HYDRAULIC DESIGN OF SLUICE GATES FOR FLOOD REGULATION IN THE POKORIA RIVER, MORIGAON DISTRICT.

A dissertation submitted in

partial fulfilment of the requirement for the award of the degree of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(With specialization in Water Resources Engineering)

Under

ASSAM SCIENCE AND TECHNOLOGY UNIVERSITY

SESSION: 2022-2024

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I hereby certify that the work presented in the dissertation entitled "***Hydraulic Design of Sluice Gates for Flood Regulation in the Pokoria River, Morigaon District***" is accorded for the award of the Degree of Master of Technology in Civil Engineering with specialization in Water Resources Engineering submitted in the Department of Civil Engineering, Assam Engineering College , Guwahati, Assam, in authentic record of my work carried out under the guidance of Prof. Bipul Talukdar, Department of Civil Engineering, Assam Engineering College, Guwahati.

The matter embodied in this project has not been submitted by me for the award of any other degree.

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ACKNOWLEDGEMENT

I would like to express my sincere appreciation to my supervisor Prof. Bipul Talukdar, Department of Civil Engineering of Assam Engineering College for his extensive support, and encouragement throughout the project work. I am highly indebted for his constant guidance and supervision as well as for providing full co-operation throughout the study and preparing the finalized dissertation. Working under him has indeed been a great experience and inspiration for me.

I express my gratitude to Dr. Jayanta Pathak, Professor and Head of Department of Civil Engineering of Assam Engineering College and also towards the entire fraternity of the Department of Civil Engineering of Assam Engineering College.

I cannot help myself without thanking Assam Engineering College, which provided us the required infrastructure and comforts all throughout this course and my project in particular. I also thank all the faculty members and staff of Department of Civil Engineering, Assam Engineering College for their free exchange of idea and lending their helping hand whenever I needed them.

I would like to express my deepest gratitude and sincere thanks to Er. Dipankar Roy, AEE, Morigaon Water Resource Division, Morigaon, Assam for kind cooperation in data collection in related to Pokoria River and all the staff members of the office concerned.
I extend heartfelt gratitude to my family for their unwavering support, understanding, and encouragement throughout this journey. Their patience and belief in me have been instrumental in achieving this milestone.

I also express profound gratitude to God for His guidance, strength, and blessings that have illuminated my path and enabled me to overcome challenges during this endeavour. His grace has been a constant source of inspiration and motivation.

Lastly, I thank all my classmate and well-wishers for their constant encouragement, valuable advice and inspiration during the course of my study.

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ABSTRACT

The local, national, and worldwide media frequently features stories about floods in its headlines. One of the biggest and trickiest issues hydrologists are currently dealing with is the computation of floods. Flood control, structural measure design and construction, and appropriate flood mitigation measures are critical to the best possible use of water resources. Accurate and trustworthy data are needed for flood estimation utilizing statistical and/or deterministic approaches in all such hydrologic analysis and design situations. To ensure the safety of structures, the design flood estimate must also account for costs and refrain from overdesigning. Keeping in mind the inundation through Pokoria River during monsoon a hydraulic structure is designed in MS Excel by cell referencing uses Khosla's theory and the Hydraulic Jump theory to compute the hydraulic parameters of a barrage that are determined in account of surface flow, subsurface flow, and the makeup of the foundation soil.

This study presents the results and discussion on the design and implementation of sluices in the Pokoria River, focusing on hydrological, bathymetric, and hydraulic analyses to enhance flood management and irrigation. Over 21 years (2002-2023), rainfall data analysis revealed significant seasonal and annual variability, with peak daily rainfall predicted using the Gumbel method. Bathymetric surveys detailed riverbed topography, identifying critical areas for sediment deposition and erosion. The Rational Method was applied to estimate the peak discharge, yielding approximately 296.61 m^3/s. It solves the uplifting pressure head distribution on the structure using regression from Khosla's pressure curves, allowing for the approximately perfect design of structures with and without consideration of concentration and retrogression. Cell referencing serves as a convenient decision tool for the hydraulic design of small barrages. The discussion highlights the sluice design's effectiveness, potential challenges, and recommendations for future monitoring and technological advancements. The study concludes that the sluice design significantly advances regional water management, emphasizing the need for continued monitoring and adaptive strategies to maintain long-term efficacy.

(Keywords: Pokoria river, Flood mitigation, Khosla's theory, Sluice design)

CHAPTER 1

INTRODUCTION

1.1 Prologue

Flood disasters in Assam, a northeastern state of India, are a recurring and significant problem, primarily due to the region's unique geographic and climatic conditions. The annual monsoon season brings heavy rainfall, leading to rivers like the Brahmaputra and Barak overflowing and causing widespread flooding. This situation is exacerbated by several factors, including deforestation, poor land management, and inadequate flood control measures.(Pal et al., 2013)

Unusual and heavy rainfall results in natural occurrences of floods. In Assam, a flood occurs every year over the wide valley of the Brahmaputra River. In the Brahmaputra valley during August 2000 was one of the worst hit floods of Assam. Flood causes rapid changes in the valley every year including geomorphological changes, land erosion, disruption of roads and rail, human habitats and finally affecting cultivable lands adversely. Brahmaputra River includes part of China, India, Bhutan and Bangladesh, so the perspective of the flood is very diverse and also brings transboundary issues to attention. Extreme monsoon regime, unique physiographic setting and orographic nature of upper catchments and snow-fed Eastern Himalayas. Being harnessed with one of highest basin water yields among the large rivers of the world with a total basin area of 5.8 Lakhs Sq. Km runs a distance of 2880 Km. The gradient of the river bed ranges from 4.3 m/km (meters per Kilometre) in the orographic hilly terrain of the Himalayas to 0.27 m/km near Pasighat where it touches Assam. In Assam, the river flows 670 Km long with a gradient ranging from 0.27 m/km to 0.16 m/km with a river width of 1 Km to 18.6 Km having vast floodplains and the valley is about 35 to 90 Km wide(Sharma et al., 2020) .

1.2 Floods

1.2.1 Definition Floods

Floods indeed represent one of the most prevalent and destructive natural disasters worldwide. A "flood" is defined as "an overflow of water that submerges a normally dry area (*Wikipedia Contributors. (2024, April 30). Flood. In Wikipedia, The Free*

Encyclopedia. Retrieved 06:04, June 20, 2024, from Https://En.Wikipedia.Org/w/Index.Php?Title=Flood&oldid=1221508308, 2024.) Coastal areas are particularly vulnerable due to their exposure to tropical cyclones, tsunamis, and storm surges, which can lead to widespread flooding. However, floods can also occur inland, especially in areas near rivers and streams, where riverine flooding is a common occurrence. The impact of floods can be devastating, resulting in loss of life, damage to property, disruption of essential services, and harm to public health infrastructure. Floodwaters can destroy homes, roads, bridges, crops, and other structures, leading to significant economic and social consequences for affected communities.

One of the challenges with floods is their unpredictability. While some floods may develop gradually over hours or days, others can occur suddenly with little warning, leaving people with limited time to prepare or evacuate. Efforts to mitigate the impact of floods include the construction of flood control infrastructure such as levees, dams, and flood barriers, as well as the implementation of early warning systems and emergency response plans. Additionally, land-use planning and zoning regulations help to minimize the risks associated with building in flood-prone areas. Education and awareness about flood risks and preparedness measures are also crucial for reducing the vulnerability of communities to flooding, particularly for those living in floodplains or in poorly constructed or non-resilient structures. Overall, managing the risks associated with floods requires a comprehensive approach that integrates engineering solutions, disaster preparedness measures, and community engagement to enhance resilience and minimize the impact of these natural disasters.

Hydrology is a field of study that includes flooding. It is the most frequent and extensive type of severe weather that occurs naturally. There is typically a chance of fatalities, indignation from the public, and extensive property damage when floods happen. Floods possess the ability to endanger lives as well as cars, residences, bridges, and other structures. In addition to destroying trees and other land-based constructions, flooding also kills crops. Those who reside in floodplains, in non-resilient buildings, or without access to a flood warning and awareness system are particularly vulnerable to flooding.

1.2.2 Types of Floods

There are several types of flooding, each with its own causes and characteristics. Here are some common types:

1**. Riverine Flooding**: This type of flooding occurs when rivers or streams overflow their banks due to heavy rainfall, snowmelt, or ice jams. Riverine flooding is one of the most common forms of flooding and can affect both urban and rural areas located near water bodies. Riverine flooding refers to the overflow of rivers or streams beyond their normal banks, leading to the inundation of adjacent land areas. This type of flooding is common in regions where rivers and streams are a prominent feature of the landscape.

Riverine flooding can have significant impacts on communities, infrastructure, and ecosystems. It can result in property damage, displacement of residents, disruption of transportation networks, contamination of water supplies, and loss of agricultural land. Effective floodplain management, early warning systems, and infrastructure improvements such as levees and floodwalls are essential for reducing the risks associated with riverine flooding and enhancing community resilience.

2. Flash Flooding: Flash floods are rapid-onset floods that occur within a short period, often a few hours or even minutes, following intense rainfall or other sudden water release. They can be particularly dangerous due to their swift onset and the high velocity of floodwaters. Flash floods are indeed one of the most dangerous and unpredictable natural disasters. They can occur swiftly and with little warning, causing widespread devastation in their path. It's crucial for people living in flood-prone areas to stay informed and prepared to minimize the risks associated with flash floods.

Understanding the factors that contribute to flash floods, such as heavy rainfall, dam or levee failures, and ice jams, is essential for assessing the level of risk in a particular area. Monitoring weather forecasts and staying alert to any warnings issued by authorities can help individuals and communities take necessary precautions in advance. Having a comprehensive disaster preparedness plan in place is vital for mitigating the impact of flash floods. This plan should include evacuation routes, emergency supplies, and communication strategies to ensure that everyone can respond swiftly and effectively in the event of a flood. Ultimately, awareness, preparedness, and proactive measures are key to reducing the loss of life and property damage caused by flash floods.

3. Glacial Lake Outburst flood (GLOF): A Glacial Lake Outburst Flood (GLOF) is a type of outburst flood that occurs when a dam containing a glacial lake fails. This dam can be composed of glacier ice or a terminal moraine. Several factors can lead to the failure of the dam, such as Erosion, Buildup of water pressure, Avalanche of rock or heavy snow, Earthquake or cryoseism, Volcanic eruptions under the ice, Massive displacement of water due to a large portion of an adjacent glacier collapsing into the lake. An event similar to a GLOF, where water contained by a glacier melts or overflows the glacier, is known as a jökulhlaup. A jökulhlaup, an Icelandic term now adopted into the English language, originally described glacial outburst floods from Vatnajökull triggered by volcanic eruptions. Today, it broadly refers to any abrupt and large release of sub-glacial water.

The water body can either be a marginal lake, dammed by the front of a glacier, or a sub-glacial lake, capped by the glacier. When a marginal lake bursts, the event is referred to as a marginal lake drainage. When a sub-glacial lake bursts, it is known as a jökulhlaup. (Wikipedia contributors. (2024, June 16). Glacial lake outburst flood. In *Wikipedia, The Free Encyclopedia*. Retrieved 16:25, July 12, 2024, from https://en.wikipedia.org/w/index.php?title=Glacial_lake_outburst_flood&oldid=1229412432)

4. Coastal Flooding: Coastal flooding is caused by storm surges, high tides, or tsunamis that inundate coastal areas. It is common in regions prone to tropical cyclones, hurricanes, or typhoons, where strong winds push seawater inland. Coastal flooding poses significant risks to coastal communities and ecosystems, often resulting from a combination of natural factors such as tidal surges, high winds, and atmospheric pressure changes. These conditions can be exacerbated by storms at sea, including tropical cyclones, tsunamis, and unusually high tides, leading to more severe and widespread flooding events.

Tidal surges, driven by the gravitational pull of the moon and the sun, can cause water levels to rise rapidly along coastlines, especially during high tide cycles. When combined with strong winds and low atmospheric pressure associated with storms, such as hurricanes are particularly vulnerable to flooding. Tsunamis, triggered by seismic activity such as underwater earthquakes or volcanic eruptions, can generate powerful ocean waves that inundate coastal regions with water. While tsunamis are less frequent than other types of coastal flooding events, they can cause immense destruction when they occur. Higher-than-average tides, often referred to as king tides or spring tides, occur periodically due to the alignment of the sun, moon, and Earth. When these tides coincide with storms or other weather phenomena, they can lead to elevated water levels and increased risk of coastal flooding. Coastal flooding can have devastating consequences for coastal communities, causing property damage, erosion, displacement of populations, and disruption of critical infrastructure. Effective coastal management strategies, including coastal defence systems, land use planning, and early warning systems, are essential for reducing the impacts of coastal flooding and building resilience in vulnerable areas.

5. Urban flooding: Urban flooding occurs in cities and towns due to inadequate drainage systems, impermeable surfaces, and rapid urbanization. Heavy rainfall overwhelms stormwater drainage systems, leading to localized flooding in streets, neighbourhoods, and low-lying areas.

Flash floods are indeed highly dangerous and unpredictable events that can cause significant destruction and loss of life. They typically occur within a short timeframe following intense rainfall, often in just a few hours. The rapid accumulation of water overwhelms natural drainage systems, leading to torrents that can sweep away anything in their path.

Several factors can contribute to the occurrence of flash floods. The primary cause is heavy rainfall, especially from intense thunderstorms or other weather systems that produce high rainfall rates over a short period. Additionally, failures of dams or levees, as well as ice jams releasing large volumes of water, can trigger flash floods, amplifying the destructive potential.

One of the most alarming aspects of flash floods is their ability to strike with little or no warning, particularly in cases where infrastructure such as dams or levees fails unexpectedly. This lack of advance notice underscores the importance of preparedness and awareness. The National Weather Service advises individuals and communities to familiarize themselves with the flood risk in their area and develop comprehensive catastrophe plans to respond effectively in the event of a flash flood.

Given the swift and destructive nature of flash floods, proactive measures such as early warning systems, evacuation planning, and infrastructure improvements are crucial for mitigating their impact and safeguarding lives and property.

6. Pluvial Flooding: Pluvial flooding, also known as surface water flooding, happens when rainfall exceeds the capacity of the ground to absorb water, leading to ponding or runoff on the surface. It often occurs in urban areas with poor drainage infrastructure or during intense rainfall events.

7. Fluvial Flooding: Fluvial flooding is caused by prolonged rainfall over large river basins, resulting in the gradual rise of river levels and subsequent flooding of adjacent areas. It is common in regions with monsoon climates or during periods of sustained precipitation.

8. Dam or Levee Failure Flooding: Flooding can occur when dams or levees fail, releasing large volumes of water downstream. This type of flooding can result from structural failures, overtopping, or breaches in the dam or levee systems.

9. Ice Jam Flooding: In cold climates, ice jams can form on rivers when floating ice accumulates and obstructs the natural flow of water. This can lead to localized flooding as water backs up behind the ice jam and spills over onto adjacent land.

Understanding the different types of flooding is essential for developing effective mitigation strategies, emergency preparedness plans, and infrastructure improvements to reduce the impact of floods on communities and infrastructure.

1.2.3 Causes of Floods in Assam

The North Eastern part of India mainly comprises of Assam, and its neighbouring seven states are getting a sufficient amount of rainfall in the monsoon and hence the rivers become sufficient in water and cause flood in summer. Assam is one of the worst sufferers of flood due to River Brahmaputra and its tributaries and it comes in every year as a festival which causes woes rather than joy(Bora & Chandra Bora, 2010). Some reasons of flood occurrence in the state can be summarised below:

1. Heavy Monsoon Rains: Assam receives heavy rainfall during the monsoon season, which typically lasts from June to September. The intensity and duration of these rains often lead to rivers swelling and breaching their banks.

2. River Characteristics: The Brahmaputra and Barak rivers, along with their numerous tributaries, have high sediment loads. This sedimentation reduces the water-holding capacity of the rivers, leading to frequent overflow.

3. Topography: Assam's topography, with its low-lying floodplains, makes it prone to flooding. The floodplains act as natural reservoirs, but they can be overwhelmed by excessive water.

4. Deforestation and Land Use Changes: Deforestation in the upstream areas leads to increased surface runoff and reduced soil absorption capacity, contributing to higher flood peaks.

5. Poor Drainage Systems: Inadequate urban planning and poor drainage infrastructure in both rural and urban areas hinder effective water management during heavy rains.

1.2.4 Mitigation and Management Strategies

Flood mitigation in Assam is a critical concern given the state's recurrent and severe flooding problems. Effective flood mitigation requires a multifaceted approach that includes structural measures, non-structural measures, community engagement, and policy initiatives. Here are some key strategies for flood mitigation in Assam:

1.2.4.1: Structural Measures

1. Embankments and Levees: Strengthening Existing Embankments by regular maintenance and reinforcement of existing embankments to prevent breaches. In vulnerable areas to contain river overflow construction of new embankments can be made.

2. Dams and Reservoirs: Constructing Multipurpose dams that can serve both flood control and irrigation purposes. Optimizing reservoir operation to ensure water is released in a controlled manner to prevent downstream flooding.

3. River Channelization and Dredging: Regular dredging of rivers like the Brahmaputra to increase their water-holding capacity and reduce sedimentation. Deepening and widening river channels to improve flow and prevent overflow.

4. Flood Control Infrastructure: Constructing flood walls and hydraulic structures to protect critical infrastructure and populated areas. Developing efficient urban drainage systems to manage surface runoff during heavy rains. Control structures, such as sluice gates, play a crucial role in managing water flow and levels in various hydraulic environments. These structures create a relationship between the discharge (the volume of water flowing per unit time) and the depth of the surrounding water flow.

1.2.4.2: Non-Structural Measures

1. Early Warning Systems: Implementing advanced meteorological and hydrological forecasting systems to predict floods. Establishing community-based alert systems to provide timely warnings to residents.

2. Land Use Planning: Implementing land use zoning to restrict construction in flood-prone areas. Protecting and restoring wetlands to act as natural buffers and water storage areas.

3. Afforestation and Soil Conservation: Promoting afforestation in catchment areas to reduce runoff and improve soil absorption. Implementing soil conservation techniques such as terracing and contour ploughing to reduce soil erosion.

1.2.4.3: Community Engagement and Preparedness

1.Community Awareness Programs: Conducting awareness campaigns to educate the public on flood risks and safety measures. Organizing training sessions and mock drills to prepare communities for emergency response.

2.Community-Based Flood Management: Establishing local flood management committees to involve residents in planning and response activities. Engaging communities in the development and implementation of flood mitigation plans.

1.2.4.4: Policy and Governance

1. Integrated Flood Management Plans: Making Comprehensive Planning for Developing integrated flood management plans that consider both structural and non-structural measures. Ensuring coordination among various government departments, NGOs, and other stakeholders.

2. Legislation and Regulation: Enforcing floodplain regulations to control development in flood-prone areas. Implementing building codes that require flood-resistant construction practices.

3.Funding and Resources: Allocating sufficient government funds for flood mitigation projects and infrastructure development. Seeking support from international organizations for technical expertise and financial assistance.

1.2.4.5: Technological and Scientific Measures

1. GIS and Remote Sensing: Using GIS and remote sensing technologies to map flood-prone areas and monitor real-time flood conditions. Analysing historical flood data to predict future flood events and plan mitigation strategies.

2. Hydrological Modelling: Developing hydrological models to simulate river behavior and assess the impact of different flood control measures. Using models to evaluate the effectiveness of various flood mitigation scenarios.

1.3 Hydraulic Structure:

Hydraulic structures play a crucial role in flood management by controlling, diverting, and managing water flow to prevent or mitigate flood damage. These structures include dams, levees, floodgates, and other infrastructure designed to manage water resources effectively. By integrating these hydraulic structures into comprehensive flood management plans, communities can effectively reduce the risk and impact of floods, protect lives and property, and ensure the sustainable use of water resources. Types of hydraulic structures for flood management are Dams, Levees and Floodwalls, Floodgates and Sluice Gates, Retention and Detention Basins, Temporary Storage, Diversion Channels, Spillways etc.

Sluices indeed have a rich history and diverse applications across various environments, from ancient hydraulic systems to modern water management facilities. The water gate, or sluice gate, serves as a pivotal component in controlling water levels and flow rates in rivers, canals, and wastewater treatment plants.

Control structures, such as sluice gates, play a crucial role in managing water flow and levels in various hydraulic environments. These structures create a relationship between the discharge (the volume of water flowing per unit time) and the depth of the surrounding water flow. Sluice gates, in particular, are versatile and used in multiple settings, from rivers and canals to wastewater treatment facilities. (Yoosefdoost & Lubitz, 2022)

1.3.1 Sluice Gate

A sluice gate is a hydraulic structure designed to control water levels and flow rates. It typically consists of a gate or valve that can move vertically to open or close an aperture through which water flows. The movement of the sluice gate regulates the amount of water passing through, allowing for precise control over water distribution. Sluices are used in a number of settings and come in various forms and sizes. The "water gate" refers to the valve that regulates the water gate. These one-way valves are commonly used to manage water levels and flow rates in rivers and canals. It is also used in waste water treatment plants.

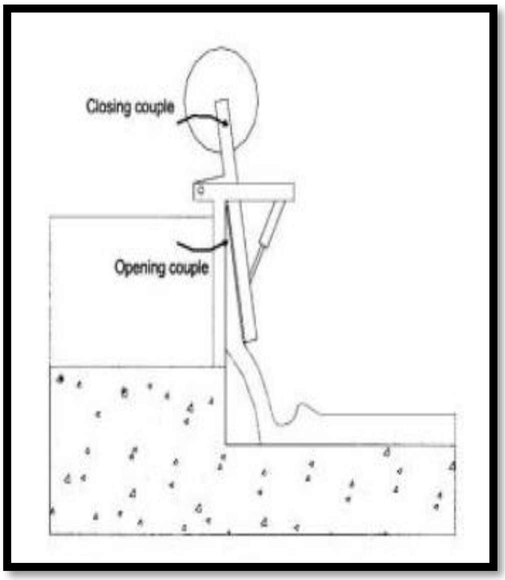
A sluice is a hydraulic structure used by ancient civilizations to regulate gate openings in rivers, streams, and open irrigation networks to manage upstream water levels. The sluice gates move up and down on a vertical plane above the spillway to moderate flow. Water flows through the gate as it flows up the spillway. As a result, they're often referred to as vertical gates or downflow gates. It is necessary to determine the width of the sluice gate. To reduce water waste, management and monitoring should be implemented in the irrigation network's water control and distribution, and flow control structures such as gates should be carefully selected and fitted to the demands of each region. A sluice is a wooden or metal barrier that slides in a groove at the river's edge and is analogous to a lower aperture in a wall.

While sluice gates offer many benefits in flood management, their effectiveness depends on proper design, operation, and maintenance. When integrated into a comprehensive flood management strategy, sluice gates provide a reliable and efficient means of controlling water levels and mitigating the impact of floods.

1.3.2: Types of Sluice Gates

1.Flap Gate: The flap gate is a simple hydraulic automatic upstream water level control gate. Its simplicity stems from its ease of production and maintenance; unlike other hydraulic automated doors, it requires just flat plate and tube fabrication rather than curved surfaces. This door is operated similarly to a check valve by a pressure head located above it. At the top, there is a hinged gate. When pressure is applied from one side, the gate is always closed; when pressure is applied from the other side, the floodgate opens when the threshold pressure is surpassed.

Figure 1.1. Flap Gate



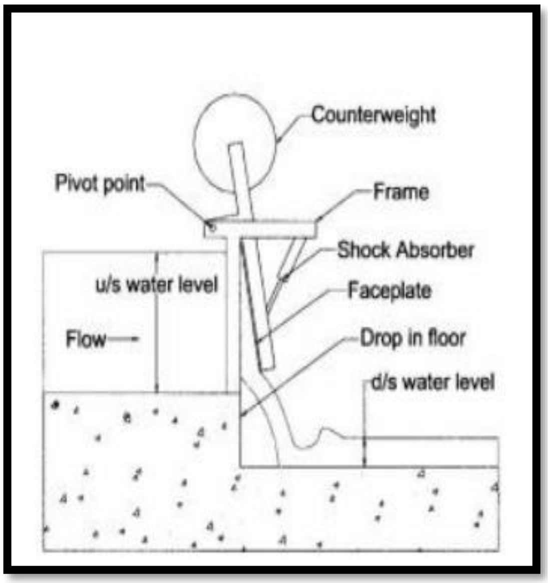


Figure 1.2: Balance of Couples on Flap Gate **Figure 1.3: Side View of Flap Gate**

2. Vertical Rising Sluice Gate: This is a vertical rising sluice gate, which means that the gate's height is adjusted by equipment. The sluice gate may regulate the flow of water by altering its height. Sluice gates are often used to manage the level and flow velocity of water in a river. When the gate is fully lowered, water may pour over the top, producing the same effect as a weir. Sluice gates are used to manage the flow of water in a river, which is especially important during times of floods.

Figure 1.4: Vertical Rising Sluice gate

3.Radial Sluice Gate: The most frequent forms of spill gates in use today are radial gates, sometimes known as tainter gates (after the structural engineer who popularised them). Because water runs beneath the gate, it is categorised as an undershot gate. It is made out of curved skin plates that are held together by a structural steel frame. To raise or lower the gate, a lift system is employed. Water pressure is carried from the curved face to the radial door arm, which distributes the load to a common bearing on both sides of the door opening in a radial door. Load combinations for water control gates can be complicated, and great effort should be made to identify all critical combinations.

Figure 1.5: Vertical Rising Sluice gate

4. Rising Sector Sluice Gate: These gates are mainly used to regulate the river's flow and level. A portion of the cylindrical surface may also be found on this door. It's at the very bottom of the canal. To enrich, the gate will raise by revolving around its centre, allowing water to pass through. These doors operate quickly because they move in an arc. Because the full water load is carried radially via the arm to the pivot, the lifting capacity is diminished. However, a recess space in the floor is necessary for the entry, which can be an issue in areas with a high.

Figure 1.6: Rising Sector Sluice Gate

5. Needle Sluice: As in a needle dam, a sluice constructed by a number of tiny needles pressed against a solid frame by water pressure. A needle dam is a weir that uses thin "needles" of wood to maintain a river's level or flow. The needles are supported by a strong frame and are not meant to be watertight. Individual needles can be added or withdrawn by hand to constrain the river's flow and construct a sluice.

