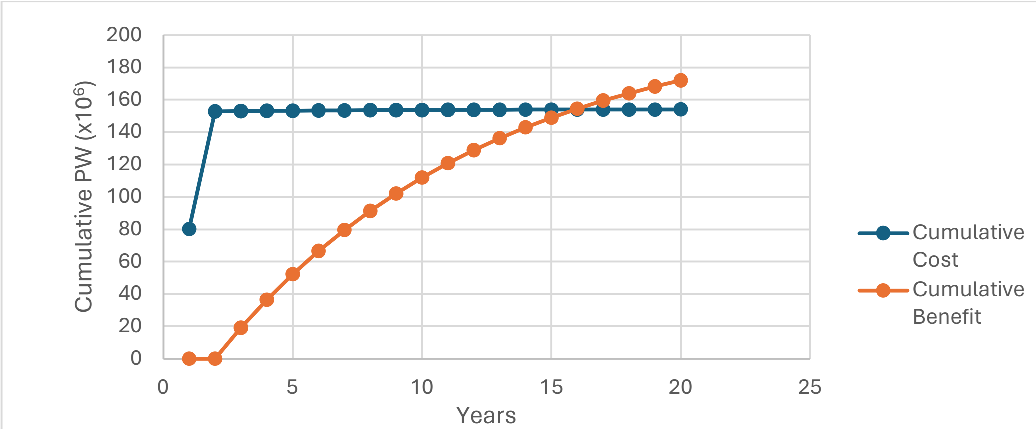

Fig H.1: Breakeven Analysis Chart

Thus, the breakeven point occurs at:

$$15 + \frac{0 - (-4933676.943)}{549273.345 - (-4933676.943)} = 15.9\ yrs$$

H.4.3: Benefit Cost Ratio Analysis:

Cumulative Benefit = Rs. 171959503.671

Cumulative Cost = Rs. 154030007.326

BCR = cum. Cost/ cum. Benefit = 1.116 > 1

So, this project is economically feasible.