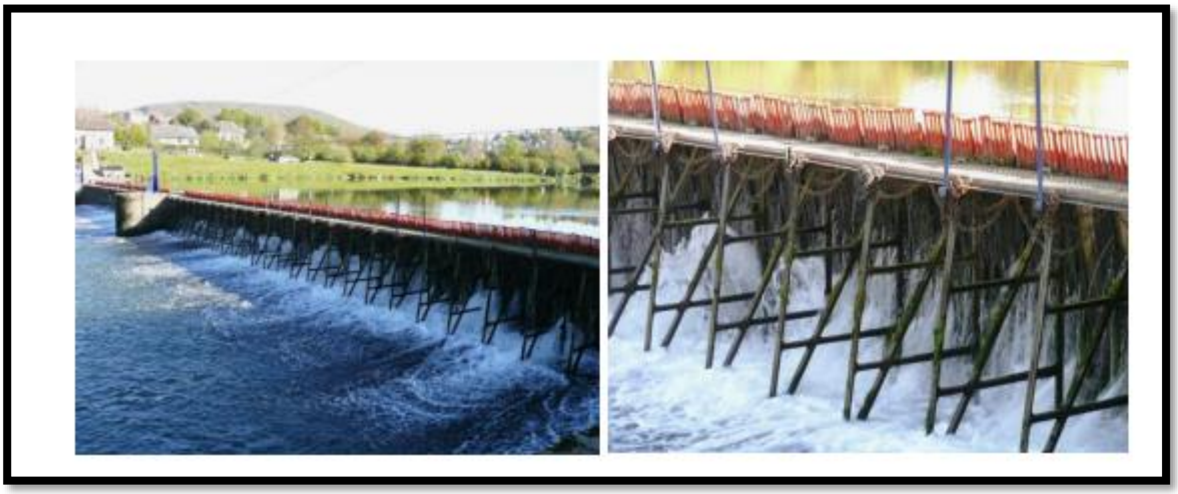


Figure 1.7: Needle Sluice Gate

6. Fan gate: The Fan door has the unique ability to open in the direction of high water merely by using water pressure. This gate type was mostly used to flood certain areas. This sort of gate may still be found in a few places today, such as Gouda. A fan gate has a separate chamber that can be filled with water and is separated from the sluice on the high-water level side by a huge door. When a tube connecting the separate chamber to the sluice's high-water-level side is opened, the water level and therefore the water pressure in this chamber rise to the same level as on the high-water-level side. The bigger gate produces no force since there is no height difference across it. The smaller gate, on the other hand, has a greater level on the upstream side, which applies a force to close the gate. The water level in the chamber will fall when the tube to the low water side is opened. Because the surface areas of the doors differ, there will be a net force shutting the gate.

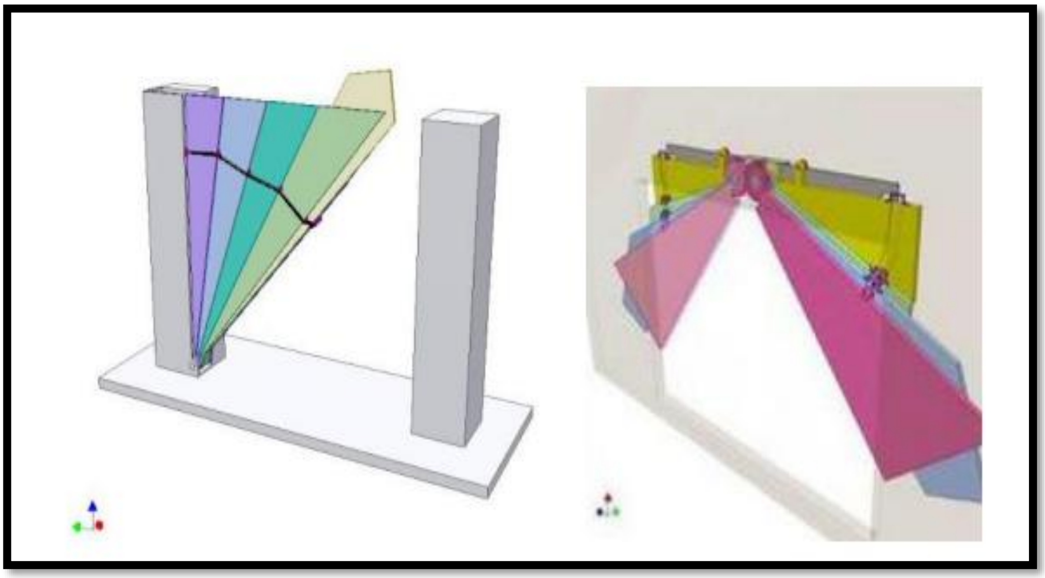


Figure 1.8: Fan Sluice Gate

Traditionally, sluice gates were made from wood or metal, sliding in grooves at the river's edge. Modern sluice gates may use more advanced materials and designs to enhance durability and efficiency. These gates typically feature a one-way seal, allowing water to pass through in one direction while preventing backflow. They are crucial for regulating water levels upstream, ensuring efficient distribution in irrigation networks, and managing flow rates to prevent wastage. The design of sluice gates can vary depending on the specific needs of the environment they serve. They may be constructed from materials like wood or metal, and their sizes and mechanisms can be tailored to suit different requirements. In traditional usage, sluice gates move vertically along a plane above the spillway, controlling the flow of water as it passes through. This design allows for precise management of water levels and helps prevent flooding or drought in surrounding areas. Effective management and monitoring are essential for maintaining optimal performance in irrigation networks and other water control systems. Proper selection and installation of sluice gates ensure that they meet the demands of each specific area, contributing to efficient water distribution and conservation efforts.

1.4 Study Area

The study area is situated at a distance of about approximately 32km from Morigaon on the South bank of the river Brahmaputra. The climate in the area is characterized by tropical and humid type. The area is a thickly populated homestead including enormous crop producing cultivable land comprising of number of business establishments including public utility. The Pokoria River meanders across the landscape for about 140.82 Km long. It originates in Nagaon district near Chaparmukh, Raha at location 26° 13’ 26” N and 92° 31’ 11” E after the merge of Kollong River with rivulet from Kopili river .The river flows from Nagaon district to Morigaon and joins Brahmaputra at 26° 16’ 37” N and 92° 02’ 51” E at Kasohilla Mayong. The Pokoria river receives runoff from Lali channel, Sunduba channel etc.

1.5 Problem Statement

During Monsoon the Brahmaputra River on the south bank at Murkata, Mayong area backflows i.e. flow from Brahmaputra to the countryside, through the Pokoria river and inundates a huge area of homestead and cultivable land. The concerned reach has been under threat/ affected by the flood for the past several years. Due to the erosion, a considerable area has already been lost in the river bed with a negative impact on the environment. The area remains inundated for the entire monsoon season. The backflow of river Brahmaputra through the Pokoria channel inundates approximately 7000-8000 hectares of land comprising large numbers of fisheries and agricultural land.

During the non-monsoon period, the Pokoria River flows into the Brahmaputra i.e. from the countryside to the river Brahmaputra, leading to water shortages for the local inhabitants. Installing sluice gates or barrages at strategic points can help regulate the flow of water between the Pokoria and Brahmaputra rivers. These structures can be used to control the flow of water during both monsoon and non-monsoon periods.

1.6 Objectives of the Study

The objective of this work was to determine the peak discharge of Pokoria river by rational method and design the hydraulic part of the sluice gate using Khosla’s theory at the Pokoria river to manage of flood water of the Pokoria river using cell referencing in MS Excel.

1.7 Methodology

Using bathymetric data, in Manning's equation the peak discharge of Pokoria River is obtained by Area Velocity Method. Using historical daily rainfall data of 21 years (2003-2023) by Gumbel's distribution method peak daily rainfall for return periods 25, 50 and 100 years is calculated . Again the recorded 21 years HFL of Brahmaputra river at Ulubari (Morigaon district) is used to determine the HFL for return peak periods 25, 50 and 100 years is calculated. Sluice at river Pokoria is designed using Khosla's theory because it provides a comprehensive and practical method to address issues related to seepage, uplift pressure, and structural stability of hydraulic structures. Cell referencing in MS Excel is used as a design platform as an innovative approach, leveraging its grid system, data manipulation capabilities, and visualisation tools.

1.8 Organisation of the report

The report is organised into following chapters.

Chapter 1: This chapter deals with a brief introduction on the topic of the study and discusses about types of floods and sluices used to achieve the objective of the study. Further this chapter includes the area of study and a brief introduction problem statement of the area and the objective of the study.

Chapter 2: This chapter depicts the literature survey of the previous literatures.

Chapter 3: This chapter portrays detailed theory used for design of sluice.

Chapter 4: This chapter gives an overview of the study area.

Chapter 5: This chapter contains details of methodology used during study.

Chapter 6: This chapter highlights the database used and generation of parameters necessary for the design.

Chapter 7: This chapter encloses the design of sluice .

Chapter 8: This chapter briefs the result of the study .

Chapter 9: This chapter contains the conclusion and scope of future study.

CHAPTER 2

LITERATURE REVIEW

2.1 Flood mitigation

Hydraulic structures are vital components of flood mitigation strategies. They provide a range of solutions tailored to different flood scenarios and geographical conditions. The design and implementation of these structures require careful planning, consideration of environmental impacts, and ongoing maintenance to ensure their effectiveness in protecting communities and infrastructure from the devastating effects of floods.

2.2 Intensity-duration-frequency

The rainfall intensity-duration-frequency relationship is one of the most widely used methods of urban drainage design and floodplain management. The design of any infrastructure requires an understanding of the desired function of the structure and the physical environment in which it must perform this function. Thus, in case of storm water management, the dimensions of various components of the infrastructure system are based on the return period of heavy rainfall events.

2.3 Previous Literatures

In this project, literature and information from previous research and books has been used as a reference. An overview has been prepared of the previous researches as given below:

The design of diversion weir on permeable foundations has been thoroughly dealt with by different researchers. Its design the component directly dependent on the possibilities of percolation in the porous soil on which the apron built ***(B.C.Punmia, 2007).***

Until recently, ***Bligh's (1912)*** creep theory was being adopted for designing weirs with component parts on sand or alluvial soil. The theory assumed the total head loss up to any point along the base to be proportional to the distance of the point from the upstream of the foundation ***(Garg, 2005)***.The method does not discriminate between the horizontal and vertical creeps in estimating the exit hydraulic gradient. This theory has been found to be defective from actual field observations due to the inherent assumptions of creep length.

Lane (1935) based on his experiment on large number of dams, proposed a method in which the creep is weighted to allow for the variation in creep along vertical and horizontal directions. It is an improvement over the Bligh‟s creep theory but the method for determination of uplift pressure is criticized because it is an empirical method and not based on any mathematical approach. Therefore, he suggested a factor of one third for horizontal creep against 1 for the vertical creep (Robel, 2009).

Khosla(1954) evolved the "method of independent variables" in which a composite barrage or weir section is split up into a number of simple standard forms of known analytical solution, these are a straight horizontal floor of negligible thickness with a sheet pile at either end, a straight horizontal floor depressed below the bed but with no vertical cut off and a straight horizontal floor of negligible thickness with sheet pile at some intermediate position.

Das et al. (2016) generated IDF curve for Guwahati city using Gumbel's Extreme Value distribution. Short duration rainfall data was collected for the study. IDF empirical formula $i = a * (t_d)^{-c}$ was used in the study for estimating the maximum rainfall intensities for different duration and return periods. The IDF parameters a and c can be used to derive rainfall intensity for a given duration of a rainfall event. The computed data shows that the intensity of rainfall decreases as duration increases and for a particular duration as return period increases intensity tends to increase.

Zope et al. (2016) initially proposed the IDF curves using an empirical equation (Kothyari and Garde) by using probability distribution for annual maximum rainfall and then IDF curves are derived by modifying the equation. IDF curves developed by the modified equation gives good results in the changing hydrologic conditions and are compatible even with the extreme rainfall of 26th July 2005 in Mumbai. Alternatively, it was proposed that the IDF relationship developed for Santacruz rain gauge station may be used for the entire city, since it is located centrally and also being expressing the higher intensities of rainfall, the design would be safer to avoid flooding in the future.

(Al-Amri & Subyani, 2017) used to estimate rainfall intensity for design purposes for the ungauged location using empirical intensity frequency equation. The results of this research contribute to the development of IDF-based design criteria for water projects in ungauged sites located in arid and extreme arid regions. The analyses focused on the application of two distributions: the Gumbel and Log Pearson III functions combined, to estimate the maximum rainfall for the various return periods in three stations in Al-Madinah region.

(Ahmad, 2002.) in his work shows that the design of the Chiniot dam on impermeable foundations is economical and safe under conditions of infiltration and increased water pressure. Barrage design uses a traditional approach. The results obtained provide the best barrage parameters, including upstream and downstream sheet piles/cross-section depth, floor length and thickness. Soil type and hydrological conditions are combined in the design of the dams through the safety outlet slope and the seepage head respectively. The findings indicate that the lifting force will rise if the d/s pile is deeper than the upper cutting wall. The resulting hydraulic gradient decreases as the depth of the d/s pile increases and as the floor's total length increases. The exit is lower when the two piles meet at the hydraulic structure's end than it is when the pile in the d/s direction is greater.

(Chaurasia, 2003.)states that the design and analysis of stilling basins, which are essential for dissipating energy in hydraulic systems, often involve complex calculations related to hydraulic jumps. A hydraulic jump is a phenomenon where a high-velocity, low-depth flow (supercritical) transitions to a low-velocity, high-depth flow (subcritical), resulting in a sudden rise in water level. This process is accompanied by significant energy loss. Understanding the characteristics of hydraulic jumps, such as the prejump and post-jump depths, is crucial for the effective design of stilling basins. The use of direct explicit empirical equations for hydraulic jump elements provides a more straightforward and accurate method for engineers to design and analyse stilling basins and other hydraulic structures. These equations eliminate the need for iterative solutions, making the design process more efficient and reliable, especially for a wide range of discharge intensities and head losses.

As per ***Bibhabasu M, (2012)*** headwork serves to raise the water level at the head of the canal, to control the intake of water into the canal, to control the entry of silt in to the canal and to control deposit of silt at the head of the canal, to store water for small period of time so that water is available throughout the year and to control the fluctuation of water level in the river during different season.

(Ljubenkov, 2015) in his study exemplifies the challenges and complexities of managing flood risk in karstic landscapes of Imotsko-Bekijsko Polje. While hydraulic structures have enhanced flood security, their efficiency and capacity need continual evaluation amidst evolving hydrological conditions. The research underscores the need for integrated water management approaches that balance engineered solutions with natural hydrological processes, ensuring sustainable flood mitigation in the polje region.

G.Arun et.al (2020) carried out a flood assessment study in the state Assam using real time precipitation data and flood inundation mapping. In this study, he had used Sentinel 1, C Band Synthetic Aperture Radar (SAR) based Interferometric Wide (IW) Swath Satellite Data for Mapping of flood affected zones in Assam. It is seen that during flood events and cloud cover Landsat 8 data often have lower frequency. Therefore, Sentinel-1 data were utilized to seamlessly extract flood inundation areas and correlate them with satellite rainfall products based on the GPM IMERG Algorithm Version 7. The GPM data were processed and extracted online. He concluded that continuous monitoring of the flood area using integrated methods of satellite precipitation would later help to develop AI-based models which would forecast flood inundation areas.

Arash Yoosefdoost and William David Lubitz (2022) has focussed their study on the performance on five different models for distinguishing flow regimes and estimating discharge coefficients (Cd) and flow rates. Models were developed under different scenarios such as upstream and downstream conditions, different flow rates and different gate openings.EML model and Swamees model performed well for distinguishing flow regimes. Whereas EM, EML and Henry’s model performed well for free flow conditions and estimated the flow rates. A new equation and method were framed to calibrate k in EML model. It is seen that HEC RAS model estimation does not performed well.

CHAPTER 3

THEORETICAL BACKGROUND

Hydraulic structures are vital components of flood mitigation strategies. They provide a range of solutions tailored to different flood scenarios and geographical conditions. The design and implementation of these structures require careful planning, consideration of environmental impacts, and ongoing maintenance to ensure their effectiveness in protecting communities and infrastructure from the devastating effects of floods. It plays a crucial role in flood management by controlling, diverting, and managing water flow to prevent or mitigate flood damage. These structures include dams, levees, floodgates, sluices and other infrastructure designed to manage water resources effectively.

3.1 Barrage/Sluice planning

Diversion of water for various purposes like irrigation, drinking, flood control etc has been practised since ancient times. In India before the British rule diversion/irrigation was done generally by Inundation canals in north India and by Tanks in south India. (भारत सरकार जल शिक्त मं त्रालय जल सं साधन, नदी िवकास और गं गा सं रक्षण िवभाग के द्रीय जल आयोग राष्ट्रीय जल अकादमी, प ु णे के द्रीय जल अिभयां ित्रकी से वा के नव िनय ु क्त अिधकािरयय का इककीसवां प्रवे शन प्रिशक्षण कायर् क्रम िडजाइन और अन ु सं धान Module IV: Design of Weirs, Barrages and Canals, 2019)

Diversion structures are run-off river structures and involve only incidental storage, providing better sediment control, very low submergence, and fewer ecological problems. However, these do not offer flood moderation and generally serve less command area than storage projects.

Barrages are structures where the crest is kept at a low level and the rising of the water level (or ponding) is accomplished mainly through the gate. During flood, these gates can be raised clear off the high flood level and thus enable the flood to pass with minimal afflux (or heading up of water on the upstream side).(Sunu et al., 2020)

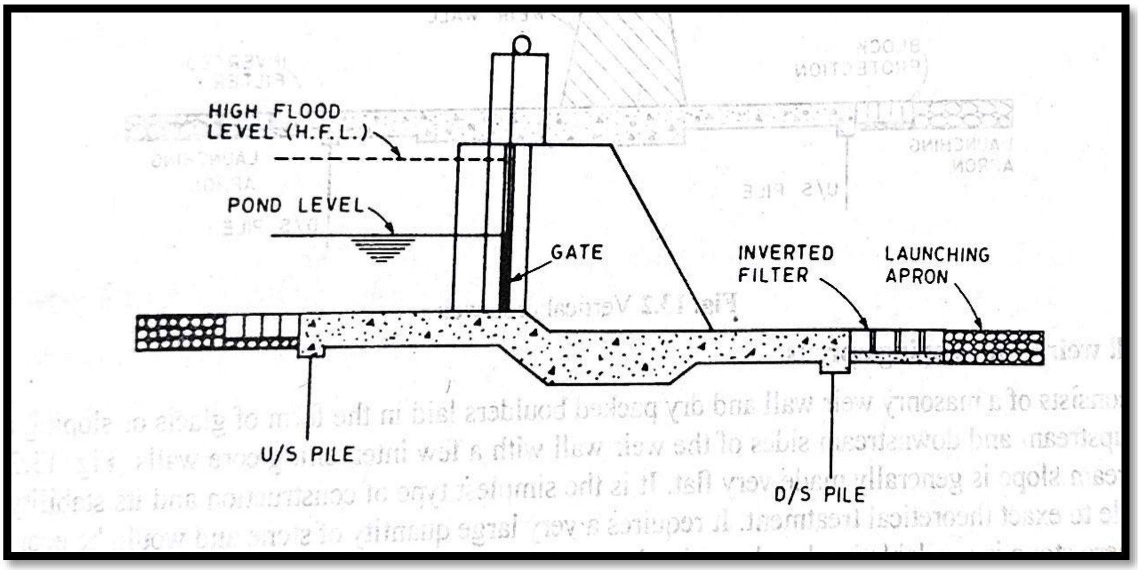


Figure 3.1 Cross section of Barrage

The basic components of a Barrage (Figure 3.2) are Upstream protection arrangement, Upstream Pucca Floor, Crest Stilling Basin, Downstream protection arrangement, Piers, Divide Walls, FishPass, Undersluices, Abutments, Flare out wall, Flank wall, Guide Bunds, Return Walls, Afflux bunds, Cutoffs, Intake Regulator with similar arrangements.

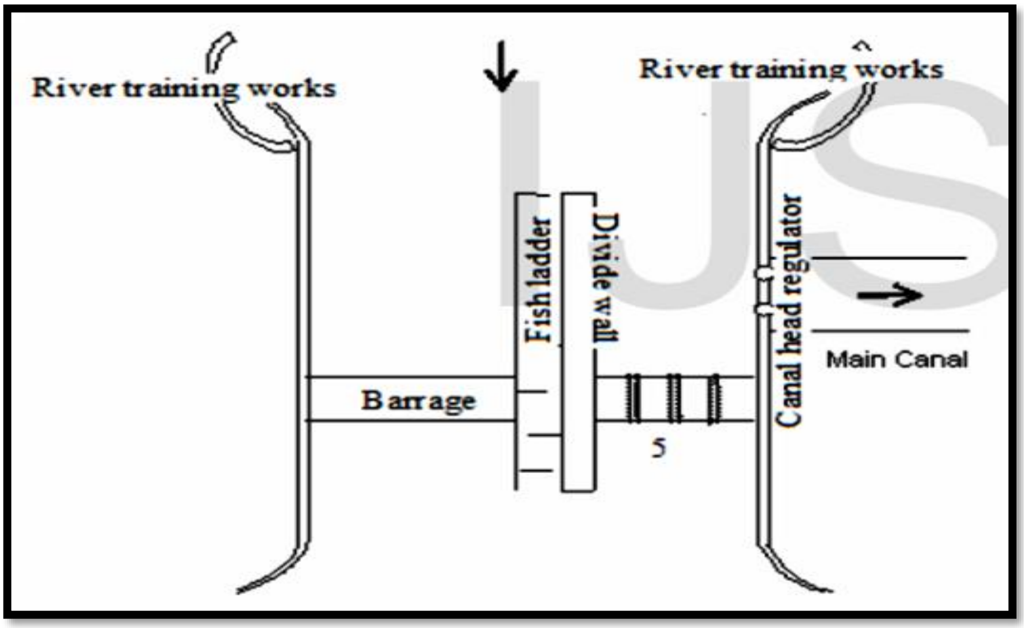


Figure 3.2. Typical Schematic layout of Barrage

The methods of designing weirs have evolved on a large scale, mainly based on the causes behind their failure studies. Like any other hydraulic structure, it consists of many phases which are summarised in Figure 3.3 below

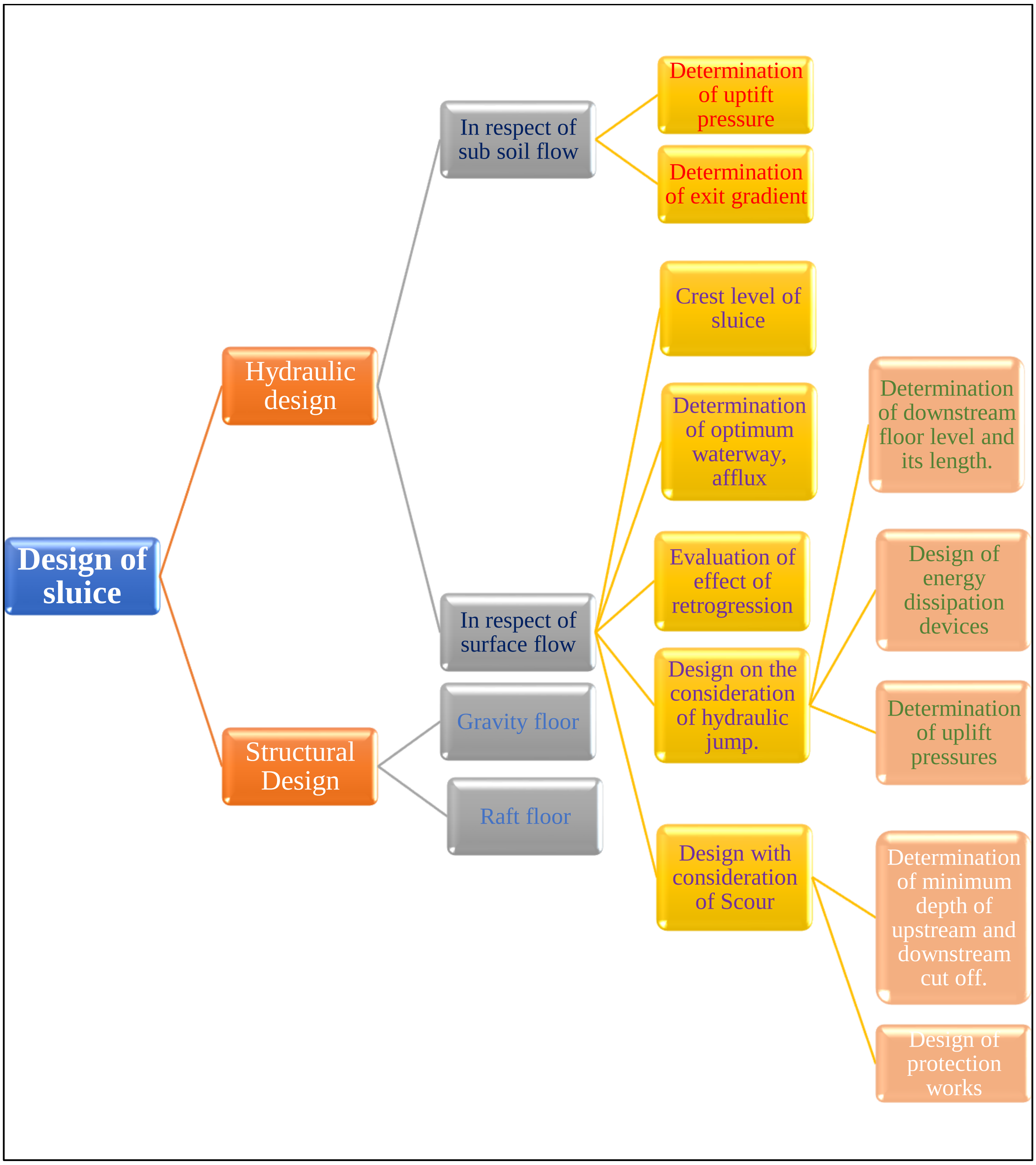


Figure 3.3 : Phases of Sluice design

3.2 Detailed Investigations Required

The main investigation required for the planning and design of barrages and weirs as per various IS codes are given below:-

1. Topographical Investigations
2. Hydrological Investigations
3. Surface and sub-surface Geotechnical Investigation

Topographical survey consisting of contour plan of the area, cross-sections and longitudinal section of the river should be carried out. conforming to IS : 6966-1973. The survey should be plotted to suitable scale. The survey should show all the salient features like firm banks, rock outcrops deep channels, large shoals and islands, deep pools and important land marks, etc. The length of the survey may depend upon the nature of stream, the size of the barrage or weir and the purpose of diversion. If the river course on the upstream and downstream of the site is straight, the length of survey can be shortened whereas in case of meandering rivers the length of survey may be increased so as to cover at least two fully developed meanders on the upstream of the barrage axis and one meander length on the downstream or as may be required for detailed model studies.

The aim of the collection of hydrological data is two-fold, namely:

(a) for computing the design flood.

(b) for assessing the available weekly and monthly run-off on a more realistic basis.

The following data should be collected for estimating the maximum anticipated flood :

a) Rainfall—Daily rainfall recorded at different stations the catchments area and data regarding storms in respect of successive positions of the center of the storm on the catchments should be collected. Storms causing peak discharges should be separated for unit hydrograph analysis;
b) Flood hydrographs for isolated rain storms for working out unit hydrograph;
c) Catchment characteristics, such as shape, slope, orientation, drainage system and infiltration capacity for developing synthetic hydro-graph, inadequate data are not available ;
d) Peak flow data for the river for as many years as possible for frequency analysis;
e) Flood marks by local enquiry to estimate maximum flood by slope area method.

A complete picture of Surface and Sub-surface Investigation is very necessary for the Proper design of the diversion structure and its components. Bore holes should be driven at specified intervals covering the barrage or weir area and appurtenant structures. The locations of borings shall be correctly marked and numbered on the survey sheets. These borings should be carried to hard rock level or to a depth of 15 to 25 m below the deepest river bed level depending on the strata and the structure (pier, abutments, floor etc). Soil classification, unit weight of soil, angle of internal friction of soil, void ratio and specific gravity up to foundation level. In case of clayey and silty foundations undisturbed sampling should be done and tests conducted for determination of unconfined compressive strength and consolidation characteristics.

3.3 A Parametric Study for the Design of Barrages

The design of the diversion structure comprises two parts, namely hydraulic and structural design. In hydraulic design, the overall dimensions and profile of the main structure and a few of the components are calculated to get the satisfactory hydraulic performance of the structure. Structural design is carried out to get the different sectional and reinforcement details. Fixed dimensions and layout obtained from hydraulic design is tested by model studies and the recommendations of model study are incorporated in structural design.

In hydraulic design, the diversion structure has to be properly designed for both the surface and sub-surface flow condition. The design for surface flows will include the fixing of the waterway, top profile of various components, energy dissipation arrangements, protection works, scour values, length and protection of divide walls, alignment, and levels and protection of guide bunds, afflux bund, etc. The design for sub surface flows will include fixing of the depth and section of cut-off, uplift pressure calculation, exit gradient, etc.

3.3.1 Basic Parameters Required for Hydraulic Design

The following basic parameters to be obtained before taking up detailed hydraulic designs:

(i) Design flood discharge
(ii) Afflux
(iii) Pond level
(iv) Rating curve and down stream retrogression of water levels
(v) Silt grade
(vi) Specific gravity, permeability and depth of permeable strata.

3.3.1.1 Design flood discharge

For purposes of design of items other than free board, a design flood of 1 in 100 year return period flood is generally adopted for important / permanent barrages. For designing free board, a minimum of 500 year frequency flood or standard project is generally adopted.

3.3.1.2 Afflux

The width of the barrage/weir is governed by the value of afflux (at the design flood) to be permitted and the proposed crest levels. It is also important for the design of downstream cisterns, flood protection and river training works, loose protections and cut-offs.

The maximum permissible value of afflux has to be carefully evaluated depending upon the river conditions upstream and after considering the back-water effect, the area of submergence and its importance. The afflux is generally limited to 1.0 m for structures on alluvial rivers at higher reaches and 0.3 m in lower reaches. In very steep reaches of the river with boulders or rocky beds, the afflux may safely be higher, say of the order of 2-3 meters or higher.

3.3.1.3 Pond level

Pond level, in the under-sluice pocket, upstream of the canal head regulator shall generally be obtained by adding the working head to the designed full supply level in the canal. The working head shall include the head required for passing the design discharge into the canal and the head losses in the regulator and losses in trash-rock if provided.

3.3.1.4 Rating curve and downstream retrogression of water levels

In the design of the diversion structures, the rating curve, otherwise called as Gauge Discharge Curve, plays a very important part. Hence it is very imperative to have a good rating curve of the river at the site of the structure. If no data is available on the Gauge Discharge values, the same may be worked out by Manning's formula. This is the un-retrogressed G-D curve to be used for waterway and free-board calculations. Silt-free water flowing down the diversion structure causes degradation or retrogression of the downstream bed. The lowering of bed levels affects the exit gradient and energy dissipation. If the retrogression of the downstream bed is not duly allowed for in designs, it may fail the structure. A value of 1.25 to 2.25 m may be adopted at lower river stages and 0.3 to 0.5 m at the design flood. For intermediate discharges, the effect of retrogression may be obtained by plotting the retrogressed high flood levels on log-log graph.

3.3.1.5 Silt grade

Maximum depth of scour hole along the structure at various points depends on the grade of bed material. Silt grade and silt load also influence the design of silt extractor near the canal head regulator. It also gives an indication about the looseness factor to be provided.

Table 3.1 Silt factor and Looseness factor

Silt Factor	**Looseness Factor**
Less than 1	1.2 to 1 1
1 to 1.5	1 to 0.6

3.4 Waterway Calculations

The waterway includes that of under sluice, spillway and river sluice bay, if any. In deep and confined rivers with stable banks the overall waterway between the abutments would normally be adjusted to the actual width of river at design flood level. For shallow and meandering alluvial rivers for minimizing the shoal formation, the following looseness factor shall be applied to Lacey's waterway for determining the primary value of the waterway.

Lacey's waterway is given by the formula,

$$P = 4.83Q^{\frac{1}{2}} \text{ …….. (3.1)}$$

Where, Q = design flood discharge in cumecs for 100-year frequency flood.

For deciding the final waterway, the following additional considerations may also be taken into account:

a) Cost of protection works and cutoffs,

b) Repairable damage for floods of higher magnitude, and

c) Afflux constraints as determined by model studies.

With the selection of span width of each of bay and adjustment of the no. of under sluice bays, spillway bays and also fish passage required if any, the width between the abutment can be calculated by adding the thickness of single and double piers. The actual waterway thus provided is checked for its adequacy by assuming the crest level of under sluice equal to the average bed level and spillway crest is usually taken 1 to 2 m higher than the crest level of under sluice.

3.5 Adequacy of waterway

Adequacy of the waterway is checked for passing Design flood which is found by the following formula:

$$Q = CLK^{\frac{3}{2}} \ldots\ldots \text{(3.2)}$$

Where, Q = Discharge in cumecs,

C = Coefficient of discharge,

L = clear waterway of the barrage or weir in m, and

H = total head causing flow over the crest of weir or barrage in m.

The value of the Coefficient of discharge C is usually taken as 1.7

The discharge intensity q(=Q/L) is thus given by

$$q = CK^{\frac{3}{2}}$$

$$or\ q = 1.7K^{\frac{3}{2}} \ldots\ldots \text{(3.3)}$$

3.6 Hydraulic Jump and Energy Dissipation

Hydraulic Jump and Energy Dissipation Hydraulic jump results when there is a conflict between upstream and downstream controls which influence the same reach of channel. For example, if the upstream control causes a supercritical flow while the downstream control dictates a sub-critical flow, a Hydraulic jump results when there is a conflict between upstream and downstream controls which influence the same reach of the channel. For example, if the upstream control causes a supercritical flow while the downstream control dictates a sub-critical flow, Hydraulic jump results when there is a conflict between upstream and downstream controls which influence the same reach of the channel. For example, if the upstream control causes a supercritical flow while the downstream control dictates a subcritical flow, then there is a conflict which can be resolved only if there is some means for the flow to pass from one flow regime to the other. The phenomenon of jumping of water from super-critical flow to sub-critical flow is known as Hydraulic jump (HJ). Hydraulic jump is generally accompanied by large-scale turbulence, dissipating most of the kinetic energy of supercritical flow which has got detrimental effect on the surface. For a structure to be safe, the formation of jump should be confined to the sloping glacis and not allowed to be formed on the cistern level beyond the toe of glacis. The cistern level and its length are to be worked out for various set of condition imposed on the structure on the basis of the gate regulation proposed. The most critical condition gives the lowest cistern level and its length. These are generally determined by the use of curves available for various discharge intensities and water depths or by analytical method.

3.6.1 Basic relations of Hydraulic Jump

The different parameters of the HJ are shown in figure 3.4.

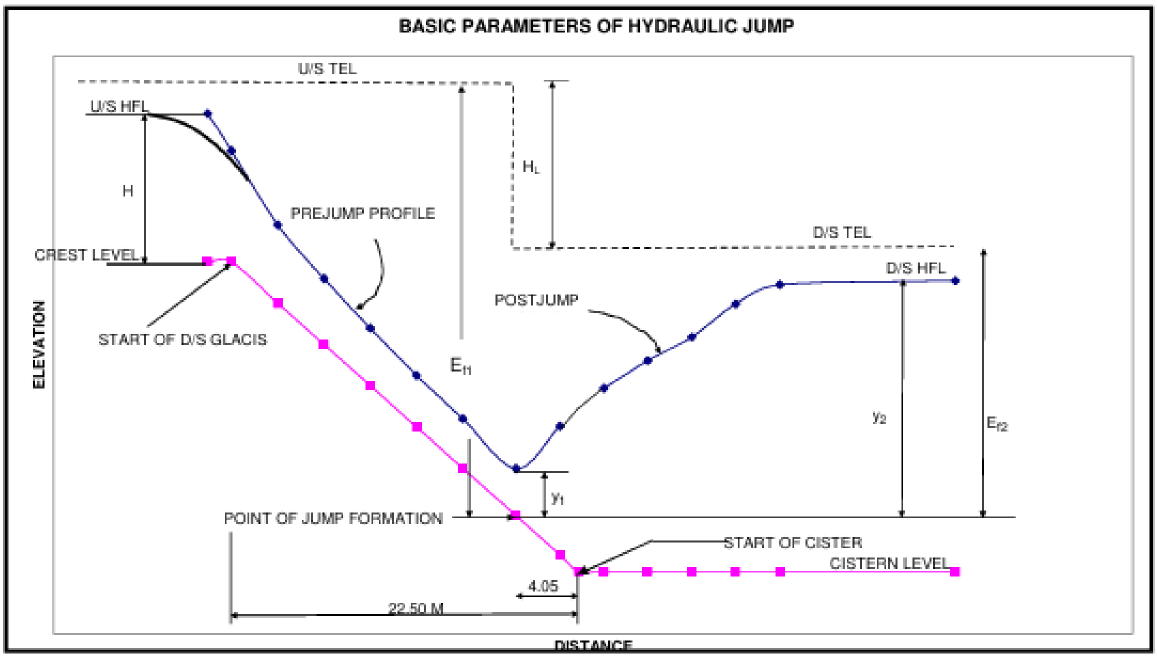


Figure 3.4 : Hydraulic Jump Parameters

The main unknown parameters of the HJ are the conjugate depths linked by the following formulae

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} \quad \text{...........(3.4)}$$

$$H_L = \frac{(y_1 - y_2)^3}{4 y_1 y_2} \quad \text{...............(3.5)}$$

$$E_{f1} = y_1 + \frac{q^2}{2g{y_1}^2} \quad \text{...........(3.6)}$$

$$E_{f2} = y_2 + \frac{q^2}{2gy_2^2} \qquad \text{...........(3.7)}$$

where y_1 and y_2 are pre and post jump depths, q is the intensity of discharge, HL is the head loss in the jump, E_{f1} and E_{f2} are the specific energies before and after the jump. In a hydraulic jump, there are six independent variables viz., y_1, y_2, E_{f1}, E_{f2} , q and H_L, which are interrelated by 4 equations. If any two variables are known, the remaining four can be determined. In actual design of irrigation structures, generally the discharge intensity (q) and the loss of energy H_L are known. The solutions for the cubical equations are given below.

3.6.2 Determination of Cistern Level by Analytical Method.

Various steps involved in the determination of cistern level by analytical method are as follows:

i) Estimate the d/s total energy line (D/S TEL) = d/s retrogressed water level +velocity head

ii) Then u/s total energy line (U/S TEL) = D/S TEL + afflux

The velocity head can be worked out as, $h_{va} = {}^{q}/_{R}$

Where, $R = 1.35\ (q^2/f)^{1/3}$ for looseness factor < 1

$R = 0.475\ (Q/f)^{1/3}$ for looseness factor > 1

iii) Assume an arbitrary cistern level for the particular discharge.

iv) then, E_{f1} = U/S TEL – Assumed cistern level

v) from the known values of E_{f1} and q with concentration, y_1 can be calculated from the relationship,

$$E_{f1} = y_1 + \frac{q^2}{2gy_1^2}$$

vi) Calculate the pre-jump Froude's number as

$$F_1 = \frac{q^2}{2gy_1^3} \qquad \text{...........(3.8)}$$

vii) From the calculated values of y_1 and F y_2 can be calculated from the relationship

$$y_2 = \frac{y_1}{2}\left(-1 + \sqrt{8F_1^2 + 1}\right) \text{ ....(3.9)}$$

viii) The required cistern level = d/s water level (retrogressed) $- y_2$

ix) Compare the assumed cistern level (step iii) and the actual cistern level (step ix). If they are not same, the cistern level assumed initial needs to be raised or lowered accordingly and the steps iii) to ix) are repeated till a nearer value is obtained.

x) Cistern length = 5(y_2-y_1).

The steps are repeated from i) to x) for various discharge conditions and gate regulations. Then the lowest value of step viii) is taken as the cistern level and the highest value of the step x) is taken as the cistern length. The above process of solving the cubical equation for y_1 is cumbersome because it involves trial and error. To simplify the laborious process the help of curves can be taken.

3.6.3 Determination of Cistern Level by the Use of Curves.

For the determination of cistern levels, a set of curves known as Blench Curves and Montague curves are generally used. For different cases and their discharge values the procedure for determining the HJ parameters is as follows.

The procedure is same as above analytical method except the finding of E_{f2} and y_1, y_2. From the known values of the intensity of discharge with concentration and the head loss, using Blench curve (figure 3.5), the values of E_{f2} (energy of flow d/s of the jump formation) can be obtained for different discharges. E_{f1}, i.e., energy of flow just upstream of the point of the jump formation is found by adding H_L. From the known values E_{f1} and the intensity of discharge, using Montague Curves (figure 3.6), the values of y_1 and y_2 i.e., the hypercritical and sub-critical depths of water before and after the hydraulic jump can be read out.

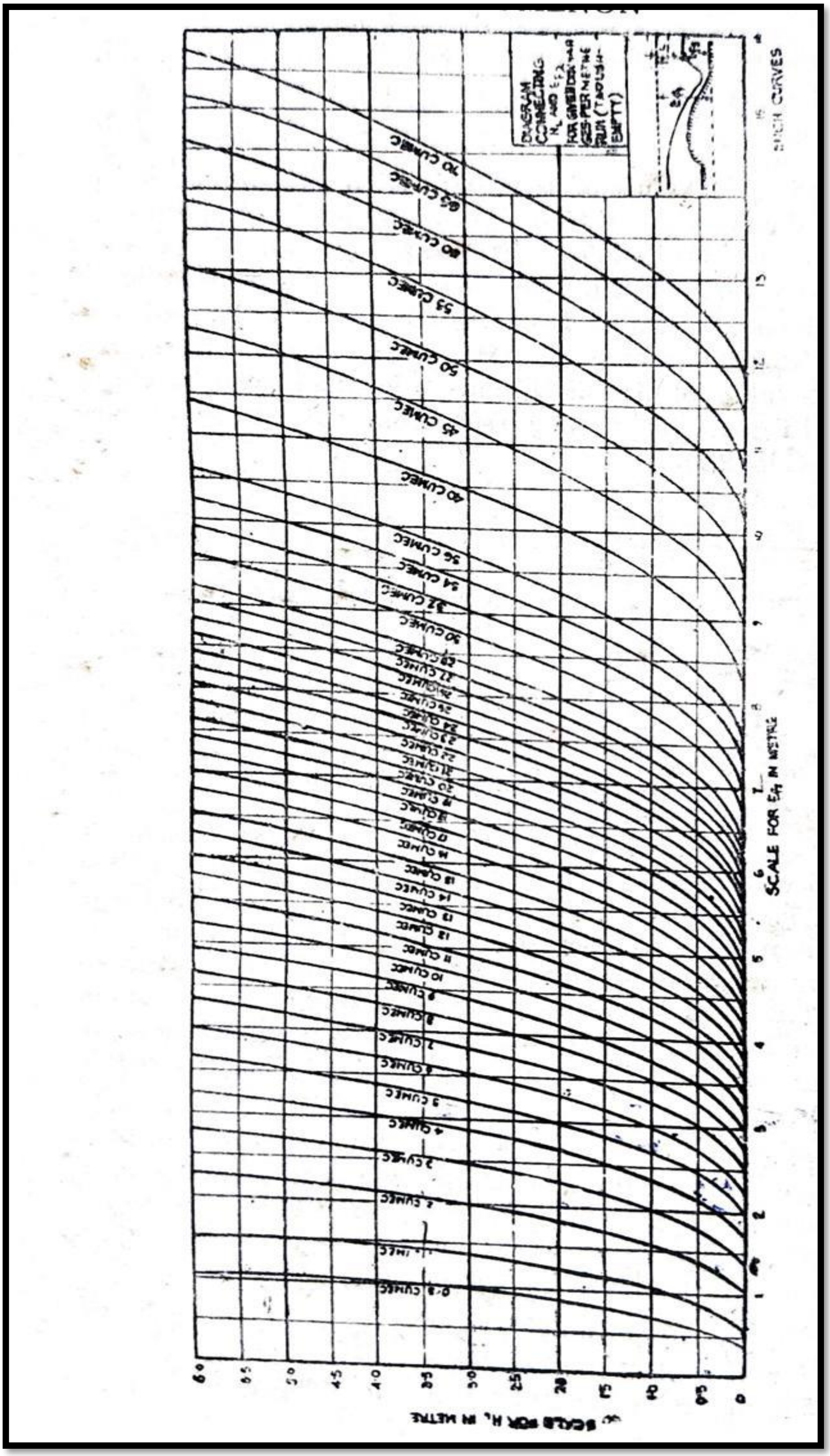


Figure 3.5 Blench curve

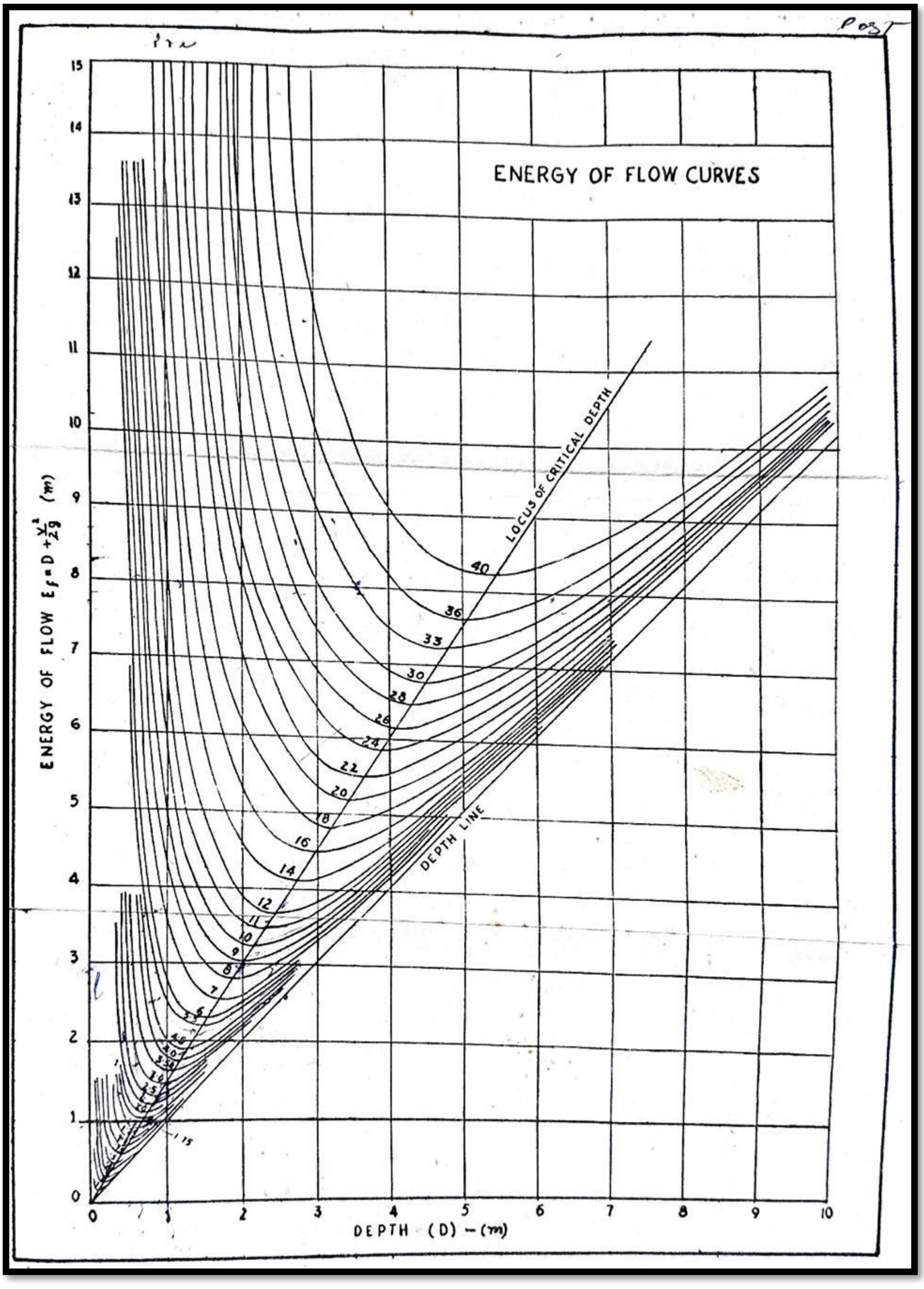


Figure 3.6: Montague curve (Energy flow curves)

3.7 Theories of Sub-Surface flow

Whenever a structure is founded on a pervious foundation, uplift pressure will be exerted beneath the structure by seeping water, in addition to all other forces. The water seeping below the body of the hydraulic structure endangers the stability of the structure and may cause its failure either by uplift or by undermining.

3.7.1 Khosla's Theory

Khosla stated that undermining starts only when the exit gradient is unsafe for the soil on which it is founded and it is necessary to have a deeper d/s cut-off to prevent undermining. The depth of d/s cut-off is determined by

1) Maximum depth of scour
2) Safe exit gradient.

While designing a weir, d/s cut-off from the maximum scoured depth consideration is first taken and checked for exit gradient. If a safe value of exit gradient is not found then the depth of cut-off is increased. Barrage might also fail due to surface flow (i.e. when flood water flows) may cause scour, dynamic action; and in addition, will cause suction pressures in jump trough. These uplift pressures must be checked for different condition of flow. The maximum uplift due to this dynamic action (i.e. for surface flow); should be compared with the maximum uplift under steady seepage (i.e. subsurface flow); and the maximum of the two chosen for designing the aprons and the floor of the Barrages.

To know the behaviour of seepage flow for a given boundary condition, it is required to plot the flow net. For that, we need to solve the flow net either by the electrical analogy method or the mathematical solution of Laplacian equations. These are complicated methods and for this, Khosla suggested a simple, quick and accurate method of Independent Variables. In this method, a complex profile like that of a barrage is divided into a number of simple profiles, each of which can be solved mathematically. He presented the solution of flow nets for these simple standard profiles in the form of curves that can be used for finding out percentage pressures at different key points. The simple profiles which are most useful are:

i) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s or d/s end (Fig.3.7 a and b) .

ii) A straight horizontal floor depressed below the bed but without any cut-off (Fig.3.7c)

iii) A straight horizontal floor of negligible floor thickness with a sheet pile line at some intermediate point. (Fig. 3.7 d)

The key points are the junction of the floor and the pile lines on either side, the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressure at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself if corrected for

(a) Correction for the mutual interference of piles;

(b) Correction for thick. of floor ;

(c) Correction for the slope of the floor.

(a) Correction for the Mutual Interference of Piles.

The correction C to be applied as the percentage of head due to this effect, is given by

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

Where b' is the distance between the two pile lines,

D is the depth of the pile line, the influence of which has to be determined on the neighbouring pile of depth d. D is to be measured below the level at which interference is desired,

d is the depth of the pile line on which the effect is considered and

b is the total floor length.

This correction is positive for the points in the rear or back water and subtractive for the points forward in the direction of flow.

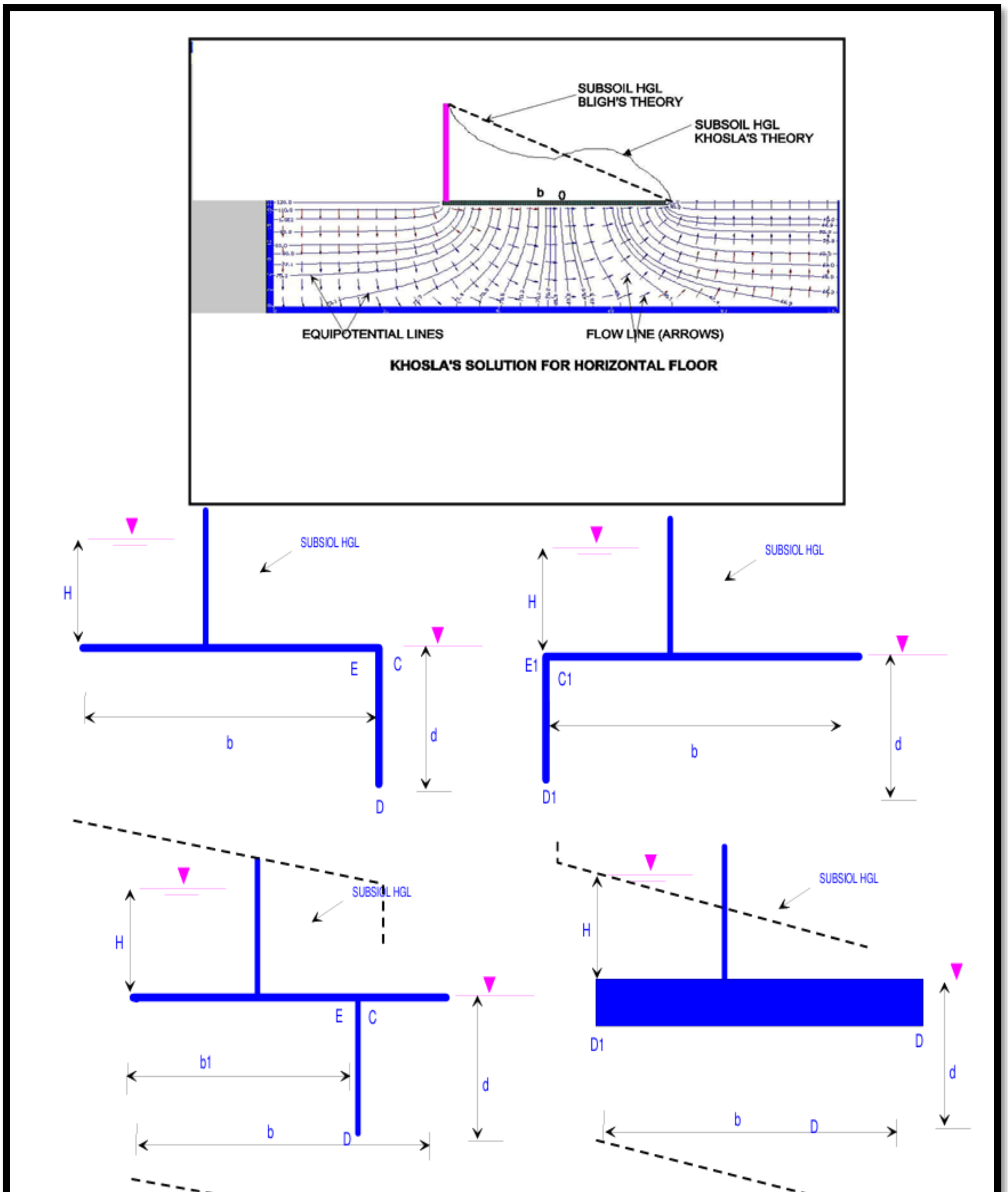


Fig. 8 (a) to (d): Khosla's simplified standard profiles

Figure 3.7 (a) to (d): Khosla's simplified standard profiles

(b) Correction for the thickness of floor:

In the standard form profiles, the floor is assumed to have negligible thickness. Hence, the percentage pressures calculated by Khosla's equations or graphs shall pertain to the top levels of the floor. While the actual junction points E and C are at the bottom of the floor. Hence, the pressures at the actual points are calculated by assuming a straight

Since the corrected pressure at E_1 should be less than the calculated pressure at E_1', the correction to be applied for the joint E_1 shall be negative. Similarly, the pressure calculated C_1' is less than the corrected pressure at C_1, and hence, the correction to be applied at point C_1 is positive.

(c) Correction for the slope of the floor

A correction is applied for a sloping floor and is taken as positive for the downward slopes, and negative for the upward slopes following the direction of flow. Values of correction of standard slopes are tabulated in Table 3.2

Table : 3.2 Slope and correction factors

Slope (H : V)	Correction Factor
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8
6:1	2.5
7:1	2.3
8:1	2.0

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the sloping floor is located. This correction applies only to the key points of the pile line fixed at the start or the end of the slope.

3.7.2 Exit gradient (G_E)

It has been determined that for a standard form consisting of a floor length (b) with a vertical cutoff of depth (d), the exit gradient at its downstream end is given by

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

Where, $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$

$$\alpha = \frac{b}{d}$$

H = Maximum Seepage Head

Table3.3 Soil type and safe exit gradient

Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

The exit gradient calculated should lie within safe limits as given in the Table above.

3.7.3 U/S And D/S Protection Works

Nearly u/s and d/s of the floor of the spillway apron, the streambed is ensured for protection by certain strategies like loose stone apron, block protection, etc. as represented in Figure 3.8 showing a typical section of the spillway of a barrage. These protection works are discussed below:

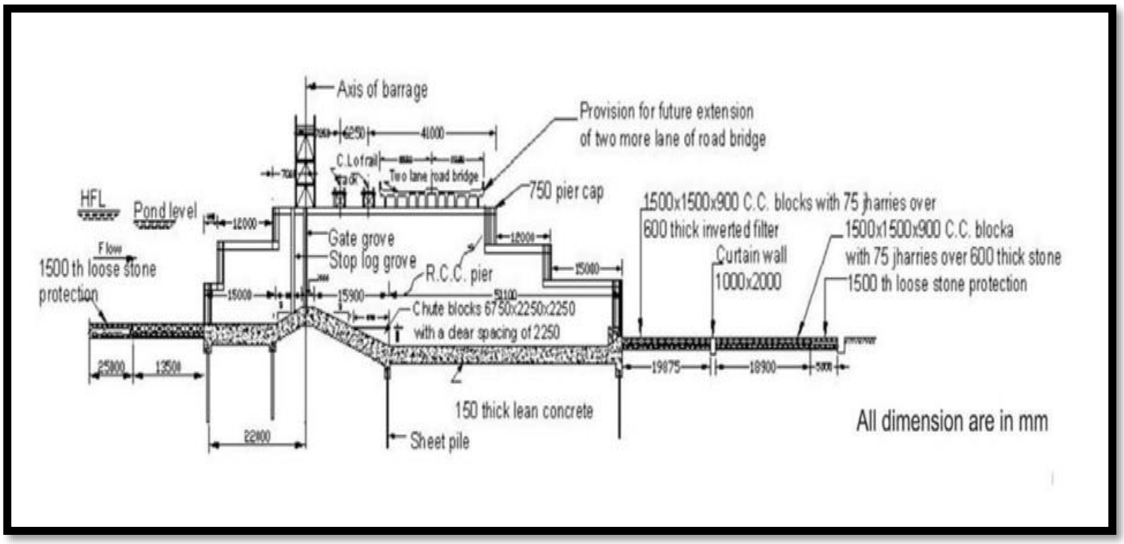


Figure 3.8 Section through a typical barrage

3.7.3.1 Upstream Block Protection

Just beyond the impervious upstream floor, pervious protection consisting of cement concrete blocks of satisfactory size laid over loose stone will have to be provided. The blocks of size around 1.5m x 1.5m x 0.9m made of cement concrete are used for barrages in alluvial streams. The length of the u/s block protection might be kept equivalent to a length D, the design depth of scour beneath the floor level as shown in Figure 3.9.

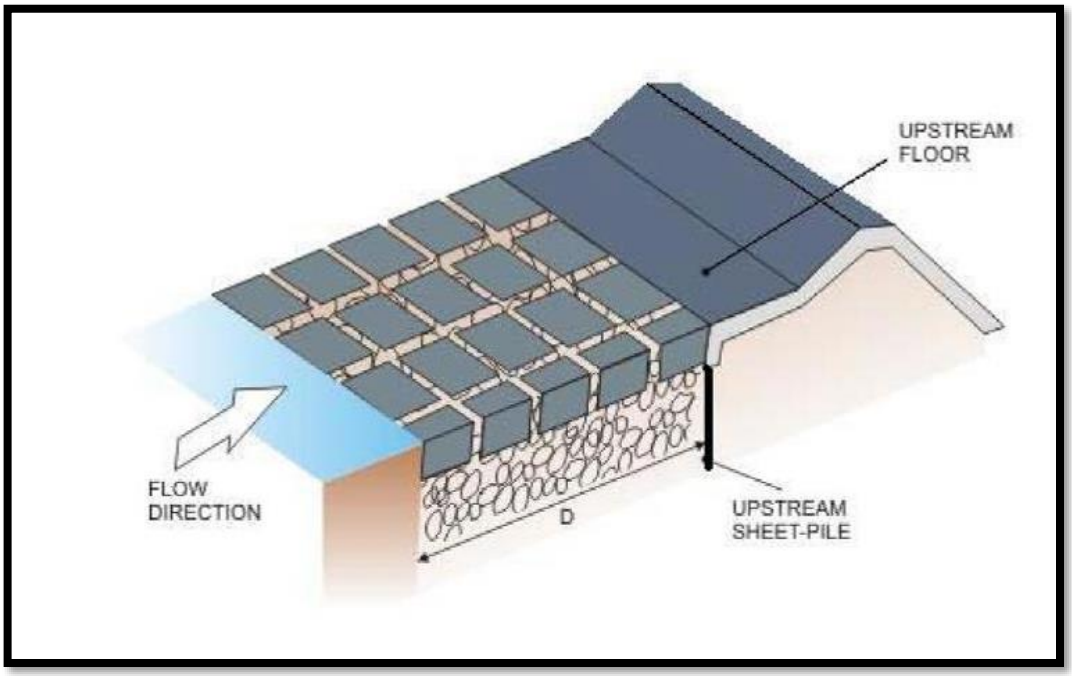


Figure 3.9 Upstream Block Protection

3.7.3.2 Downstream Block Protection

The pervious block protection will be provided just beyond the d/s impervious floor. It contains blocks of size 1.5m x 1.5m x 0.9m made up of cement concrete laid with gaps of 75mm width and are packed with gravel. The d/s block protection is arranged on a graded inverted filter intended to prevent the uplift of the fine sand particles upwards as a result of seepage forces. The filter should roughly follow this design criteria:

1) $\frac{d_{15} \text{ of filter}}{d_{15} \text{ of foundation}} \geq 4 \geq \frac{d_{15} \text{ of filter}}{d_{85} \text{ of foundation}}$

Where d15 and d85 represent grain sizes. Dx is the size such that x% of the soil grains are smaller than that particle size. Where x may be 15 or 85 percent.

2) The filter may be provided in two or more layers. The grain size curves of the filter layers and the base material have to be approximately parallel. The length of the d/s block protection must be 1.5 times D, where D is the depth of cover below the level of the floor. The block protection with an inverted filter may be provided as shown in Figure 3.10.

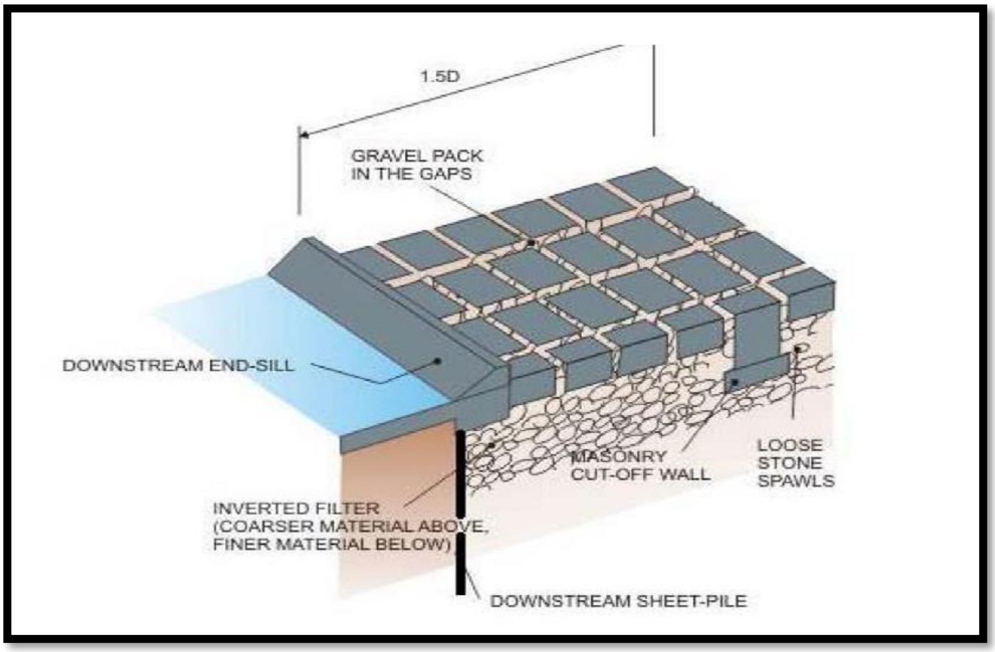


Figure 3.10 Downstream Block Protection

3.7.3.3 Loose Stone Protection

Beyond the block protection on the u/s and d/s of a barrage located on the alluvial foundation, a layer of loose boulders or stones has to be laid, as shown in Figure 3.11(a). The boulder size should be more than or equal to 0.3m and should not weigh less than 40kg. This layer is expected to fall below, or launch, when the downstream riverbed starts getting scoured at the initiation of a heavy flood (Figure 3.11(b). The length of the riverbed that must be protected with loose-stone blocks shall be approximately 1.5D, where D is the depth of scour below the average riverbed.

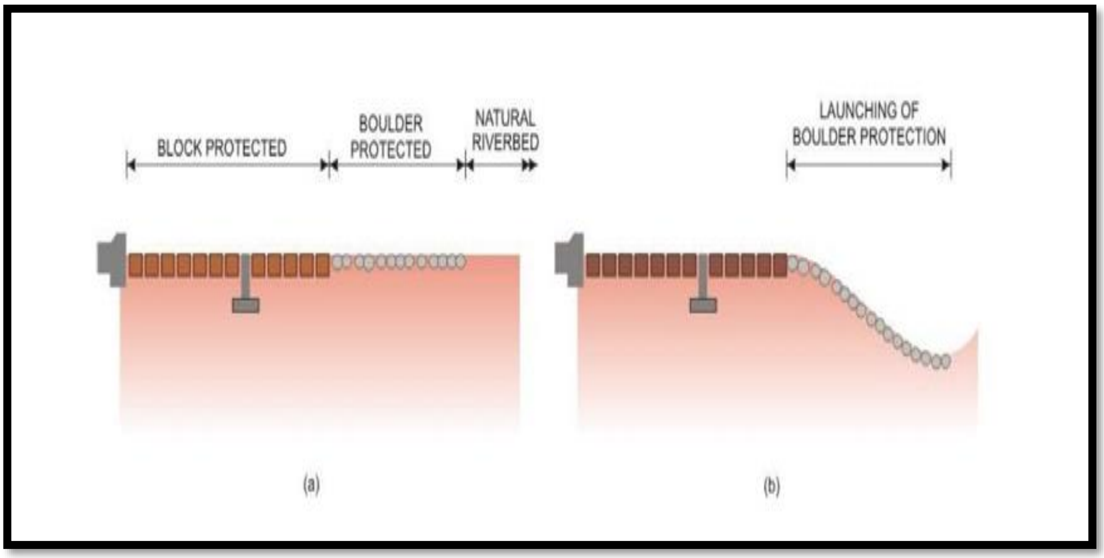


Figure 3.11 Section through downstream protection

It might be mentioned that the loose-stone protection must be laid not only downstream of the barrage floor, but also well as up and down the base of guide bunds, flank dividers, abutment walls, divide walls, and under sluice tunnels.

After fixing the dimensions, they barrage components are designed structurally, considering the forces evaluated from the hydraulic analysis, The Bureau of Indian Standards Code 11130 Criteria for Structural Design of Barrages and Weirs, Bureau of Indian Standards , 1984.specifies the recommendations in this regard.

3.7.3.4 Cut-Off (Sheet Pile)

The upstream and downstream cut-offs of a structure might be steel sheet-piles anchored to the barrage floor utilizing RCC caps, or might be worked of masonry or RCC. The sheet pile cut offs should be made as retaining walls sheet pile anchored at the top end. They will be designed to oppose the worst combination of movements and forces considering possible scour on the external side, earth pressure and surcharge due to floor loads on the internal side, differential hydrostatic pressure computed by the pressure of seepage below the floor etc. In case the impact of cut-offs is taken into account for resistance against the forward sliding of the structure, the cut-offs should also be intended to withstand the passive pressures developed there. The RCC pile caps should be designed to transmit the forces and bending moments acting on the steel sheet piles to the barrage floor.

3.7.3.5 Impervious Floor (Solid Apron)

There are two kinds of floors, the first being called the Gravity type and the second as the Raft type. In the former kind, the uplift pressure is balanced by the self-weight of the floor only considering unit length of the floor, whereas the latter considers the uplift pressure to be adjusted by the floor as well as the piers and other superimposed dead loads considering a unit span. Contemporary outlines of barrages have also been of the raft-type, and therefore, this type of construction is suggested. The thickness of the impervious floor might be adequate to counterbalance the uplift pressure at the considered point. The thickness of the downstream floor (cistern) must be checked under hydraulic jump conditions also, as in this case, the resultant vertical force on the floor is to be calculated from the difference of the vertical uplift resulting from the sub-surface flow and the weight of water column at any point from above because of the flowing water. The design of the raft must be done using the beams on elastic foundations theory and the forces as shown below, or their worst combination has to be taken:

- Differential hydrostatic pressure
- Forces due to water current
- Buoyancy
- Wind forces
- Hydrodynamic forces due to seismic conditions
- Seismic forces, if any

The pier must be designed per the IS-456 as an RCC column.

For the design of the remaining components of a barrage project, like Divide walls, Abutments, Flank walls, Return walls, etc., IS: 11130-1984 should be followed.

These sections are taken from NPTEL, Hydraulic Structures for Flow Diversion and Storage, IIT Kharagpur.

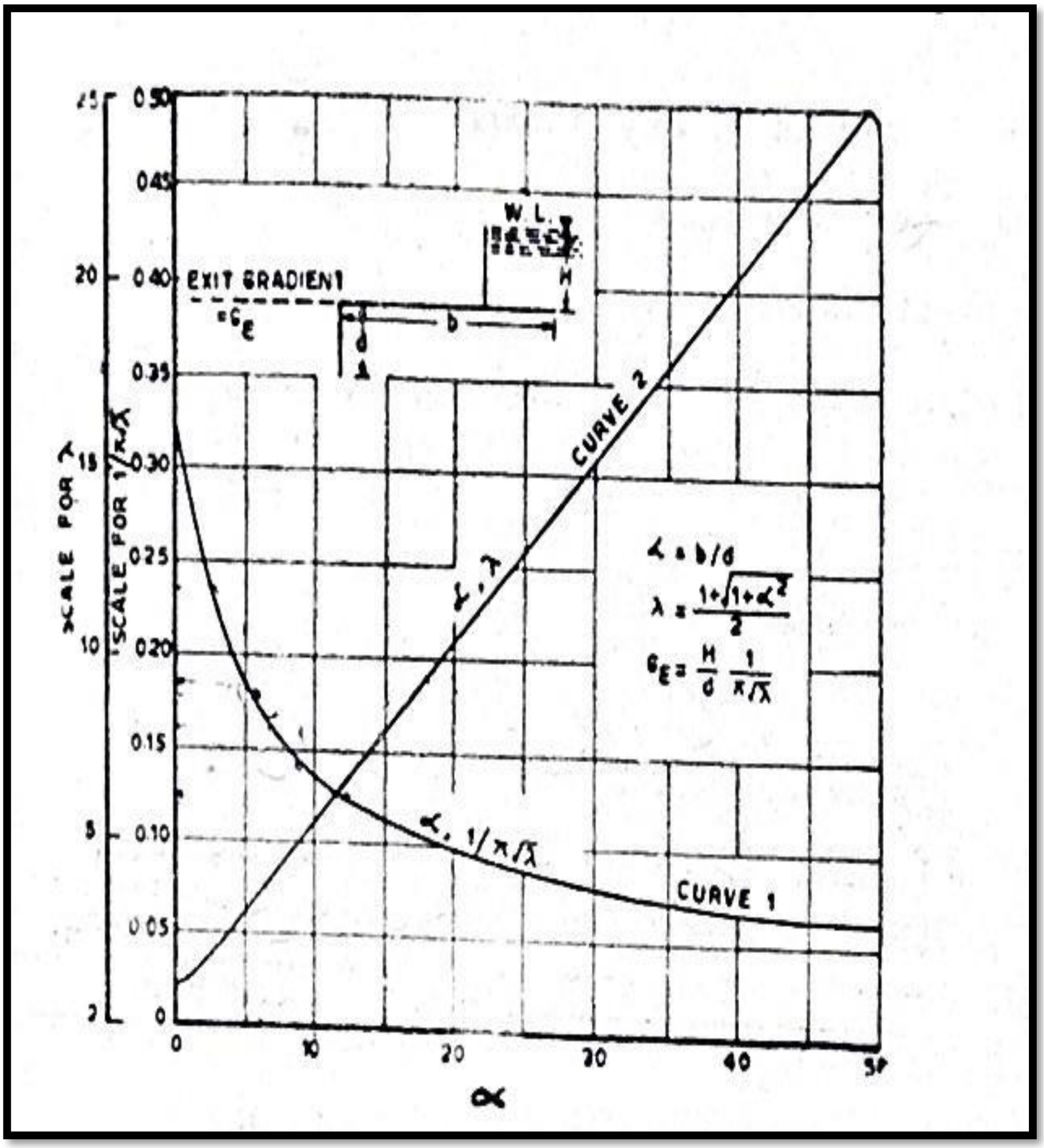


Figure 3.12 : Khosla's safe exit gradient curve

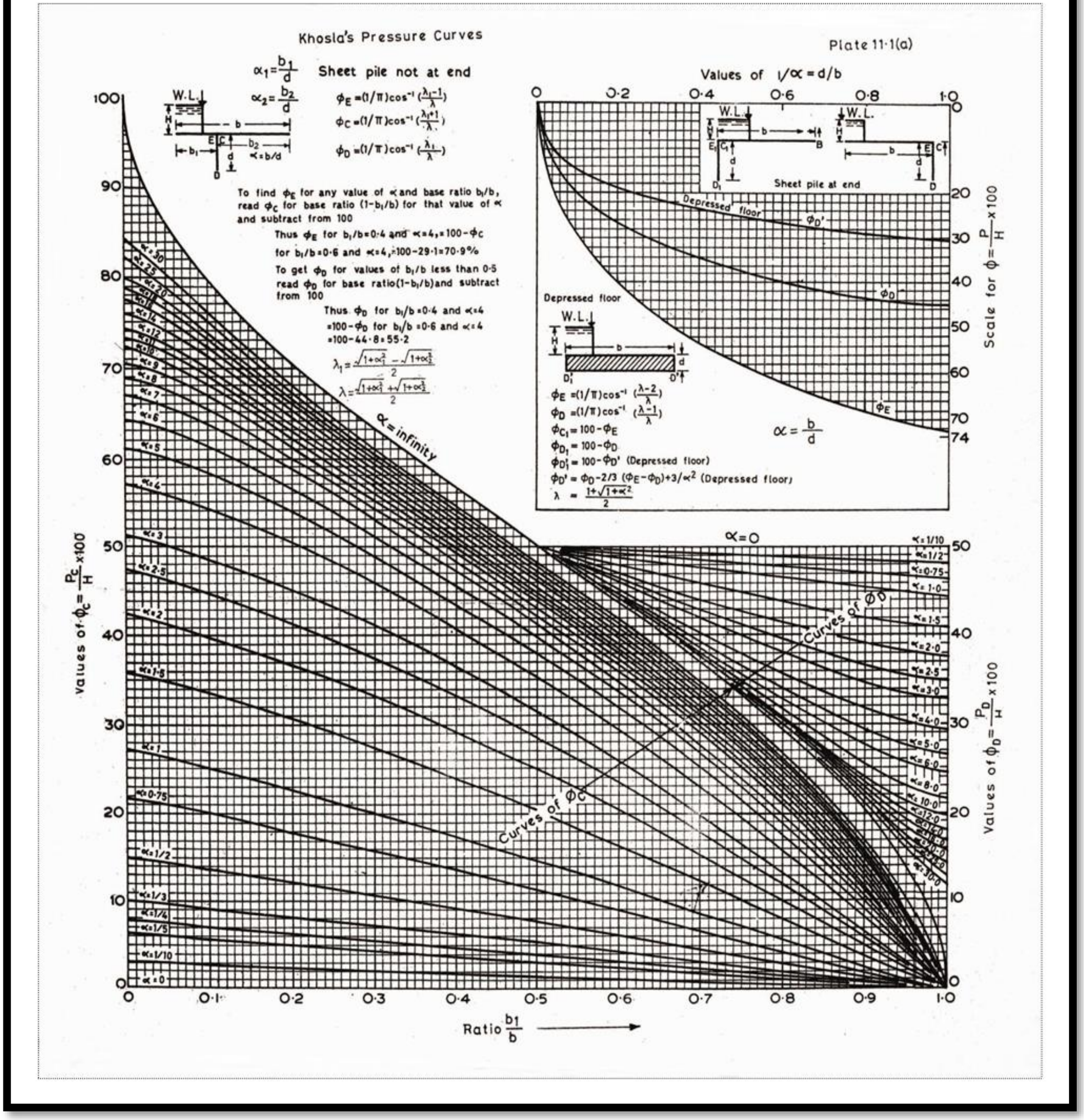


Figure 3.13: Khosla's pressure curve (plate no.1)

CHAPTER 4

STUDY AREA

4.1 Description

The Brahmaputra is the largest river in the Indian subcontinent. With respect of average annual runoff, it is the 4th largest river in the world. It is the third-largest river in the world in terms of sediment load. In its entire reach in Assam from Pasighat to Dhubri, the river flows in an alluvial plain. The Morphology and behaviour of the river Brahmaputra undergo drastic changes in response to various flow regimes and patterns of sediment transportation. The river Brahmaputra is a classic example of a braided river with multiple channels as well as lateral sand bars called chars formed due to excessive sediment load. Due to the braided nature, an oblique/spill channel develops between the sand chars which changes their magnitude and orientation after each flood. These oblique channels are found to be largely responsible for bank erosion. The great earthquake of 1950 is considered a major cause of the unstable character of the river.

The Brahmaputra is 2906 km long. It originates from the Himalayas in Tibet and travels 1625 km in the mountain region of China. It enters India and flows 253 km through the sub-Himalayan ranges in Arunachal Pradesh with the name ‘Dehang’. It confluences with two other courses namely Lohit & Debang and takes the name ‘Brahmaputra’ after it enters the alluvial plains of Assam where it flows a length of 665 km. It flows another 337 km up to the confluence with the Ganges in Bangladesh and ultimately outfalls at the Bay of Bengal. Its total drainage area is 5,80,000 sq.km out of which 2,93,000 sq.km is in China, 45,000 sq.km in Bhutan, 1,94,413 sq.km in India & 47,000 sq.km in Bangladesh. Its bed slope is 1.63m/Km in Tibet and 0.62 m/km in the flood plains up to Kobo near foot hill. The origin of the Brahmaputra is shown in Figure 4.1 and its basin on figure 4.2. Its elevation is 5,300 m at Tibet and 660m at Monko, near entry to India.(Pal et al., 2013)

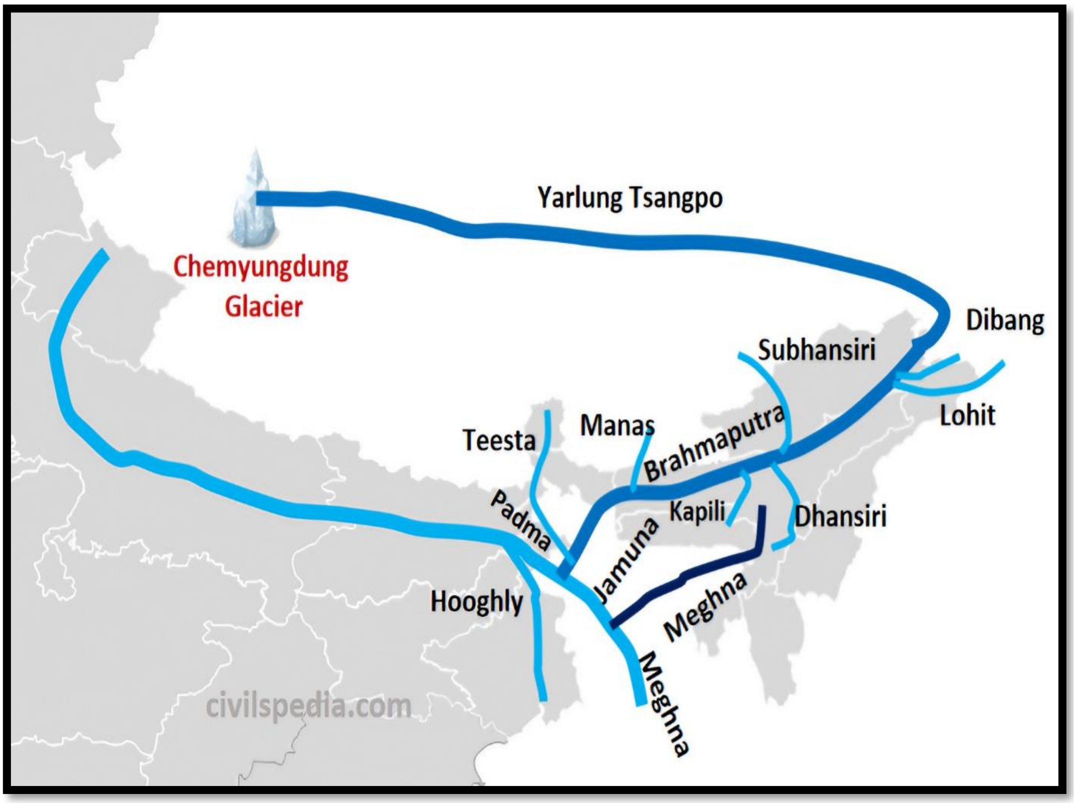


Figure 4.1 Origin of Brahmaputra (source google)

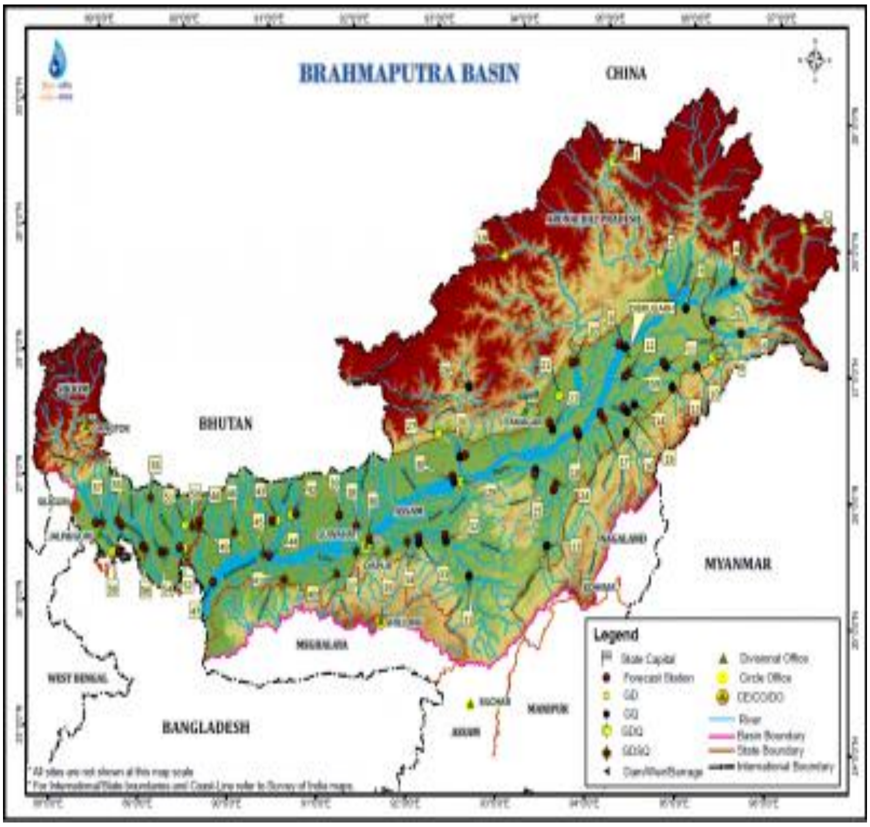


Figure 4.2 Brahmaputra Basin (source India WRIS)

4.2 Salient features of the Study Area

Morigaon is a district in Assam, located on the south bank of the River Brahmaputra, which is highly flood-prone and gets eroded almost every year. The months from May to August experience large-scale bank erosion annually during the rainy seasons under the influence of the southwest monsoon. It is evident that the river Brahmaputra has been shifting slowly southward and became a perennial problem for people living near the bank line of the river in Morigaon district. Erosion has been a regular occurrence in the subdivision of Bhuragaon, Laharighat and Mayong under Morigaon district. The erosion is especially attributed to extreme sediment charge to the braided river and the formation of sand bars within the midst of the river (Nath & Medhi, 2021)

The district occupies part of the Brahmaputra valley and the mighty river Brahmaputra flows on westerly direction along its northern boundary. The district is drained by several perennial rivers flowing from south to north. Rivers Kollong and Kopili are two major rivers. This area falls partly or fully in the quadrants of Survey of India Topo-sheets bearing nos.78 N/15, 78N/16, 83 B/3, 83 B/4, 83 B/6, 83 B/7, 83 B/8 . The base map of the study area is shown in fig.4.3.**(*"AQUIFER MAPPING AND MANAGEMENT*, 2022.)**

The mighty Brahmaputra flows along with the northern boundary of the district. The district is drained by Kapili, Kollong, Kiling and Sonai rivers. The Kapili and Sonai rivers are tributaries to Brahmaputra River. There are a number of Beels which are remnants of old channels of these rivers. They are Arum, Kiling, Sikhora and Bar Jalah Beels in the vicinity of Kiling river ; Sholmari, Dandua, Marakalang, Sara, Sarumanaha, Dekhal, Baral, Habari, Nakara Maudubi, Goranga, Taranga, Donga, Jan, Khar, Udmari Bils along the Kapili river and Garajan Khanagharia-Srikanda, Chilpi-Bhangamur-Goroimari-Goranga Bils adjacent to Sonai river. These are the indications of tectonic disturbances the area has undergone. The Kiling river joins Kapili, west of Dharamtul and which in turn joins Kalang NNE of Jagiroad. The Kapili drains the southern part of the district and the Kiling, Mikir Hills. The northern part of the district is drained by Sonai river.

<u>Morigaon</u>

Latitude	22° 19' to 28° 16' N
Longitude	89° 42' to 96°30' E

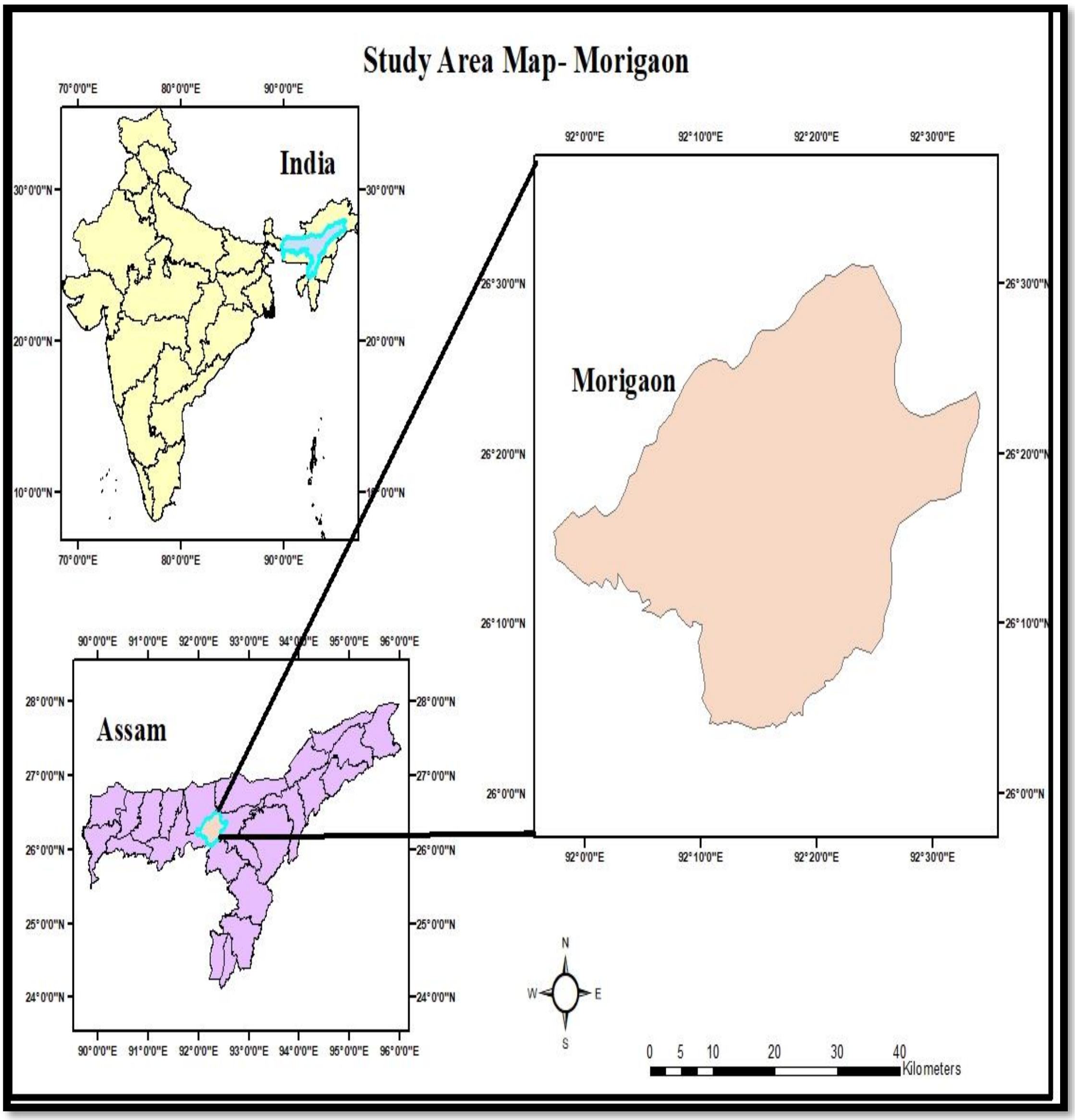


Figure 4.3 Location map of the study area

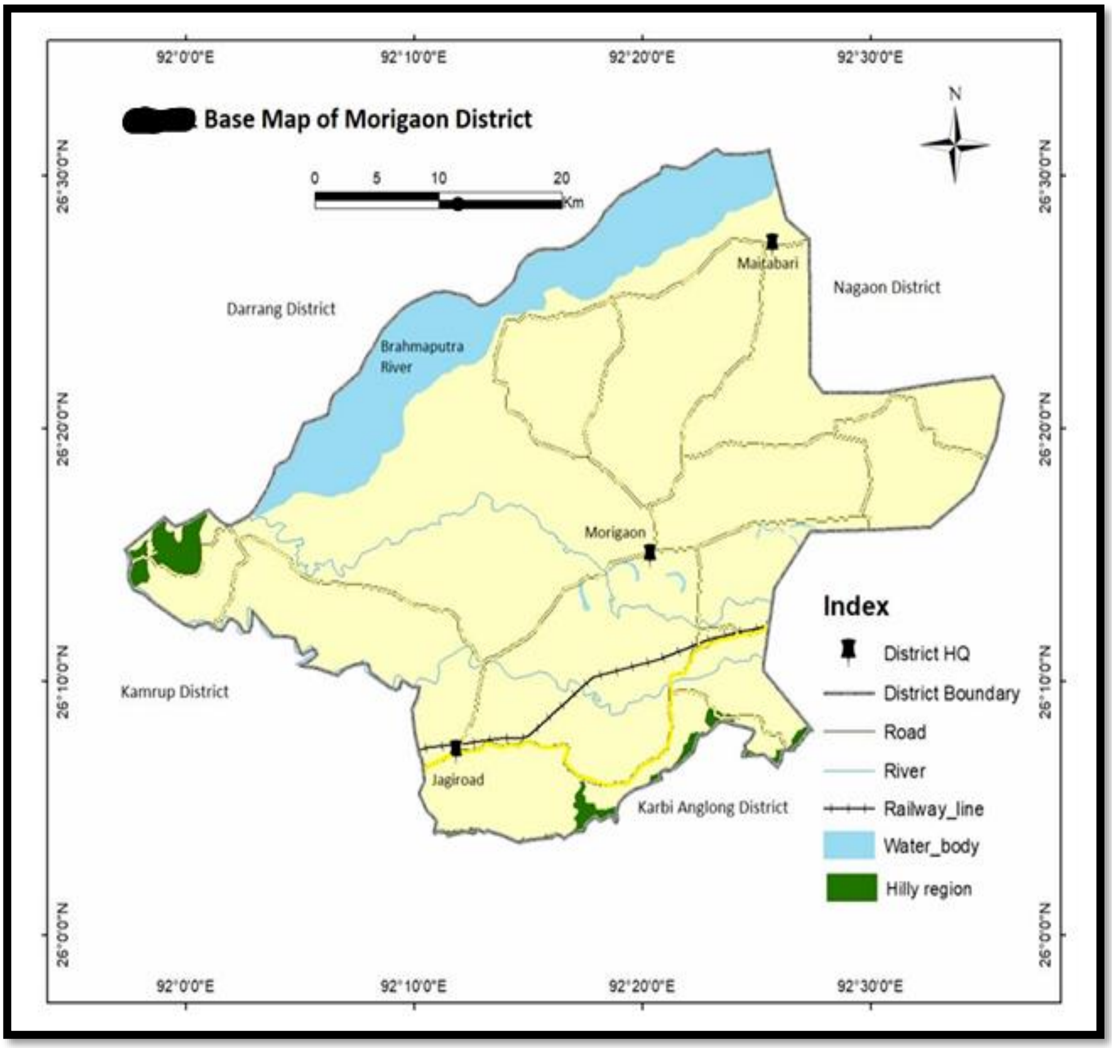


Figure 4.4 Base map of Morigaon District

4.3 Climate

The climate in the area is characterized by tropical and humid type. Based on the monthly mean temperature efficiency ratio (Thermal efficiency ratio) Koppen and Geiger 1930). The thermal efficiency ratio for Morigaon district is 133.90. Since this value is more than 128, the climate is classified as tropical (Koppen & Geiger). The district receives rainfall mainly from the southwest monsoon which commences in the month of may and lasts till October, while the maximum rainfall occurs during the month of July while October and November forms the hot monsoon period. The average annual rainfall is 1770 mm. More than 93 % of annual rainfall occurs between April to September. The winter rains account only 7% of annual precipitation. The rainfall in general decreases from south to north. The district has sub-tropical and humid type of climate. The humidity data reveal that the air is humid throughout the year. It has the value of 67% to 79 % during dry period. The cold season is found from December to February followed by the pre monsoon season of thunder storm from March to May. The months of March, April and May are the period of Pre-monsoon season. The temperature begins to rise from the month of March. The frequency of storms also increases during these months. During the monsoon period, due to heavy rainfall, the weather is suitable for crop plantation especially paddy. The average altitude of the study area is 79m.

In Morigaon district , the wet season is hot, oppressive, and mostly cloudy and the dry season is warm and clear. Over the course of the year, the temperature typically varies from *11°C* to *32°C* and is rarely below *9°C* or above *35°C*. The maximum annual rainfall observed in last 20 years is 124 mm. The most precipitation falls in July.

4.4 Hydrology

The Kopili River is the largest south bank tributary of the Brahmaputra River in Assam, India. It originates in the Meghalaya Plateau and flows through Central Assam and the hill districts of Assam before ultimately joining the Brahmaputra. The river traverses through the districts of Karbi Anglong, Dima Hasao, Kamrup, and Nagaon in Assam.

The river's course takes it through diverse landscapes, including plateaus and hills, contributing to the drainage of these regions. The Kopili River plays a crucial role in the hydrology and ecology of the areas it passes through, influencing local communities and ecosystems. Rivers like Kopili often hold cultural, economic, and ecological significance for the regions they flow through. They provide water resources for agriculture, support biodiversity, and have historical and cultural importance for the local populations.

The Kolong River, or Kailang, is an anabranch of the Brahmaputra River in Assam, India. It diverges from the Brahmaputra River in the Hatimura region of Jakhalabandha, which is in the Nagaon district of Assam. The anabranch rejoins the Brahmaputra at Kolongpar near Guwahati. The Kolong River has a length of approximately 250 kilometers (160 miles) and flows through the districts of Nagaon, Morigaon, and Kamrup. During its course, the Kolong River receives contributions from various smaller streams, including the Diju, Missa River, and others. As it flows through the region, the river plays a significant role in the local hydrology and influences the landscape. It passes through the heart of the Nagaon urban area, dividing the town into Nagaon and Haiborgaon. Rivers like the Kolong have ecological, cultural, and economic importance for the regions they traverse. They often serve as vital water sources for agriculture, provide habitats for diverse flora and fauna, and have cultural significance for the communities along their banks.

At Murkata, Mayong, the Brahmaputra River empties into the Pokoria River, flooding a vast expanse of arable land and homesteads. The flood has threatened or impacted the afflicted area over the past few years. A sizable portion of the riverbed has already been lost as a result of erosion, negatively affecting the ecosystem. Throughout the whole monsoon season, the area is submerged in water. The Brahmaputra River's backflow through the Pokoria Channel submerges between 7000 and 8000 hectares of land, mostly agricultural and fishery area.

With the above-mentioned prevailing conditions, there is an increasing demand from diverse segments of society to reassess the river flow, address erosion, and rejuvenate the riverine ecosystem. This urgency stems from the river's profound connection to the heritage and sentiments of the inhabitants of Morigaon district.

4.4 River Pokoria

The Pokoria River about 76.37 Km long, originates in Nagaon district near Chaparmukh, Raha at location 26° 13’ 26” N and 92° 31’ 11” E after the merge of Kollong River with rivulet from Kopili river .The river flows from Nagaon district to Morigaon and joins Brahmaputra at 26° 16’ 37” N and 92° 02’ 51” E at Kasohilla Mayong. The Pokoria river receives runoff from Lali channel, Sunduba channel etc. Figure 4.5 shows the Pokoria river journey through the region. The river begins near Raha Gaon, as indicated by the blue marker on the right side of the map. The yellow line traces the river's path as it meanders through various regions. It passes through areas like Morigaon, Bhurbandha, and Mikirbheta, flowing generally in a westward direction. The river ultimately converges and ends near Mayang, as indicated by the blue marker on the left side of the map and flows into Brahmaputra.

Figure:4.5 Origin of Pokoria river

The Pokoria River watershed, positioned within the coordinates of latitude 26°40′ N to 26°58′ N and longitude 92°04′ E to 93°15′ E, spans an expansive area covering approximately 1988.08 square kilometre, as depicted in figure 4.6. This watershed holds significant geographical importance within the Morigaon district, being home to the Pokoria River, a major watercourse that meanders across the landscape for about 140.82 kilometres. Originating amidst the foothills of Mikir, the river embarks on a journey through the scenic terrains of the Morigaon district, carving its path and contributing to the region's ecological and hydrological dynamics. Eventually, the Pokoria River gracefully merges its waters with the mighty Brahmaputra River near the vicinity of Mayang, where its flow becomes part of the larger aquatic ecosystem of the Brahmaputra basin. The location of the study area is situated at a distance of about approximately 32km from Morigaon on the South bank of river Brahmaputra. The Ulubari rain Gauge station is 40 km u/s and Pandu rain Gauge station is 37km d/s of the Pokoria channel mouth figure 4.7.

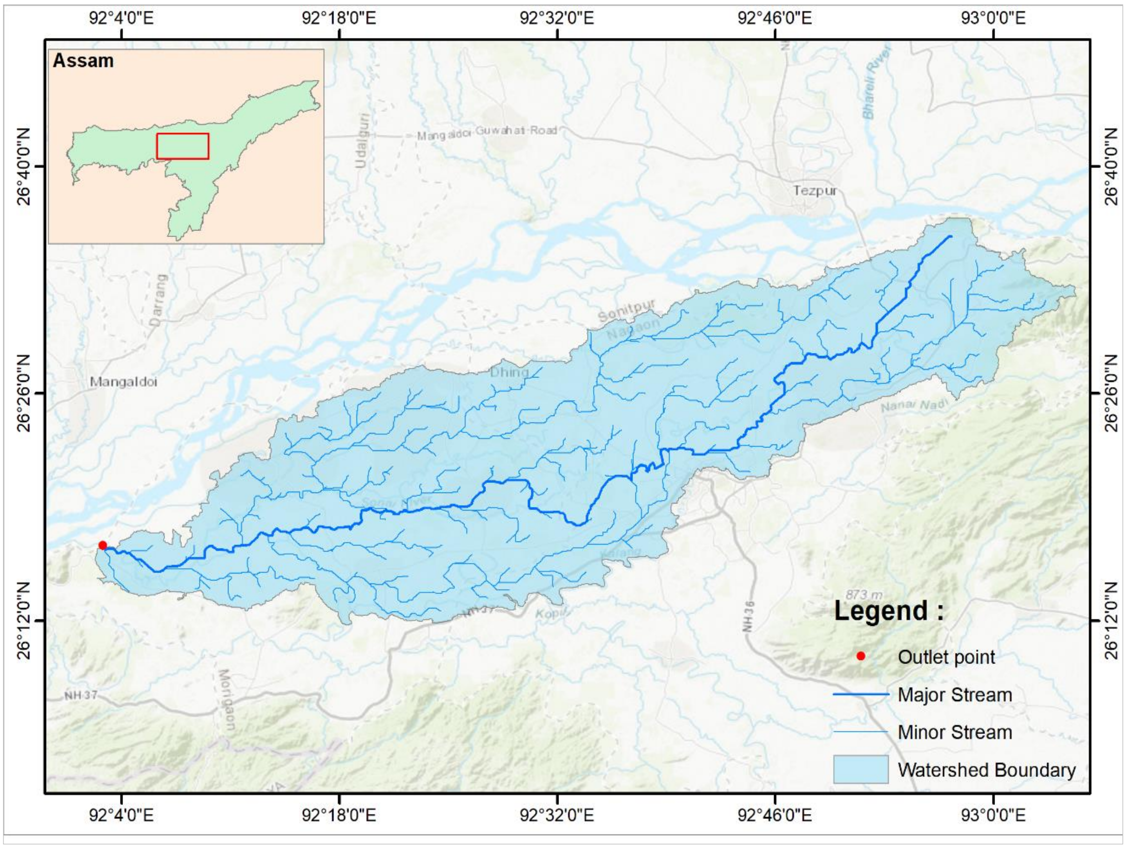


Figure 4.6 Watershed of Pokoria river

Figure 4.7 Raingauge location.

4.5 Problem

During Monsoon the Brahmaputra River on the south bank at Murkata, Mayong area backflows i.e. flow from Brahmaputra to the countryside, through the Pokoria river and inundates a huge area of homestead and cultivable land. The concerned reach has been under threat/ affected by the flood for the past several years. Due to the erosion, a considerable area has already been lost in the river bed with a negative impact on the environment. The area remains inundated for the entire monsoon season. The backflow of river Brahmaputra through the Pokoria channel inundates approximately 7000-8000 hectares of land comprising large numbers of fisheries and agricultural land.

During the non-monsoon period, the Pokoria River flows into the Brahmaputra i.e. from the countryside to the river Brahmaputra, leading to water shortages for the local inhabitants. Installing sluice gates or barrages at strategic points can help regulate the flow of water between the Pokoria and Brahmaputra rivers. These structures can be used to control the flow of water during both monsoon and non-monsoon periods. From Figure 4.8 Approximate measurements of the Proposed Sluice Site based on the scale of the map are 1-2 km to the north to Bharali Store, 3-4 km to the northwest to Dolphin Viewing Point, 2-3 km to the west to Xorapat Eco Resort, 1.3km from existing bank dyke and 2km from existing PWD road.

Figure 4.8 Plan for proposed Sluice at Pokoria Channel

4.6 Glimpse of affected area during Monsoon.

Figure 4.9 (A):Garmari Bangalpara Sukutiputa road Submerged at 3rd and 4th

Figure 4.9(B): Sildubi Gagalmari to Kharkharijaan road submerged at 1st

Figure 4.9(C): Chotogarjan to Karchowabori road submerged at 5th &

Figure 4.9(D):Morisuti Tup to Murkata No.1 road submerged at 3rd & 4th km

Photo courtesy Water Resource Department, Assam.

CHAPTER 5

METHODOLOGY

5.1 Data pre-processing

For hydrological databases, long-term historical daily rainfall data during the monsoon period i.e. from May to October is collected from the Water Resources Department of the state, for a period of 21 years (2003-2023). The data includes high-resolution rainfall measurements as it is daily rainfall for over many years(21 years).

For the generation of maximum rainfall, daily precipitation data in the study area are obtained from Daily precipitation data, was disaggregated into smaller durations of 30 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 12 hours and 24 hours.

5.2 Daily Rainfall Disaggregation Method

From the daily precipitation database, the annual maximum series (AMS) was extracted and disaggregated into shorter duration rainfall series by using the Indian Meteorological Department (IMD) one-third reduction formula in the equation. ***Chowdhury et al. (2007)*** used the IMD one-third reduction formula to estimate short-duration rainfall for the Morigaon district and found the formula gave the best estimation for short-duration rainfall (Rashid et al., 2012).

$$p_t = p_{24}\left(\frac{t}{24}\right)^{1/3} \tag{5.1}$$

Where,

p_t denotes the rainfall depth (millimetres) for t-hours duration,

p_{24} denotes the daily rainfall (millimetres),

and t denotes the duration (hour) of rainfall for which the rainfall depth is required.

The rainfall events were analysed by breaking down the AMS into shorter durations of 30 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 12 hours and 24 hours. For each duration, the maximum rainfall intensity for each year is determined.

5.3 Intensity-Duration-Frequency Analysis

The intensity duration frequency analysis was started by gathering the daily precipitation data. After the precipitation data was collected, the annual maximum was extracted from the records. The annual maximum data was then fitted to probability distribution functions to estimate rainfall quantities for desired return periods.

5.4 Gumbel Theory of Distribution

The Gumbel method calculates the 2, 5, 10, 25, 50, 75 and 100-year return intervals for each duration period and requires several calculations. Frequency precipitation P_T (in mm) for each duration with a specified return period T (in year) is given by the following equation:

$$\boldsymbol{P_T = P_{ave} + KS} \qquad \textbf{(5.2)}$$

Where K is the Gumbel frequency factor given by:

$$\boldsymbol{K = -\frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left[\ln\left[\frac{T}{T-1}\right]\right]\right]} \qquad \textbf{(5.3)}$$

Where P_{ave} is the average of the maximum precipitation corresponding to a specific duration.

In utilizing Gumbel's distribution, the arithmetic average in equation
4.4 is used:

$$\boldsymbol{P_{ave} = \frac{1}{n}\sum_{i=1}^{n} P_i} \qquad \textbf{(5.4)}$$

Where Pi is the individual extreme value of rainfall and n is the number of events or years of record. The standard deviation is calculated by equation (4.5) computed using the following relation:

$$S = [\frac{1}{n-1}\sum_{i=1}^{n}(P_i - P_{ave})^2]^{1/2} \qquad (5.5)$$

Where S is the standard deviation of P data. The frequency factor (K), which is a function of the return period and sample size, when multiplied by the standard deviation gives the departure of a desired return period rainfall from the average. Then the rainfall intensity, I_T (in mm/hr) for return period T is obtained from:

$$I_T = \frac{P_t}{T_d} \qquad (5.6)$$

Where T_d is duration in hours.

The frequency of the rainfall is usually defined by reference to the annual maximum series, which consists of the largest values observed in each year. An alternative data format for rainfall frequency studies is based on the peak-over threshold concept, which consists of all precipitation amounts above certain thresholds selected for different durations. Due to its simpler structure, the annual-maximum-series method is more popular in practice.

5.5 Area Velocity Method

The area-velocity method is effective for estimating streamflow in the absence of high-resolution DEMs because it relies on direct physical measurements rather than detailed topographic data. This method is practical, cost-effective, and can provide accurate real-time flow data, making it a valuable tool for hydrological studies and water resource management.

As the name suggests, in this method, discharge is computed by measuring river depths and velocity at a number of regularly or irregularly spaced verticals. This set of information is eventually integrated by mid-section method to determine river discharge. This satisfies the equation

$$\mathbf{Q = A * V.} \qquad (5.7)$$

Succeeding paragraphs of this module elaborate on these aspects in greater depth.

5.6 Design of Sluice

Sluice gates are vital hydraulic structures used to control water levels and flow rates in various environments. By regulating gate openings, they manage upstream water levels in rivers, streams, and irrigation networks. Proper design, material selection, and management of sluice gates are crucial to optimizing their performance and preventing water waste. Through effective use of sluice gates, water resources can be better managed to meet the needs of agriculture, flood control, and water treatment, ensuring sustainable and efficient water distribution.

5.6.1 Initial Data

To proceed with the hydraulic design of the sluice following data and parameters are necessary: *Varshney, R. S., Gupta, S. C., Gupta, R. L. (1979). Theory and Design of Irrigation structures. India: Nem Chand & Bros..*

(i) Maximum flood discharge (Q)

(ii) Stage discharge curve of the river at the site

(iii) Minimum water level

(iv) Cross section of the river at the sluice site

The following has to be decided

(i) Lacey silt factor (f). This is determined from the equation, $f = 1.76\sqrt{M_r}$

(ii) Length of waterway, discharge per metre and afflux

(iii) Safe exit gradient

(iv) Depth of sheet piles in relation to (i) scour depth and (ii) exit gradient.

(v) Level and length of the horizontal part of the downstream impervious floor in coordination with hydraulic jump.

(vi) Thickness of downstream impervious floor:

(a) with reference to uplift pressure,

(b) with reference to hydraulic jumps or standing waves.

(vii) Length and thickness of protection works beyond the pucca floor upstream and downstream.

5.6.2 Stepwise procedure of design

Designing a sluice involves several critical steps that combine hydrological, hydraulic, structural, and environmental considerations. Here's a step-by-step procedure for sluice design

STEP I : Determine head loss (HL) for different flow conditions:

- If there is no retrogression, $\boldsymbol{H_L = afflux}$
- If allowance for retrogression is taken in downstream bed level, then $\boldsymbol{H_L = afflux + Retrogression.}$ Usually, 0.5 m retrogression is sufficient in most cases.

STEP II: For known values of q and H_L corresponding values of E_{f2} is obtained from Blench curves (Fig. 4.2). With known values of E_{f2} corresponding values of D_2 is obtained. In this study Direct equations as stated by(Chaurasia, n.d.) are used.

$$D_1 = 0.196q^{0.98}H_L^{-0.47}[1 + 1.262qH_L^{-1.50}]^{-0.206}$$

$$D_2 = 1.0518q^{0.518}H_L^{0.223}[1 + 0.0476q^{2.667}H_L^{-4.0}]^{-0.0221}$$

$$E_{f2} = 1.029q^{0.513}H_L^{0.223}[1 + 0.4329q^{0.733}H_L^{-1.10}]^{0.1631}$$

$$\boldsymbol{Cistern\ Level = Downstream\ T.E.L - E_{f2}}$$

STEP III: Find $\boldsymbol{E_{f1} = E_{f2} + H_L}$. Knowing E_{f1} , E_{f2} and q values of D_1 and D_2 are obtained from energy flow curves at Figure 4.2

Provide minimum cistern length = $\boldsymbol{5(D_2 - D_1)}$

STEP IV: Determine scour depth from the formula

$$\boldsymbol{R = 1.35\ (\frac{q^2}{f})^{\frac{1}{2}}}$$

- Depth of upstream sheet pile form scour consideration 1 R to 1.25 R
- Depth of downstream sheet pile from scour consideration = 1.25 R to 1.5 R

An intermediate pile line need not normally be provided. If at all provided, its depth should not be less than that of the upstream pile line.

STEP V: The value of $\frac{1}{\pi\sqrt{\lambda}}$ from the equation $\frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H}$, is worked out for the given values of G_E and the known values of d (downstream depth of sheet pile) and H (Maximum static head). Value of α is obtained with the Corresponding value of $\frac{1}{\pi\sqrt{\lambda}}$ from Figure 4.3.

STEP VI: Provide total length of floor (b) =α d

Disposition of total floor length may be as follows:

(1) Cistern length = $\mathbf{5\ (D_2 - D_1)\ to\ 6\ (D_2 - D_1)}$

(2) Glacis length = 3 to 5 times (crest level-cistern level) for **3: 1 to 5:1** slope of glacis.

(3) upstream floor = the balance.

If the total length is excessive, it would be economical to reduce it by providing a deeper downstream sheet pile.

STEP VII: To determine uplift pressures acting on the floor, the % pressures at upstream and downstream sheet pile lines are worked out. The pressure distribution from the upstream sheet pile line to the downstream sheet pile line is assumed to be linear.

% pressures at the upstream sheet pile line.

For the known values of b and d_1, $\frac{\mathbf{1}}{\boldsymbol{\alpha}} = \frac{\boldsymbol{d_1}}{\boldsymbol{b}}$. With the known value of α , $\emptyset_D$ and $\emptyset_E$ can be obtained from plate no. 1 as in Figure 4.4

% pressure at the bottom of sheet pile **= 100 -** $\boldsymbol{\emptyset_D}$

% pressure at the bottom of the floor =**100 -** $\boldsymbol{\emptyset_D}$

% pressures at downstream sheet pile line.

From the known values of b and D_2, $\frac{1}{\alpha} = \frac{d_2}{b}$, values of $\emptyset_E\ and\ \emptyset_D$ are obtained corresponding to $\frac{1}{\alpha}$ from plate no. 1 which would be % pressure at the bottom of the floor and sheet pile respectively

% pressures at the intermediate pile line.

For the known values of the depth of the intermediate pile line d_3 and total floor length b, determine $\alpha = \frac{b}{d_3}$. Also, the base ratio is calculated by

$$\frac{\boldsymbol{b_1}}{\boldsymbol{b}} = \frac{\boldsymbol{Horizontal\ distance\ between\ upstream\ pile\ and\ intermediate\ pile}}{\boldsymbol{Total\ floor\ length}}$$

The value of $\emptyset_D$can be read directly from plate no 1 for given values of α and base ratio $\frac{b_1}{b}$. To find $\emptyset_E$ for the known value of α and base ratio $\frac{b_1}{b}$, read for base ratio $1 - \frac{b_1}{b}$ for that value of α, and subtract from 100. To find $\emptyset_D$ for $\frac{b_1}{b}$ less than 0.5, read $\emptyset_D$ for base ratio $1 - \frac{b_1}{b}$ and subtract from 100.

Step VIII: The thickness of the floor at the location of the sheet piles are tentatively assumed for correcting the values of $\emptyset_c$ in the upstream and $\emptyset_D$ in the downstream. If t_1 is the floor thickness at the upstream sheet pile of depth d_1, correction due to floor thickness $= \frac{t_1}{d_1}(\emptyset_D - \emptyset_c)$ which is positive. If t_2 is the floor thickness at the downstream sheet pile of depth d_2, the correction $= \frac{t_1}{d_1}(\emptyset_E - \emptyset_D)$ which is negative.

The correction due to mutual interference of sheet piles is worked out by

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

Step IX: The protection works are designed in respect of scour depth.

5.7 Microsoft Excel :

MS Excel is used as a design platform as an innovative approach, leveraging its grid system, data manipulation capabilities, and visualisation tools. Excel may not replace professional design software, but it can be a surprisingly versatile tool for planning, prototyping, and visualising ideas quickly.

Designing a barrage is indeed a complex and time-consuming task that involves numerous calculations and data management. Utilizing cell referencing in Microsoft Excel can significantly streamline this process. Here's how cell referencing in Excel can help in designing a barrage: Efficient Data Management, Dynamic Calculations, Complex Formula Handling, Scenario Analysis and What-If Calculations, Scenario Analysis and visualisation.

Using cell referencing in MS Excel for designing a barrage provides a systematic and efficient approach to handle complex calculations and data management tasks. It enhances accuracy, reduces the time required for design iterations, and facilitates better decision-making through dynamic analysis and visualization tools. So, in this project design has been done on the MS Excel platform by cell referencing which seems helpful in respect of time saving.

CHAPTER 6

DATA COLLECTION AND GENERATION.

In this chapter, all the necessary data required for the hydraulic design of the sluice has been collected from different sources and is described in it.

6.1 Rainfall data:

For the study, daily monsoon (May to October) rainfall data for 21 years from 2003 to 2023 is collected from the Morigaon Division Water Resources Department. With these data IDF curves and Peak rainfall for particular return periods are generated.

Table 6.1 Monthly rainfall (in mm) recorded from 2003 to 2023 during Monsoon period

Month→ Year↓	May	June	July	August	September	October	Annual Maximum rainfall
2003	31.200	33.000	124.500	53.800	39.900	104.100	**124.500**
2004	30.500	59.700	41.900	30.500	52.100	111.800	**111.800**
2005	12.700	41.900	32.500	43.200	33.000	12.200	**43.200**
2006	31.500	83.800	79.200	40.600	5.600	10.700	**83.800**
2007	35.600	57.100	73.700	71.900	53.300	45.700	**73.700**
2008	7.100	58.400	63.500	44.900	73.700	10.700	**73.700**
2009	44.400	31.700	66.000	57.900	57.900	37.100	**66.000**
2010	45.700	87.600	83.800	43.700	71.100	22.900	**87.600**
2011	22.300	55.900	36.300	57.100	24.900		**57.100**
2012	20.300	63.500	27.400	46.500	58.400	9.400	**63.500**
2013	17.800	44.400	70.300	38.100	31.000	26.700	**70.300**
2014	27.900	32.200	38.100	62.200	74.900	3.500	**74.900**
2015	35.600	63.500	41.100	36.100	34.800	3.500	**63.500**
2016	48.300	61.500	50.800	32.500	43.700	20.800	**61.500**
2017	36.800	61.000	27.200	76.200	76.200	25.900	**76.200**
2018	17.800	36.300	69.100	44.400	13.700	14.700	**69.100**
2019	44.200	17.800	41.600	37.600	76.200	65.500	**76.200**
2020	68.600	56.600	50.800	88.900	86.900	25.400	**88.900**
2021	20.800	30.400	38.100	121.900	80.000	119.400	**121.900**
2022	21.600	53.300	33.500	45.700	31.700	67.300	**67.300**
2023	54.600	60.000	101.600	99.100	27.400	61.200	**101.600**
Maximum rainfall for the month	**68.600**	**87.600**	**124.500**	**121.900**	**86.900**	**119.400**	**124.500**

The highest daily peak of each year is used to disintegrate into minutes from 30min to 24hr i.e 24 hr using formula $p_t = p_{24}\left(\frac{t}{24}\right)^{1/3}$ which are placed in the table below and corresponding mean and standard deviation is calculated.

Table 6.2 Annual Maximum daily rainfall distributed for 24 hourly

Year	Annual rainfall	Duration						
		30min	**1hr**	**2hr**	**3hr**	**6hr**	**12hr**	**24hr**
2003	124.500	34.257	43.162	54.380	62.250	78.430	98.816	124.500
2004	111.800	30.763	38.759	48.833	55.900	70.430	88.736	111.800
2005	43.200	11.887	14.977	18.869	21.600	27.214	34.288	43.200
2006	83.800	23.058	29.052	36.603	41.900	52.791	66.512	83.800
2007	73.700	20.279	25.550	32.191	36.850	46.428	58.496	73.700
2008	73.700	20.279	25.550	32.191	36.850	46.428	58.496	73.700
2009	66.000	18.161	22.881	28.828	33.000	41.577	52.384	66.000
2010	87.600	24.104	30.369	38.263	43.800	55.185	69.528	87.600
2011	57.100	15.712	19.795	24.941	28.550	35.971	45.320	57.100
2012	63.500	17.473	22.014	27.736	31.750	40.002	50.400	63.500
2013	70.300	19.344	24.372	30.706	35.150	44.286	55.797	70.300
2014	74.900	20.610	25.966	32.716	37.450	47.184	59.448	74.900
2015	63.500	17.473	22.014	27.736	31.750	40.002	50.400	63.500
2016	61.500	16.922	21.321	26.863	30.750	38.743	48.813	61.500
2017	76.200	20.967	26.417	33.283	38.100	48.003	60.480	76.200
2018	69.100	19.014	23.956	30.182	34.550	43.530	54.845	69.100
2019	76.200	20.967	26.417	33.283	38.100	48.003	60.480	76.200
2020	88.900	24.462	30.820	38.831	44.450	56.003	70.560	88.900
2021	121.900	33.542	42.260	53.245	60.950	76.792	96.752	121.900
2022	67.300	18.518	23.332	29.396	33.650	42.396	53.416	67.300
2023	101.600	27.956	35.223	44.378	50.800	64.004	80.640	101.600
Mean		21.70231	27.3432	34.45027	39.43571	49.68589	62.60029	78.87143
Standard Deviation		5.7745	7.275415	9.166448	10.49296	13.22031	16.65654	20.98593

Now after using Gumbel's method rainfall (in mm) and rainfall intensities (mm/hr) are calculated for return periods of 2, 5, 10, 25, 50, 75 and 100 years.

Table 6.3 Computed rainfall depth (in mm)

Duration (in Hr)	Return period (in years)						
	2	5	10	25	50	75	100
0.5	20.8488	27.0070	31.0843	36.2360	40.0578	42.2792	43.8514
1	26.2679	34.0267	39.1638	45.6545	50.4697	53.2684	55.2493
2	33.0954	42.8710	49.3433	57.5211	63.5878	67.1140	69.6097
3	37.8848	49.0751	56.4840	65.8452	72.7899	76.8264	79.6833
6	47.7319	61.8307	71.1654	82.9597	91.7095	96.7952	100.3946
12	60.1384	77.9018	89.6627	104.5227	115.5467	121.9543	126.4893
24	75.7696	98.1501	112.9680	131.6904	145.5797	153.6527	159.3665

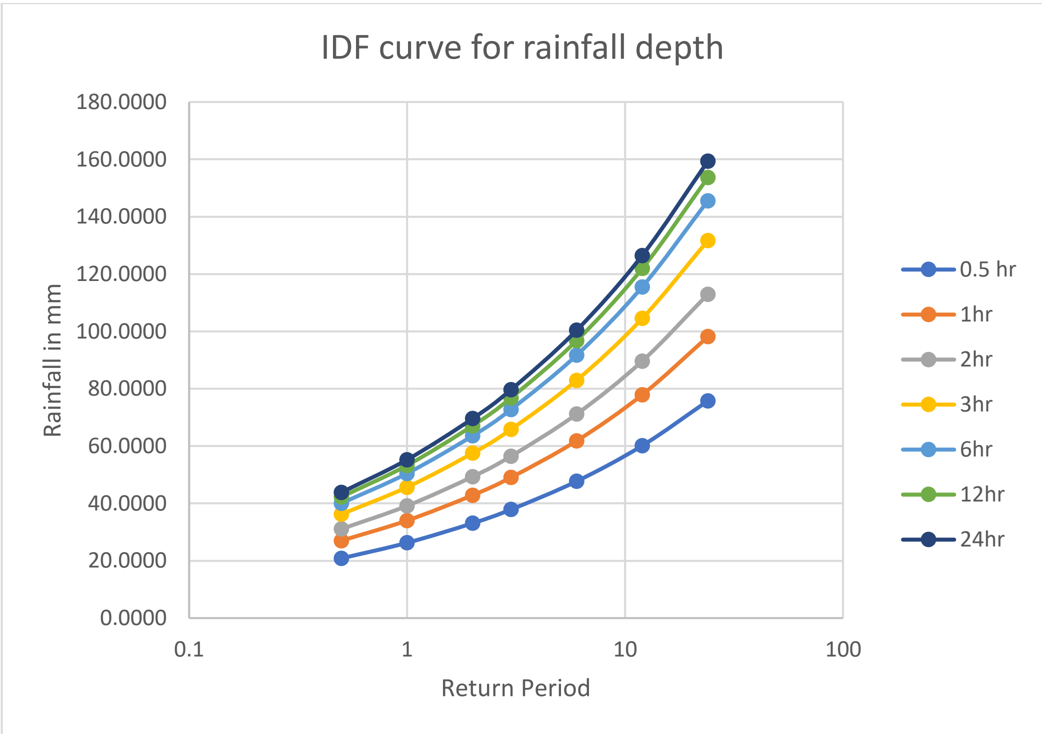


Figure 6.1 : IDF curve for rainfall depth

Table 6.4 Computed rainfall intensity (in mm/hr)							
Duration (in Hr)	Return period (in years)						
	2	5	10	25	50	75	100
0.5	41.6976	54.0141	62.1687	72.4720	80.1156	84.5584	87.7028
1	26.2679	34.0267	39.1638	45.6545	50.4697	53.2684	55.2493
2	16.5477	21.4355	24.6717	28.7605	31.7939	33.5570	34.8049
3	12.6283	16.3584	18.8280	21.9484	24.2633	25.6088	26.5611
6	7.9553	10.3051	11.8609	13.8266	15.2849	16.1325	16.7324
12	5.0115	6.4918	7.4719	8.7102	9.6289	10.1629	10.5408
24	3.1571	4.0896	4.7070	5.4871	6.0658	6.4022	6.6403

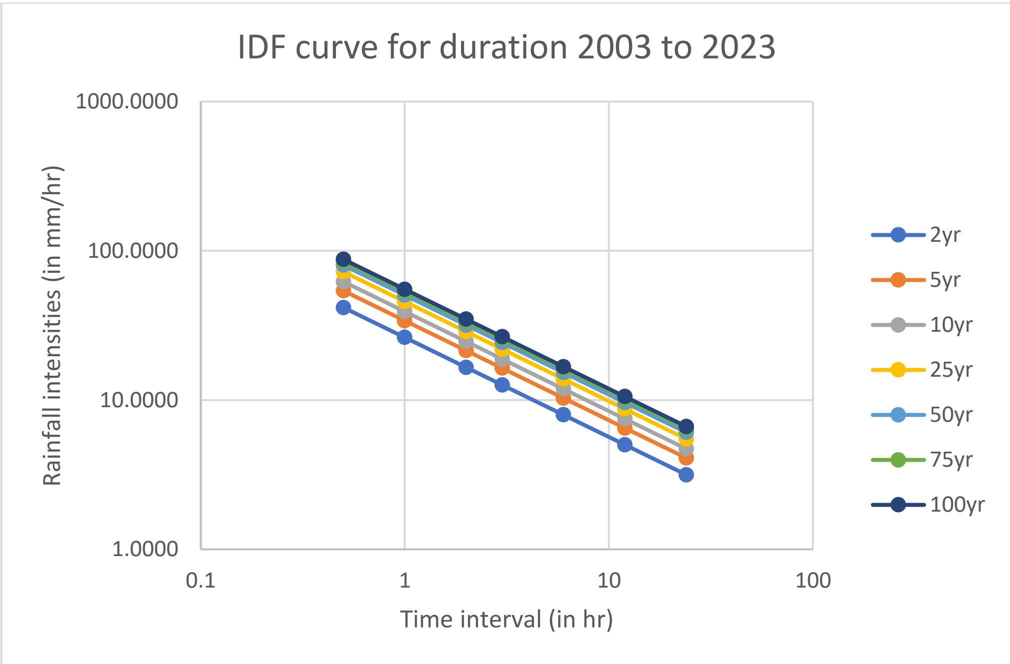


Figure 6.2 : IDF curve for rainfall intensity

Table 6.5: Gumbel's Distribution for Design Rainfall in Pokoria Basin

Order No	Maximum annual daily rainfall (X)	Tp (Years) (N+1)/m	X- ẍ	(X- ẍ)^2
1	124.50	22.00	45.63	2081.97
2	121.90	11.00	43.03	1851.46
3	111.80	7.33	32.93	1084.29
4	101.60	5.50	22.73	516.59
5	88.90	4.40	10.03	100.57
6	87.60	3.67	8.73	76.19
7	83.80	3.14	4.93	24.29
8	76.20	2.75	-2.67	7.14
9	76.20	2.44	-2.67	7.14
10	74.90	2.20	-3.97	15.77
11	73.70	2.00	-5.17	26.74
12	73.70	1.83	-5.17	26.74
13	70.30	1.69	-8.57	73.47
14	69.10	1.57	-9.77	95.48
15	67.30	1.47	-11.57	133.90
16	66.00	1.38	-12.87	165.67
17	63.50	1.29	-15.37	236.28
18	63.50	1.22	-15.37	236.28
19	61.50	1.16	-17.37	301.77
20	57.10	1.10	-21.77	474.00
21	43.20	1.05	-35.67	1272.45
Mean 'ẍ'	**78.87**		**∑(X- ẍ)^2 =**	**8808.18**

From Table 6.4 we get,

$$N = 21$$
$$\ddot{x}' = 78.87$$

$$\delta_{n-1} Standard\ deviation = \sqrt{\sum (X - \ddot{x})^2 / N - 1}$$

$$= 21.0$$

From the chart,

$$\bar{y}_n = 0.5252 \ , \qquad S_n = 1.0696$$

Table 6.6: Calculated Peak daily rainfall for the corresponding return period.

Return Period (in years)	Reduced Variate (Y_T)	Frequency factor (K)	Peak daily rainfall (in mm)
	$Y_T = -\left\{l_n . l_n \left(\frac{T}{T-1}\right)\right\}$	$K = \frac{Y_T - \bar{y}_n}{S_n}$	$X_n = \ddot{X}' + K\delta_{n-1}$
25	3.19853	2.50	131
50	3.90194	3.16	145
100	4.600149	3.81	159

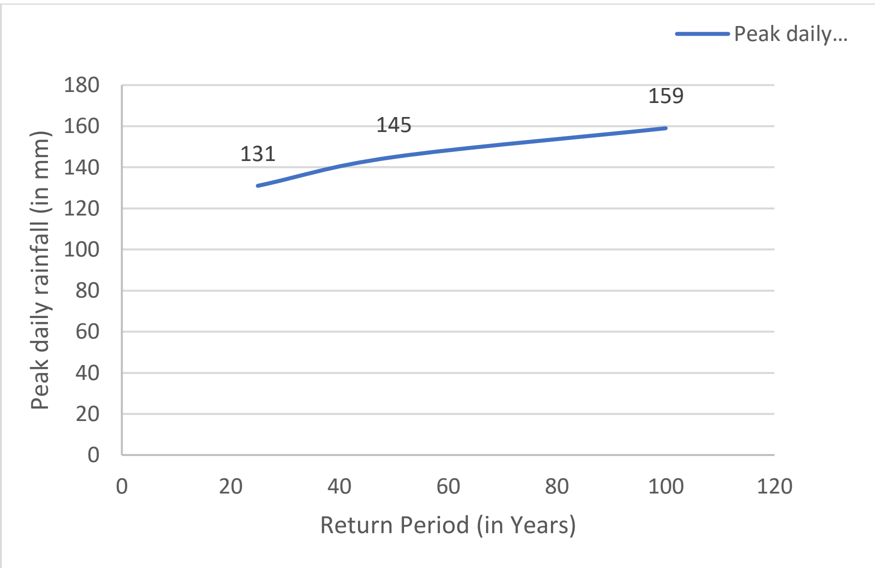


Figure 6.3: Graphical representation of Peak daily rainfall for the corresponding Return period.

6.2 Highest flood level (HFL)

The HFL of Brahmaputra River recorded by the Water Resource Department, Assam, at Ulubari station Morigaon district was collected from 2003 to 2023 which is tabulated below:

Table 6.7 : Recorded HFL of Brahmaputra River at Ulubari, (Morigaon Dist.)

Year	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**
HFL	58.66	59.25	59.03	57.9	59.23	58.03	57.43	59.28	58.67	59.39	58.76

Year	**2014**	**2015**	**2016**	**2017**	**2018**	**2019**	**2020**	**2021**	**2022**	**2023**
HFL	58.96	59.09	59.07	59.65	57.96	59.29	59.62	58.95	58.56	56.42

Table 6.8 : Gumbel's Distribution for HFL of Brahmaputra River at Ulubari

Order No	HFL (X)	Tp (Years) (N+1)/m	X- ẍ	(X- ẍ)^2
1	59.65	22.00	0.93	0.86
2	59.62	11.00	0.90	0.80
3	59.39	7.33	0.67	0.44
4	59.29	5.50	0.57	0.32
5	59.28	4.40	0.56	0.31
6	59.25	3.67	0.53	0.28
7	59.23	3.14	0.51	0.26
8	59.09	2.75	0.37	0.13
9	59.07	2.44	0.35	0.12
10	59.03	2.20	0.31	0.09
11	58.96	2.00	0.24	0.06
12	58.95	1.83	0.23	0.05
13	58.76	1.69	0.04	0.00
14	58.67	1.57	-0.05	0.00
15	58.66	1.47	-0.06	0.00
16	58.56	1.38	-0.16	0.03
17	58.03	1.29	-0.69	0.48
18	57.96	1.22	-0.76	0.58
19	57.90	1.16	-0.82	0.68
20	57.43	1.10	-1.29	1.67
21	56.42	1.05	-2.30	5.31
Mean 'ẍ'	**58.72**		**∑(X- ẍ)^2 =**	**12.48**

From table 6.8 we get,

$$N = 21$$

$$\ddot{X}' = 58.72$$

$$\delta_{n-1} Standard\ deviation = \sqrt{\sum (X - \ddot{x})^2 / N - 1}$$

$$= 0.8$$

From the chart,

$$\bar{y}_n = 0.5252\ , \qquad S_n = 1.0696$$

Table 6.9 : Calculated High Flood Level (HFL) for the corresponding return period.

Return Period (in Years)	Reduced Variate (Y_T)	Frequency factor (K)	HFL (in m)
	$Y_T = -\{l_n . l_n (\frac{T}{T-1})\}$	$K = \frac{Y_T - \bar{y}_n}{S_n}$	$X_n = \ddot{X}' + K\delta_{n-1}$
25	3.19853	2.50	60.698
50	3.90194	3.16	61.218
100	4.600149	3.81	61.734

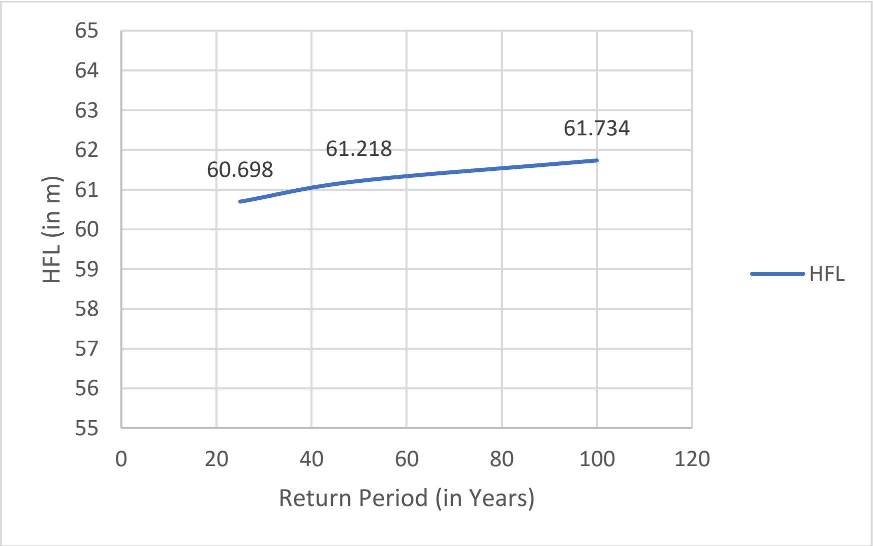


Figure 6.4: Graphical representation of HFL for the corresponding Return period.

6.3 Design Discharge

While the absence of high-resolution DEMs and precise boundary delineation of catchment areas poses challenges for rainfall modelling and flood management, employing a combination of alternative data sources, interpolation methods, field surveys, hydrological models, rational method, local knowledge, and partnerships can help mitigate these constraints. These strategies can enhance the accuracy of water body delineation and improve the effectiveness of flood management efforts.

The Rational Method is a widely used technique for estimating peak runoff rates, especially when high-resolution Digital Elevation Models (DEMs) are not available. This method is relatively simple and does not require detailed topographic data, making it suitable for situations where precise boundary delineation of water bodies is challenging.

The area-velocity method is used for estimating streamflow when high-resolution DEMs are not available because it relies on direct measurements of the physical characteristics of a stream or river, rather than on detailed topographic data. It is effective method for estimating streamflow in the absence of high-resolution DEMs because it relies on direct physical measurements rather than detailed topographic data. This method is practical, cost-effective, and can provide accurate real-time flow data, making it a valuable tool for hydrological studies and water resource management.

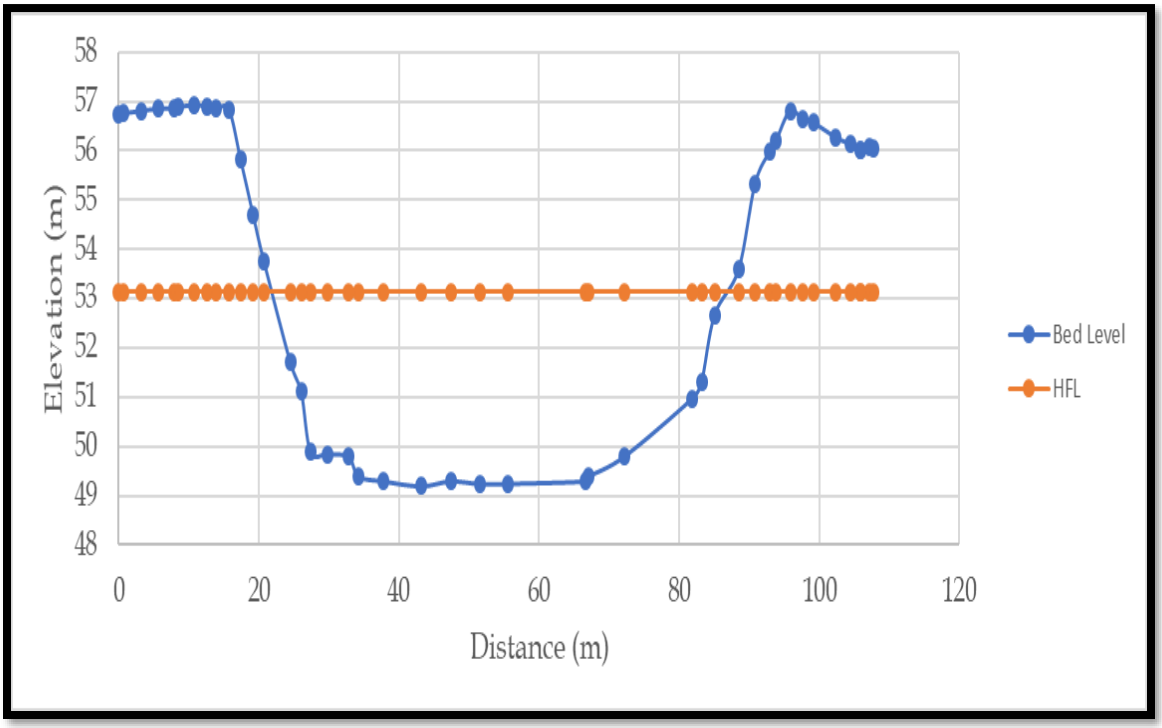
The river bathymetry in the study area was generated by the Water Resources Department, Assam through LiDAR survey and echosounder techniques. Figure 6.5 shows the high-resolution DEM generated using LiDAR and Eco-sounding method. The cross-sectional profile of the river at the bridge site (Figure 6.6) was derived from this bathymetric data. The High Flood Level (HFL) at the bridge location is recorded as 53.132 meters. Figure 6.7 illustrates the river cross-section alongside the high flood level.

According to the provided information, there is no available flow data for the river. Consequently, the discharge of the river at its highest flood level (HFL) is determined using the area velocity method. It is important to highlight that the IS code recommends the use of empirical methods in situations where discharge data is not readily accessible.

Figure 6.5 High resolution DEM of the area prepared using LiDAR and Eco-Sounding data

(Source Water Resource Department , Assam)

Figure 6.6 Bridge location



The Figure 6.7 Cross section at the bridge site

The flow cross-sectional area (A) up to the HFL is 232.28 m^2

The perimeter of the section (P) is 69.54 m

The hydraulic radius (R) is 3.34 m

The longitudinal slope of the channel is 1:4900

Manning's n of 0.025

$Q = \frac{1}{n} A R^{2/3} S^{1/2} = 296.61$ m^3/sec

The discharge corresponding to the recorded HFL is **296.61 m^3/sec**.

6.4 Geotechnical Survey

A geotechnical survey is essential for understanding the characteristics of the soil and subsurface conditions at a construction site. It helps in designing safe and cost-effective foundations, mitigating risks, ensuring regulatory compliance, and contributing to the longevity and performance of the structure. By identifying potential issues early, geotechnical surveys play a vital role in the successful execution of construction projects. The report of the geotechnical survey sourced from the Water Resources Department; Assam is enclosed in Appendices which summarised that a shallow foundation is not suitable below 4.0m depth below GL.

CHAPTER 7

Design of Sluice

The river sluice is designed for safe passage of high flood discharge from country side to river side i.e. from Pokoria Channel to river Brahmaputra and the design parameters will be checked for seepage from river side to country side at HFL of river Brahmaputra.

7.1 . SEEPAGE FROM COUNTRY SIDE TO RIVER SIDE

I . Design Data :

High flood level before construction (C/S	=	53.13	
Average bed level at sluice site	=	47.53	
Depth of water retained on C/S	=	3.00	m
Retained water level (C/S)	=	50.53	
Average GL at sluice site	=	50.90	
Average width of the river	=	65.00	m
Safe exit gradient	=	1/6	
Lacey's silt factor	=	0.78	
Concentration of flow	=	20%	
Retrogression of bed	=	0.50	m
Permissible afflux	=	0.50	m
Design discharge	=	300.00	cumec

II . Fixation of crest level and waterway :

As per Lacey's formula minimum stable waterway,

$$P = 4.83\sqrt{Q}$$

$$= 4.83\sqrt{300}$$

$$= 83.66 \text{ m}$$

Since the stream is confined with stable bank, the overall waterway is kept equal to the natural width of the river as per IS : 6966 (Part-I)-1989, Cl. 10.1

Let us assume the water-way as follows

i)	19	bays of sluice	3.00	m each	=	57.00	m
ii)	18	piers of	1.00	m each	=	18.00	m
		Total water way			=	75.00	m

Discharge Capacity

Average discharge intensity $q = \dfrac{300.00}{75.00}$ cumecs / m

$= 4.00$ cumecs / m

Depth of scour $R = 1.35\,(q^2/f)^{1/3}$

$= 3.70$ m

Velocity of approach $V = \dfrac{q}{R}$

$= \dfrac{4.00}{3.70}$

$= 1.08$ m / sec

Velocity head $h_a = \dfrac{V^2}{2g}$

$= \dfrac{1.08^2}{2 \times 9.81}$

$= 0.06$ m / sec

H.F.L. before construction of sluice = 53.13

Permissible afflux = 0.50 m

U/S H.F.L. after construction = H.F.L. before construction + afflux

= 53.13 + 0.50

= 53.63

U/S T.E.L.		=	U/S HFL + Velocity head
		=	53.63 + 0.06
		=	53.69

The upstream floor level of the sluice is kept at average bed level of the river.

Hence, U/S floor level of sluice		=	47.53
Crest level of sluice		=	Level of water retained on C/S
		=	50.53
Head over crest		=	U/S T.E.L - Crest level
		=	53.69 - 50.53
	H	=	3.16 m
Considering crest width	B	=	2.00 m

Now $\frac{H}{B} = \frac{3.16}{2.00}$

$= 1.58 > 1.50$

The weir acts as sharp crested weir

Discharge passing through sluice using discharge formula for sharp crested weir

	Q	=	1.84 (L - 0.1 n H) H3/2
Where	L	=	Length of clear water way
	n	=	No. of end contractions
		=	38
	H	=	Head over crest
Hence	Q	=	1.84 (L - 0.1 n H) H3/2
		=	1.84 (57 - 0.1 x 38 x 3.16) (3.16)^ 3/2
		=	465.03 cumec > 300.00 cumec

Hence,OK

Hence, the assumed water way and crest level are sufficient to pass the design discharge and therefore adopted.

III . Hydraulic design of Sluice :

(A) Discharge intensity and head loss under different flow conditions

i) For High Flood Condition

(a) With out flow concentration and with out retrogression

Discharge intensity between piers	=	$CH^{3/2}$
	=	$1.84 \times 3.16^{3/2}$
	=	10.34 cumec / m
D/S H.F.L	=	53.13
D/S T.E..L	=	D/S H.F.L. + velocity head
	=	53.13 + 0.06
	=	53.19
U/S H.F.L.	=	D/S H.F.L + afflux
	=	53.13 + 0.50
	=	53.63
U/S T.E..L	=	U/S H.F.L. + velocity head
	=	53.63 + 0.06
	=	53.69

Head loss H_L = U/S T.E.L. - D/S T.E.L.
= 53.69 - 53.19
= 0.50 m

(b) With 20% flow concentration and bed retrogression by 0.50 m

Discharge intensity = 1.20 x 10.34
= 12.41 cumec / m

Head required including velocity head for this discharge intensity

$$H = (q/c)^{2/3}$$

$$= \left(\frac{12.41}{1.84}\right)^{2/3}$$

= 3.57 m

U/S T.E.L. = Crest level + head over crest
= 50.53 + 3.57
= 54.10

D/S H.F.L with 0.50 m retrogression = 53.13 - 0.50
= 52.63

D/S T.E..L = D/S H.F.L + velocity head
= 52.63 + 0.06
= 52.69

Head loss H_L = U/S T.E.L. - D/S T.E.L.
= 54.10 - 52.69
= 1.41 m

(ii) Flow at Pond level condition

As there are no data of stage-Vs-discharge and the river is ungauged at the portion, the D/S water profiles could not be determined. So, calculation at pond level is given up. However, the jump profile to occur at much higher level at P.L. flow condition, the determination of D/S floor level and length of floor will not be affected by the condition.

Level, Length of floor & Hydraulic jump calculations :-

The level at which the hudraulic jump is formed is located and the level of down stream floor is fixed below the lowest level at which the hudraulic jump is formed.

Sl. No.	Item	Without concentration and retrogression	With 20% concentration and 0.50 m retrogression
1	Discharge intensity, q in cumec / sec	10.34	12.41
2	Down stream water level	53.13	52.63
3	Up stream water level	53.63	53.63
4	Upstream T.E.L.	53.69	54.10
5	Down stream T.E.L.	53.19	52.69
6	Head Loss H_L	0.50 m	1.41 m
7	Downstream specific energy level, E_{f2}	3.92 m	4.80 m
8	Upstream specific energy level, $E_{f1} = E_{f2} + H_L$	4.42 m	6.21 m
9	Level at which jump would form = D/S T.E.L .- E_{f2}	49.27	47.89
10	Pre-jump depth y_1 corresponding to E_{f1}	1.34 m	1.27 m
11	Post-jump depth y_2 corresponding to E_{f2}	3.42 m	4.38 m
12	Length of concrete floor reqd. beyond the jump $= 5 (y_2 - y_1)$	10.40 m	15.55 m
13	Froud No. F $\frac{q}{\sqrt{(g D_1^3)}}$	2.13	2.77

HF

The lowest level at which the hydraulic jump formed =			47.89		
Let the provide the downstream floor level at R.L.			47.53		
which is the level of U/S floor					
Horizontal length of D/S floor required		=	15.55	m	
Let us provide the length of down stream floor		=	18.50	m	

Depth of sheet pile line from scour consideration

U/S sheet pile line :-

Total discharge passing down the sluice		=	465.03	m^3 / sec	
Overall water-way		=	75.00	m	
Average discharge intensity	q	=	$\frac{465.03}{75.00}$	cumecs / m	
		=	6.20	cumecs / m	
Depth of scour	R	=	$1.35\ (q^2/f)^{1/3}$		
		=	4.95	m	

As per IS 6966 (part-1):1989, the U/S sheet pile should generally be provided to cater for scours up to 1R

Let us provide up stream sheet pile up to 1.25R for safety

Anticipated scour depth	=	1.25	R	
	=	1.25	x	4.95
	=	6.19	m	
R.L. of bottom of scour hole	=	U/S water level	-	Anticipated Scour depth
	=	53.63	-	6.19
	=	47.44		
Let us provide sheet pile line up to bottom level of		45.03		
Depth of sheet pile below U/S floor	=	47.53	-	45.03
	=	2.50	m	

However, the depth of upstream sheet pile line will be governed by the seepage condition from river side to country side.

D/S sheet pile line :-

As per IS 6966 (part-1):1989, the D/S sheet piles should generally be provided to cater for scours up to 1.25 R

Let us provide down stream sheet pile up to 1.50 R for safety

Anticipated scour depth	=	1.50	R	
	=	1.50	x	4.95
	=	7.43	m	
R.L. of bottom of scour hole	=	52.63	-	7.43
	=	45.20		
Let us provide sheet pile line up to bottom level of		44.03		
Depth of sheet pile below D/S floor	=	47.53	-	44.03
	=	3.50	m	

IV . Total floor length and exit gradient :

Bed level of D/S floor		=	47.53		
Maximum static head	H_s	=	Pond level	-	D/S bed level
		=	50.53	-	47.53
		=	3.00	m	
Depth of D/S sheet pile	d	=	3.50	m	
	G_E	=	1/6		

Now, $G_E = \dfrac{H_s}{d \times \pi\sqrt{\lambda})}$

i.e $\dfrac{1}{\pi\sqrt{\lambda}} = \dfrac{G_E\, d}{H_s}$

$= \dfrac{1}{6} \times \dfrac{3.50}{3.00}$

$= 0.19$

$\lambda = 2.81$

Hence $\alpha = \sqrt{\{(\ 2\lambda\text{-}1\ )^2 - 1\ \}}$

$= 4.51$

Total floor length required $= \alpha\, d$

$= 15.79$ m

Provide a total floor length $= 45.00$ m (considering the length reguired for D/S floor, crest & glacis)

Let us keep the floor length as below:

(1) D/S horizontal floor = 18.50 m

(2) D/S glacis length with 1:1 slope = 1 x (50.53 - 47.53)

= 3.00 m

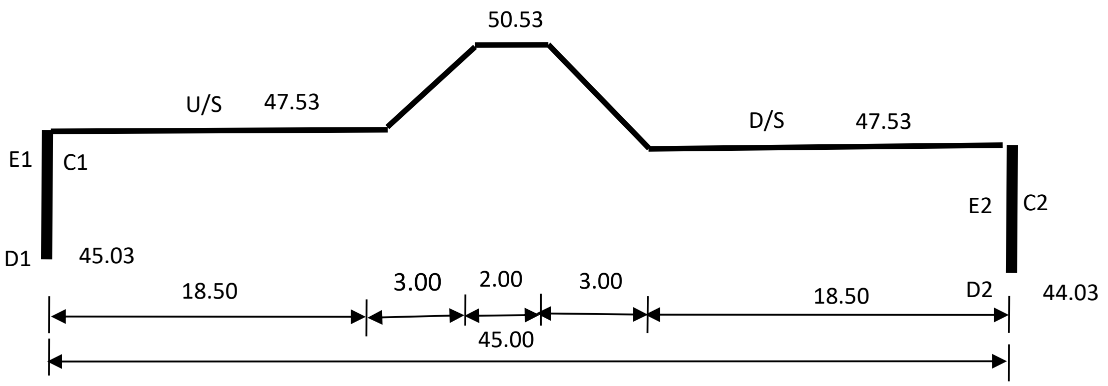
(3) Crest width = 2.00 m

(4) U/S glacis length with 1:1 slope = 1 x (50.53 - 47.53)

= 3.00 m

(5) U/S floor length = 18.50 m

Total length = 45.00 m



ARRANGEMENT OF FLOOR LENGTH

V . Pressure calculations :

For determining uplift pressures according to Khosla's theory, it is essential to assume the floor thickness at upstream and downstream sheet pile line.

Let us, assume that the thickness of U/S floor = 1.50 m

and thickness of D/S floor = 1.50 m

sheet pile thickness at both end = 0.50 m

(i) Upstream sheet pile line

$b = 45.00$ m

$d = 2.50$ m

$\alpha = \dfrac{b}{d}$

$= \dfrac{45.00}{2.50}$

$= 18.00$

$\lambda = \dfrac{1+\sqrt{(1+\alpha^2)}}{2}$

$= \dfrac{1 + \sqrt{(1 + 18.00\ )^2}}{2}$

$= 9.51$

Now $\varnothing_E$ = $100 / \pi \cos^{-1}(\lambda - 2) / \lambda$

$\varnothing_D$ = $100 / \pi \cos^{-1}(\lambda - 1) / \lambda$

$\varnothing_E$ = 21.02 %

$\varnothing_D$ = 14.73 %

$\varnothing_{C1}$ = $100 - \varnothing_E$

= 100 - 21.02

Hence, $\varnothing_{C1}$ = 78.98 %

$\varnothing_{D1}$ = $100 - \varnothing_D$

= 100 - 14.73

Hence, $\varnothing_{D1}$ = 85.27 %

Correction of pressure:

(a) Correction for floor thickness:

$\varnothing_{D1} - \varnothing_{C1}$ = 6.29 %

$$= \frac{(\varnothing_{D1} - \varnothing_{C1}) \times \text{ floor thickness}}{\text{Depth of upstream sheet pile}}$$

= 3.77 % (+ve)

(b) Correction for interference due to D/S sheet pile

Correction = $19 \times \sqrt{(D / b')} \times ((d' + D) / b))$

Where D = Depth of D/S sheet pile below point C1

= 46.03 - 44.03

= 2.00 m

b = Total length of impervious floor

= 45.00 m

b^1 = 44.00 m

d' = 2.50 - 1.50

= 1.00 m

Correction = 0.27 % (+) ve

Corrected $\varnothing_{C1}$ = 78.98 + 3.77 + 0.27

= 83.02 %

(ii) Down stream sheet pile line

b = 45.00 m

d = 3.50 m

$$\alpha = \frac{b}{d}$$

$$= \frac{45.00}{3.50}$$

= 12.86

$$\lambda = \frac{1+\sqrt{(1+\alpha^2)}}{2}$$

$$= \frac{1 + \sqrt{(1 \quad + \quad 12.86}}{2})^2$$

= 6.95

Now $\varnothing_E$ = $100 / \pi \cos^{-1}(\lambda - 2) / \lambda$

$\varnothing_D$ = $100 / \pi \cos^{-1}(\lambda - 1) / \lambda$

$\varnothing_{E2}$ = 24.77 %

$\varnothing_{D2}$ = 17.29 %

Correction of pressure:

(a) Correction for floor thickness:

Ø_{E2} - Ø_{D2} = 7.48 %

$$= \frac{(\text{Ø}_{E2} - \text{Ø}_{D2}) \text{ x floor thickness}}{\text{Depth of down stream sheet pile}}$$

= 3.21 % (-ve)

(b) Correction for interference due to U/S sheet pile

Correction = $19 \text{ x } \sqrt{(D / b')} \text{ x } ((d + D) / b))$

Where D = Depth of U/S sheet pile below point C2

= 46.03 - 45.03

= 1.00 m

b = Total length of impervious floor

= 45.00 m

b^1 = 44.00 m

d' = 3.50 - 2.00

= 1.50 m

Correction = 0.16 % (-) ve

Corrected Ø_{E2} = 24.77 - 3.21 - 0.16

= 21.40 %

(iii) The level of hydraulic gradient lines can be calculated between key points assuming linear variation for various flow condition are tabulated below:

Condition of flow	U/S water level	D/S water level	Seepage head (m)	Height / elevation of subsoil H.G. line above D/S water level and its elevation					
				U/S sheet pile line			D/S sheet pile line		
				Ø_{E1}	Ø_{D1}	Ø_{C1}	Ø_{E2}	Ø_{D2}	Ø_{C2}
				100.00	85.27	83.02	21.40	17.29	0.00
No flow condition (Max. static head)	53.63	47.53 (No water) in canal	6.10	6.10 53.63	5.20 52.73	5.06 52.59	1.31 48.84	1.05 48.58	0.00 47.53
High flood flow condition	53.63	52.63	1.00	1.00 53.63	0.85 53.48	0.83 53.46	0.21 52.84	0.17 52.80	0.00 52.63

VI . Floor thickness :

(a) Downstream Floor :-

(i) Thickness required at toe of glacis

% age of pressure = $21.40 \; .+ \left(\frac{83.02 - 21.40}{45.00} \right) \text{ x } 18.50$

= 46.73 %

Maximum unbalanced head = % of pressure x Maximum static head

= 46.73 % x 6.10

= 2.85 m

Thickness required = $\frac{2.85}{2.24 - 1}$

= 2.30 m

Provide 2.55 m in 6.00 m length from the toe of the glacis.

(ii) Thickness required at 6.00 m beyond toe

% age of pressure = $21.40 + \left(\frac{83.02 - 21.40}{45.00} \right) \times 12.50$

= 38.52 %

Maximum unbalanced head = % of pressure x maximum static head

= 38.52 % x 6.10

= 2.35 m

Thickness required = $\frac{2.35}{2.24 - 1}$

= 1.90 m

Provide 2.10 m in 6.00 m length from 6.00 m to 12.00 m from toe of glacis.

(iii) Thickness required at 12.00 m beyond toe

% age of pressure = $21.40 + \left(\frac{83.02 - 21.40}{45.00} \right) \times 6.50$

= 30.30 %

Maximum unbalanced head = % of pressure x maximum static head

= 30.30 % x 6.10

= 1.85 m

Thickness required = $\frac{1.85}{2.24 - 1}$

= 1.49 m

Provide 1.65 m in 6.50 m length from 12.00 m to 18.50 m from toe of glacis.

(b) Up stream Floor

Upstream floor thickness will be provided as per seepage condition from river side to country side.

VII . Protection works beyond imperious floor :-

(i) U/S protection :-

Length of U/S block protection and launching apron will be provided as per seepage condition from river side to country side.

(ii) D/S protection :-

Scour depth R = 4.95 m

Anticipated scour = 2 R

= 2 x 4.95

= 9.90 m

Depth of downstream scour level = D/S water level - anticipated scour

= 52.63 - 9.90

= 42.73

Scour depth D below D/S bed = 47.53 - 42.73

= 4.80 m

(a) Block protection :-

As per IS : 6966 (Part-I), the length of downstream block protection shall be approximately equal to 1.5 D, the

Length of concrete blocks = 1.5 x D

= 7.20 m

Providing 1.20 m x 1.20 x 1.00 m C.C. blocks with 10 cm gap in between filled with bajri over an inverted filter of thickness 0.50 m

Number of concrete blocks required = $\frac{7.20}{1.20 + 0.10}$

= 5.54 m

Provide 6 rows of 1.20 m x 1.20 x 1.00 m C.C. blocks with 10 cm gap in between filled with bajri over an inverted filter of thickness 0.50 m

Length of inverted filter = 6 x 1.20 + 7 x 0.10

= 7.90 m

(b) Launching apron :-

Thickness of launching apron = 1.50 m

Volume of launching apron = 2.25 D m^3 / m

Length required = $\dfrac{2.25 \times 4.80}{1.50}$

= 7.20 m

Provide 1.50 m thick launching apron for a length of 8.50 m.

(iii) Toe wall : A concrete toe wall extending up to about 500 mm below the bottom of filter shall be provided at the end of the inverted filter to prevent it from getting disturbed.

VIII . DESIGN OF ENERGY DISSIPATOR DEVICES (As per Indian Standard Stilling Basin-1)

The froude number both for barrage and spill-way portion are almost same. So, design is done for barrage portion and the same is applied for spill-way portion also. The design is done as per Fig. 2.16 (a) & (b), Page : 59-60, Stilling Basin Design for Froude Number below 4.5. "Theory and Design of Irrigation Structures" Vol. -

Froude Number = 2.77

A Basin Blocks

Discharge intensity	q	=	12.41	cumec / m
Head loss	H_L	=	1.41	m
Pre jump depth	d_1	=	1.27	m
Conjugate depth	d_2	=	4.38	m
Tail water depth	d_3	=	5.10	m

Critical depth $dc = \left(\dfrac{q^2}{g}\right)^{1/3}$

= 2.5 m

For $\dfrac{d_2}{d_c} = \dfrac{4.38}{2.5}$

= 1.75

$\dfrac{h_B}{d_c}$ = 0.40

h_B = 0.40 x 2.5

= 1.00 m

Adopt a value of h_B = 1.00 m

Again for Froude number = 2.77

$\dfrac{W_B}{h_B}$ = 0.75

W_B = 0.75 x 1.00

= 0.75 m

Adopt a value of W_B = 0.75 m for the width of floor blocks

Top width of basin blocks = 0.02 h_B

= 0.02 x 1 m

= 0.02 m

Spacing = h_B

= 1.00 m

Spacing at ends = 0.5 h_B

= 0.5 x 1 m

= 0.50 m

Again $\frac{L}{h_3} = \frac{18.50}{5.10}$

$= 3.63$

$\frac{L_B}{h_3} = 0.90$

$L_B = 0.90 \times 5.10$

$= 4.59$ m

Provide two rows of basin blocks at a distance of 5.00m from toe of glacis.

7.2 . SEEPAGE FROM RIVER SIDE TO COUNTRY SIDE

HFL on river side = 53.58

Crest level of sluice = 50.53

Bed level of U/S floor (R/S) = 47.53

(D/S bed level for seepage condition from C/S to R/S)

Bed level of D/S floor (C/S) = 47.53

(U/S bed level for seepage condition from C/S to R/S)

Maximum static head H_s = HFL (R/S) - D/S bed level (C/S)

= 53.58 - 47.53

= 6.05 m

Depth of D/S sheet pile line d = 2.50 m

(Same as US sheet pile depth for seepage from C/S to R/S)

$G_E = 1/6$

Now, $G_E = \frac{H_s}{d \times \pi\sqrt{\lambda})}$

i.e $\frac{1}{\pi\sqrt{\lambda}} = \frac{G_E\, d}{H_s}$

$= \frac{1}{6} \times \frac{2.50}{6.05}$

$= 0.07$

$\lambda = 20.70$

Hence $\alpha = \sqrt{\{(2\lambda\text{-}1)^2 - 1\}}$

$= 40.39$

Total floor length required = α d

= 100.98 m Which is excessive

Let us adopt the depth of d/s sheet pile, d = 5.40 m

$G_E = \frac{H}{d\,(\pi\sqrt{\lambda})}$

i.e $1/(\pi\sqrt{\lambda}) = \frac{G_E\, d}{H}$

$= 1/6 \times \frac{5.40}{6.05}$

$= 0.15$

$\lambda = 4.56$

$\alpha = \sqrt{[\{(2\lambda - 1)^2 - 1)\}]}$

$= 8.06$

Total floor length required = α d

= 8.06 x 5.4

= 43.52 m

Provide a total floor length = 45.00 m

(Same as for seepage condition from C/S to R/S)

R.L. of bottom of U/S sheet pile = Bed level of U/S floor - 3.50
= 47.53 - 3.50
= 44.03

R.L. of bottom of D/S sheet pile = Bed level of D/S floor - 5.40
= 47.53 - 5.40
= 42.13

Hence, the floor length for seepage condtion from R/S to C/S as below:

(1) D/S horizontal floor = 18.50 m

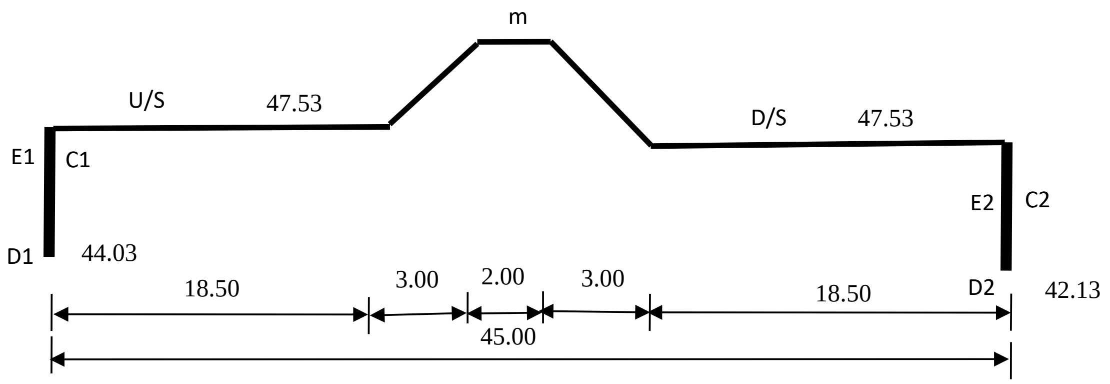
(2) D/S glacis length with 1:1 slope = 1 x (50.53 - 47.53)
= 3.00 m

(3) Crest width = 2.00 m

(4) U/S glacis length with 1:1 slope = 1 x (50.53 - 47.53)
= 3.00 m

(5) U/S floor length = 18.50 m

Total length = 45.00 m



ARRANGEMENT OF FLOOR LENGTH

V . Pressure calculations :

For determining uplift pressures according to Khosla's theory, it is essential to assume the floor thickness at upstream and downstream sheet pile line.

Let us, assume that the thickness of U/S floor = 1.70 m
and thickness of D/S floor = 2.00 m
sheet pile thickness at both end = 0.50 m

(i) Upstream sheet pile line

b = 45.00 m
d = 3.50 m
(Same as DS sheet pile depth for seepage from C/S to R/S)

$$\alpha = \frac{b}{d} = \frac{45.00}{3.50} = 12.86$$

$$\lambda = \frac{1+\sqrt{(1+\alpha^2)}}{2} = \frac{1 + \sqrt{(1 + 12.86)^2}}{2} = 6.95$$

Now $Ø_E$ = $100 / \pi \cos^{-1}(\lambda - 2) / \lambda$
$Ø_D$ = $100 / \pi \cos^{-1}(\lambda - 1) / \lambda$
$Ø_E$ = 24.77 %
$Ø_D$ = 17.29 %
$Ø_{C1}$ = $100 - Ø_E$
= 100 - 24.77

Hence, $Ø_{C1}$ = 75.23 %

$Ø_{D1}$ = 100 - $Ø_D$

= 100 - 17.29

Hence, $Ø_{D1}$ = 82.71 %

Correction of pressure:

(a) Correction for floor thickness:

$Ø_{D1}$ - $Ø_{C1}$ = 7.48 %

$$= \frac{(Ø_{D1} - Ø_{C1}) \times \text{floor thickness}}{\text{Depth of upstream sheet pile}}$$

= 3.63 % (+ve)

(b) Correction for interference due to D/S sheet pile

Correction = 19 x √ (D / b') x ((d' + D) /b))

Where D = Depth of D/S sheet pile below point C1

= 45.83 - 42.13

= 3.70 m

b = Total length of impervious floor

= 45.00 m

b^1 = 44.00 m

d' = 3.50 - 1.70

= 1.80 m

Correction = 0.67 % (+) ve

Corrected $Ø_{C1}$ = 75.23 + 3.63 + 0.67

= 79.53 %

(ii) Down stream sheet pile line

b = 45.00 m

d = 5.40 m

$$\alpha = \frac{b}{d}$$

$$= \frac{45.00}{5.40}$$

= 8.33

$$\lambda = \frac{1+\sqrt{(1+\alpha^2)}}{2}$$

$$= \frac{1 + \sqrt{(1 + 8.33}}{2})^2$$

= 4.69

Now $Ø_E$ = 100 /π $\cos^{-1}$(λ - 2) / λ

$Ø_D$ = 100 / π $\cos^{-1}$(λ - 1) / λ

$Ø_{E2}$ = 30.54 %

$Ø_{D2}$ = 21.16 %

Correction of pressure:

(a) Correction for floor thickness:

$Ø_{E2}$ - $Ø_{D2}$ = 9.38 %

$$= \frac{(Ø_{E2} - Ø_{D2}) \times \text{floor thickness}}{\text{Depth of down stream sheet pile}}$$

= 3.47 % (-ve)

(b) Correction for interference due to U/S sheet pile

Correction = 19 x √ (D / b') x ((d + D) / b))

Where D = Depth of U/S sheet pile below point C2

= 45.53 - 44.03

= 1.50 m

b = Total length of impervious floor

= 45.00 m

	b^1	=	44.00	m		
	d'	=	5.40	-	3.70	
		=	1.70	m		
Correction		=	0.25	% (-) ve		
Corrected	$Ø_{E2}$	=	30.54	-	3.47	- 0.25
		=	26.82	%		

(iii) The level of hydraulic gradient lines can be calculated between key points assuming linear variation for various flow condition are tabulated below:

Condition of flow	U/S water level	D/S water level	Seepage head (m)	Height / elevation of subsoil H.G. line above D/S water level and its elevation					
				U/S sheet pile line			D/S sheet pile line		
				$Ø_{E1}$	**$Ø_{D1}$**	**$Ø_{C1}$**	**$Ø_{E2}$**	**$Ø_{D2}$**	**$Ø_{C2}$**
				100.00	82.71	79.53	26.82	21.16	0.00
No flow condition (Max. static head)	53.58	47.53 (No water)	6.05	6.05	5.00	4.81	1.62	1.28	0.00
				53.58	52.53	21.93	18.74	18.40	17.12

VI . Floor thickness :

(a) Downstream Floor :-

(i) Thickness required at toe of glacis

% age of pressure $= 26.82 \ .+ \left(\dfrac{79.53 - 26.82}{45.00} \right) \times 18.50$

$= 48.49$ %

Maximum unbalanced head = % of pressure x Maximum static head

$= 48.49 \ \% \times 6.05$

$= 2.93$ m

Thickness required $= \dfrac{2.93}{2.24 - 1}$

$= 2.36$ m

Provide 2.60 m in 6.00 m length from the toe of the glacis.

(ii) Thickness required at 6.00 m beyond toe

% age of pressure $= 26.82 \ .+ \left(\dfrac{79.53 - 26.82}{45.00} \right) \times 12.50$

$= 41.46$ %

Maximum unbalanced head = % of pressure x maximum static head

$= 41.46 \ \% \times 6.05$

$= 2.51$ m

Thickness required $= \dfrac{2.51}{2.24 - 1}$

$= 2.02$ m

Provide 2.20 m in 6.00 m length from 6.00 m to 12.00 m from toe of glacis.

(iii) Thickness required at 12.00 m beyond toe

% age of pressure = $26.82 + \left(\frac{79.53 - 26.82}{45.00}\right) \times 6.50$

= 34.43 %

Maximum unbalanced head = % of pressure x maximum static head

= 34.43 % x 6.05

= 2.08 m

Thickness required = $\frac{2.08}{2.24 - 1}$

= 1.68 m

Provide 1.90 m in 6.50 m length from 12.00 m to 18.50 m from toe of glacis.

(b) Up stream Floor

Upstream floor thickness is provided as per downstream floor thickness for seepage condition from country side to river side.

VII . Protection works beyond imperious floor :-

(i) U/S protection :-

Length of U/S block protection and launching apron is provided as per D/S block protection and launching apron for seepage condition from river side to country side.

(ii) D/S protection :-

Scour depth R = 4.95 m

(Same as seepage condition from country side to river side)

Anticipated scour = 2 R

= 2 x 4.95

= 9.90 m

Depth of downstream scour level = D/S water level - anticipated scour

= 53.13 - 9.90

= 43.23

Scour depth D below D/S bed = 47.53 - 43.23

= 4.30 m

(a) Block protection :-

As per IS : 6966 (Part-I), the length of downstream block protection shall be approximately equal to 1.5 D, the design depth of scour below the floor level.

Length of concrete blocks = 1.5 x D

= 6.45 m

Providing 1.20 m x 1.20 x 1.00 m C.C. blocks with 10 cm gap in between filled with bajri over an inverted filter of thickness 0.50 m

Number of concrete blocks required = $\frac{6.45}{1.20 + 0.10}$

= 4.96 m

Provide 5 rows of 1.20 m x 1.20 x 1.00 m C.C. blocks with 10 cm gap in between filled with bajri over an inverted filter of thickness 0.50 m

Length of inverted filter = 5 x 1.20 + 6 x 0.10

= 6.60 m

(b) Launching apron :-

Thickness of launching apron = 1.50 m

Volume of launching apron = 2.25 D m^3/m

Length required = $\frac{2.25 \times 4.30}{1.50}$

= 6.45 m

Provide 1.50 m thick launching apron for a length of 7.50 m.

(iii) Toe wall : A concrete toe wall extending up to about 500 mm below the bottom of filter shall be provided at the end of the inverted filter to prevent it from getting disturbed.

CHAPTER 8
RESULTS

The design and implementation of sluices in the Pokoria River aim to manage water flow, mitigate flooding, and optimize irrigation. This chapter presents the results from hydrological, bathymetric, and hydraulic analyses, followed by a discussion of the findings in relation to the objectives of the sluice design.

8.1 Hydrological Analysis

8.1.1 Rainfall Data Analysis

Rainfall data over 21 years (2002-2023) was collected and analyzed to understand the hydrological dynamics of the Pokoria Basin. The data was obtained from Ulubari rain gauge station, Morigaon as provided by Morigaon Water Resource Division.

Annual Rainfall: The maximum annual rainfall in the Pokoria Basin was 124.50 mm, with significant inter-annual variability.
Seasonal Variation: The majority of the rainfall occurs during the July month (May to October), accounting for about 44% of the annual total.
Extreme Events: Several extreme rainfall events were recorded, with daily rainfall exceeding 100 mm on some occasions.

8.1.2 Gumbel Method

8.1.2.1 Peak Daily Rainfall

To estimate the peak daily rainfall, the Gumbel distribution was applied to the 21 years of rainfall data. The Gumbel distribution is widely used for extreme value analysis in hydrology.
Predicted Peak Daily Rainfall for Different Return Periods:

- **25-Year Return Period**: 131 mm
- **50-Year Return Period**: 145 mm
- **100-Year Return Period**: 159 mm

8.1.2.2 Highest Flood Level

To highest flood level(HFL) was predicted , the Gumbel distribution was applied to the 21 years of HFL recorded of Brahmaputra river.

Predicted HFL of Brahmaputra river for Different Return Periods:

- **25-Year Return Period**: 60.098 m
- **50-Year Return Period**: 61.218 m
- **100-Year Return Period:** 61.734m

8.2 Bathymetric Analysis

The river bathymetry in the study area was generated by the Water Resources Department, Assam through LiDAR survey and echosounder techniques. The cross-sectional profile of the river at the bridge site was derived from this bathymetric data. The High Flood Level (HFL) at the bridge location is recorded as 53.132 meters. The surveys covered the sections of the river where the sluices are to be installed, extending 2 kilometres upstream and downstream. The bathymetric data revealed significant variations in the riverbed's depth and morphology. The river depth ranged from 1.5 meters in shallow areas to 8 meters in deeper pools. Using bathymetric data, in Manning's equation the peak discharge is obtained corresponding to HFL is 296.61 m^3/sec of Pokoria River is obtained by Area Velocity Method.

8.3 Sluice Gate Design Summary

The hydraulic part of sluice gates was designed to accommodate the highest expected water levels and flow rates. To design the hydraulic sluice for the Pokoria River, MS Excel platform cell referencing is used. Using Khosla's theory and the Hydraulic Jump theory, it determines the hydraulic parameters of a sluice that are determined taking into account surface flow, subsurface flow, and the composition of the foundation soil. Regression from Khosla's pressure curves is used to solve the uplifting pressure head distribution on the structure, enabling the nearly flawless design of structures with and without concentration and retrogression consideration. A useful tool for making decisions regarding the hydraulic design of tiny barrages is this cell referencing.

According to the plan, the detailed drawing is shown in Appendix 3 and 4. The waterway is 75 meters long, with 19 bays measuring 3.0 meters each and 18 piers measuring one meter each. Each gate height is 3.2m and width is 3.2m.

The calculated discharge that passes through the waterway is 465,03.00 cumec, which is more than the 300 cumec assumed design discharge. The layout of the 45.00 m total floor length is shown below.

Table 8.1: Distribution of total floor length.

Floor Details	Length (m)
D/s horizontal floor	18.50
D/s glaciers in a slope of 1:1	3.0
Crest width	2.0
U/s glaciers at a slope of 1:1	3.0
U/s floor	18.50

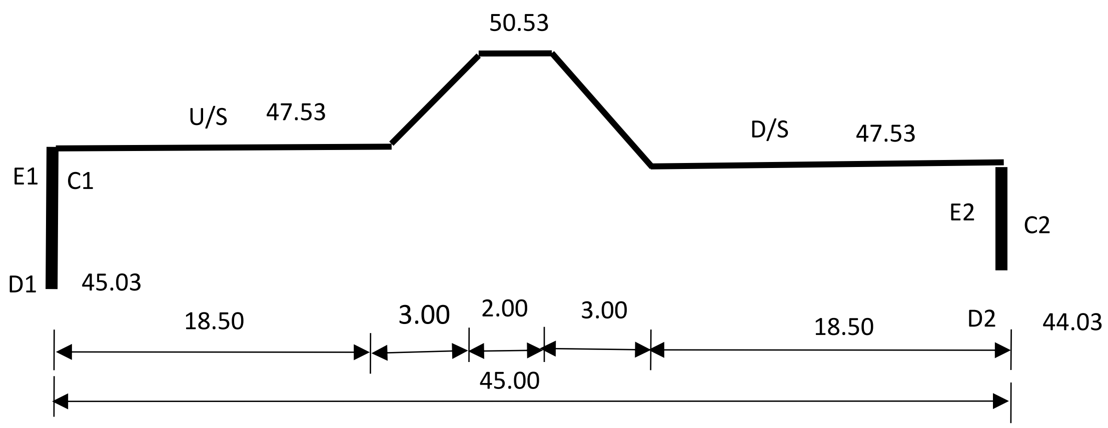


Figure 8.1 ARRANGEMENT OF FLOOR

This sluice design is special because the u/s and d/s glaciers are maintained at the same slope ratio of 1:1, allowing it to function as a two-way passage. For example, when the Brahmaputra water level is high during the monsoon, the gate may be closed to prevent floodwaters from inundating the area, and when the Pokoria river's water level is lowered, the area's water needs can be satisfied by allowing the Brahmaputra water to flow through the Pokoria river. The results indicated that the sluice gates could withstand all tested conditions without significant deformation or risk of failure.

8.4 Environmental and Social Impacts

The sluice design incorporated measures to minimize environmental impact, such as: Reinforcing riverbanks around the sluices to prevent erosion, designing sluice gates to manage sediment flow, and preventing downstream sedimentation issues, Reducing the frequency, severity of flooding in nearby communities and enhancing water availability for agriculture, supporting local farming activities.

CHAPTER 9
CONCLUSION AND RECOMMENDATION

The design and implementation of sluices in the Pokoria River represent a significant step forward in regional water management, aimed at controlling floods, optimizing irrigation, and benefiting the local community. Through comprehensive hydrological, bathymetric, and structural analyses, this study has demonstrated the effectiveness and durability of the sluice design under various environmental conditions.

9.1 Conclusion

The 21-year rainfall data analysis highlighted significant variability, with peak daily rainfall predicted using the Gumbel method. This information was crucial in estimating peak discharges and designing sluice gates to manage extreme conditions. Due to challenges in obtaining high-resolution Digital Elevation Model (DEM) data for accurate catchment area delineation, bathymetric surveys were conducted to map the riverbed topography. This approach identified areas of sediment deposition and erosion, which were addressed in the sluice design to ensure efficient operation and longevity. The Rational Method estimated a peak discharge of approximately 296.61 m^3/s, guiding the structural specifications of the sluice gates. The design incorporated measures to minimise environmental impact, such as fish passages and erosion control, while providing social and economic benefits like flood mitigation and improved irrigation. The design calculations and analysis were structured in MS Excel, utilizing cell referencing for efficient data management and scenario analysis.

9.2 Recommendation

This study focused on designing a sluice using cell referencing in Excel that would solve the surface and subsurface flow problems for the diversion structure. Little or no effort is made to include the structural design of gates, piers and some other component structures that need special structural design considerations in the study. To maintain and enhance the effectiveness of the sluice design in the Pokoria River, several key recommendations are proposed like Monitoring and Maintenance of the sluices, including water flow, sediment deposition, and structural integrity, inspect structural components, and ensuring the hydraulic systems are functioning correctly. Engage local communities through education and participation programs to foster cooperation and ownership of the project. Conduct ongoing environmental impact studies to identify and mitigate any unforeseen effects on the river ecosystem.

9.3 Future Research

- The potential impacts of climate change can be Investigated on rainfall patterns and river flow to adapt the sluice design to future conditions.
- Ways can be explored to optimize the performance of the sluices, including the use of new materials and design modifications. The integration of MATLAB and HEC-RAS for design calculations enabled detailed hydrological and hydraulic modelling. By employing these platforms, different parameters could be easily adjusted, and their impacts on the overall design could be quickly evaluated. This approach facilitated the iterative process of optimizing the sluice design and ensured that all calculations were transparent and reproducible. The utilization of MATLAB and HEC-RAS allowed the design team to streamline the workflow, minimize errors, and enhance the overall efficiency of the design process. This methodology can serve as a valuable tool for future projects, enabling precise and adaptable engineering solutions.
- If High-resolution Digital Elevation Model (DEM) data for accurate catchment area delineation is obtained then SWAT can be used for runoff modelling which would be realistic.

9.4 Final Thoughts

The sluice design for the Pokoria River is a testament to the importance of integrated water management strategies that balance structural, environmental, and social considerations. By implementing the recommendations outlined in this chapter, the project can ensure long-term success and provide a model for similar initiatives in other regions. Continued collaboration among engineers, environmental scientists, policymakers, and the local community will be essential to sustaining the benefits of this vital infrastructure. The use of bathymetric data, due to the challenges in obtaining high-resolution DEM for catchment area delineation, highlights the adaptability and resourcefulness necessary in complex water management projects.

References

11_-_theories_of_seepage_and_design_of_weirs_and_barrages. (1954.).

Ahmad, I. (n.d.). *THE CHINIOT DAM-SHEET PILE DESIGN ASPECT*.

Al-Amri, N. S., & Subyani, A. M. (2017). Generation of rainfall intensity duration frequency (IDF) curves for ungauged sites in arid region. *Earth Systems and Environment*, *1*(1). https://doi.org/10.1007/s41748-017-0008-8

"AQUIFER MAPPING AND MANAGEMENT. (2022.).

Azamathulla, H. M., Ahmad, Z., & Ab. Ghani, A. (2013). Computation Of Discharge Through Side Sluice Gate Using Gene-Expression Programming. *Irrigation and Drainage*, *62*(1), 115–119. https://doi.org/10.1002/ird.1698

Benoit, N., Motto Frederic, B., Jean, N., Banack, E., & Joseph, H. (n.d.). *38 Research and Reviews: Journal of Engineering and Technology Discharge Structure, Flood Discharge Sluice Gate and Floating Debris Sluice Design for the Mekin Hydropower Project*.

Bhange, H. N., & Deshmukh, V. V. (2018). A RS and GIS approaches for the estimation of runoff and soil erosion in SA-13 watershed. *Hydrologic Modeling*, 307-330. https://doi.org/10.1007/978-981-10-5801-1_22

Bora, M., & Chandra Bora, M. (2010). *Republic of Macedonia-25* (Vol. 29). https://www.researchgate.net/publication/264878734

Bureau of Soil Survey, N., & Use Planning NBSS, L. (n.d.). *SOILS SURVEY REPORT OF MORIGAON DISTRICT (ASSAM)*.

Chaurasia, S. R. (n.d.). *Direct Equations for Hydraulic Jump Elements in Rectangular Horizontal Channel*. https://doi.org/10.1061/ASCE0733-94372003129:4291

Chin, D. A. & Asce, F. (2019). Estimating Peak Runoff Rates Using the Rational Method. Journal of Irrigation and Drainage Engineering, 145(6), 1–8. https://doi.org/10.1061/(asce)ir.1943-4774.0001387

Chow, V. T., and D. R. Maidment. "Mays. LW (1988) Applied Hydrology." International edition. Macgraw-Hill Inc (1988)

E. M. AN Khosla, NK Bose, Design of weirs on pervious foundations. Publication number 12 of the Central Board of Irrigation, Simla, India, 1936.

Faisal, I. M., Kabir, M. R., & Nishat, A. (n.d.). *Non-structural ¯ood mitigation measures for Dhaka City*. www.elsevier.com/locate/urbwat

FLOOD MITIGATION BY DAMS IN KRISHNA BASIN OF MAHARASHTRA STATE. (n.d.).

Gupta, L. & Dixit, J. (2022). Estimation of rainfall-induced surface runoff for the Assam region, India, using the GIS-based NRCS-CN method. Journal of Maps, May, 1–13. https://doi.org/10.1080/17445647.2022.2076624
IDF (intensity-duration-Frequency) curves. (2010). *Hydrology*, 157 https://doi.org/10.1201/b10426-37

Kundwa, M. J. (2019). Development of rainfall intensity duration frequency (IDF) curves for hydraulic design aspect. *Journal of Ecology & Natural Resources*, *3*(2). https://doi.org/10.23880/jenr-16000162

Liu, Y., Yang, S. Q., Jiang, C., Sivakumar, M., Enever, K., Long, Y., Deng, B., Khalil, U., & Yin, L. (2019). Flood mitigation using an innovative flood control scheme in a large lake: Dongting Lake, China. *Applied Sciences (Switzerland)*, *9*(12). https://doi.org/10.3390/app9122465

Ljubenkov, I. (2015). Multicriteria flood mitigation in the Imotsko-Bekijsko Polje (Croatia, Bosnia and Herzegovina). *Journal of Water and Land Development*, *26*(1), 73–81. https://doi.org/10.1515/jwld-2015-0018

Nath, M. J., & Medhi, H. (2021). River Bank Line Shift Caused by Brahmaputra in Morigaon District, Assam (1996-2021). In *International Journal of Lakes and Rivers* (Vol. 14, Issue 2). http://www.ripublication.com/ijlr.htm

Nquot, I., & Kulatunga, U. (2014). Flood Mitigation Measures in the United Kingdom. *Procedia Economics and Finance*, *18*, 81–87. https://doi.org/10.1016/s2212-5671(14)00916-2

Pal, I., Singh, S., & Walia, M. A. (2013). Flood Management in Assam, INDIA: A review of Brahmaputra Floods, 2012. *International Journal of Scientific and Research Publications*, *3*(10). www.ijsrp.org

POLITEKNIK SULTAN SALAHUDDIN ABDUL AZIZ SHAH A STUDY OF SLUICE GATE IN MITIGATE FLOOD IN TAMAN SRI MUDA NUR NADHIRAH BINTI MAT HUSAIMI (08DKA20F1008). (n.d.).

Rahman, N. A., Jamaludin, M. S., Tan, L. W., Ali, Z. M., & Pinilla, C. E. (2022). Flood Mitigation Approaches: Selected Cases Across Europe, Oceania and Asia. *International Journal of Sustainable Construction Engineering and Technology*, *13*(4), 415–431. https://doi.org/10.30880/ijscet.2022.13.04.035

Rimi, S. S., and M. A. Matin. "Analysis of rainfall data for generation of Intensity-Duration-Frequency relationships for selected urban Cities of Bangladesh." Proceedings of 3rd International Conference on Advances in Civil Engineering (2016): 21-23.

Rasool, M. (2021). EFFECT OF MUTUAL INTERFERENCE PILES ON SEEPAGE PROPERTIES UNDER HYDRAULIC STRUCTURES. *Kufa Journal of Engineering*, *9*(4), 273–285. https://doi.org/10.30572/2018/kje/090419

Serra-Llobet, A., Jähnig, S. C., Geist, J., Kondolf, G. M., Damm, C., Scholz, M., Lund, J., Opperman, J. J., Yarnell, S. M., Pawley, A., Shader, E., Cain, J., Zingraff-Hamed, A., Grantham, T. E., Eisenstein, W., & Schmitt, R. (2022). Restoring Rivers and Floodplains for Habitat and Flood Risk Reduction: Experiences in Multi-Benefit Floodplain Management From California and Germany. *Frontiers in Environmental Science*, *9*. https://doi.org/10.3389/fenvs.2021.778568

Sharma, S., Lohani, A. K., Arun, G., Sharma, S. K., Tirkey, G., & Lohani, A. K. (2020). *Proceeding of National Conference on Emerging Trends in Civil Engineering Integrated flood assessment studies using real time satellite precipitation data and flood inundation mapping in Assam-A novel approach*. https://www.researchgate.net/publication/342850754

Sholihah, Q., Kuncoro, W., Wahyuni, S., Puni Suwandi, S., & Dwi Feditasari, E. (2020). The analysis of the causes of flood disasters and their impacts in the perspective of environmental law. *IOP Conference Series: Earth and Environmental Science*, *437*(1). https://doi.org/10.1088/1755-1315/437/1/012056

Smithers, J. C., & Schulze, R. E. (2002). *DESIGN RAINFALL AND FLOOD ESTIMATION IN SOUTH AFRICA*.

Sunu, P. W., Bangse, K., Rasta, I. M., T Putra, I. D. G. A., Anakottapary, D. S., & Santosa, I. D. M. C. (2020). Model development of flood control system for sluice. *Journal of Physics: Conference Series*, *1450*(1). https://doi.org/10.1088/1742-6596/1450/1/012085

Varshney, R. S., Gupta, S. C., Gupta, R. L. (1979). Theory and Design of Irrigation structures. India: Nem Chand & Bros..

Wangsu, M., Isaac, R., & Tirkey, N. (2021). Development of rainfall-intensity-Duration-Frequency (IDF) curves for Ranikhola watershed, Gangtok, India. *Journal of Soil and Water Conservation*, *20*(4), 447-454. https://doi.org/10.5958/2455-7145.2021.00056.4

Wasson, R., Saikia, A., Bansal, P., & Joon, C. C. (2020). Flood Mitigation, Climate Change Adaptation and Technological Lock-In in Assam. *Ecology, Economy and Society–the INSEE Journal*, *3*(2). https://doi.org/10.37773/ees.v3i2.150

Wikipedia contributors. (2024, April 30). Flood. In Wikipedia, The Free Encyclopedia. Retrieved 06:04, June 20, 2024, from https://en.wikipedia.org/w/index.php?title=Flood&oldid=1221508308. (n.d.).

Yasmin, M. N., Mohd Razali, S. F., Sharil, S., Wan Mohtar, W. H. M., & Saadon, K. A. (2022). Effectiveness of tidal control gates in flood-prone areas during high tide appearances. *Frontiers in Environmental Science*, *10*. https://doi.org/10.3389/fenvs.2022.919704

Yoosefdoost, A., & Lubitz, W. D. (2022). Sluice Gate Design and Calibration: Simplified Models to Distinguish Flow Conditions and Estimate Discharge Coefficient and Flow Rate. *Water (Switzerland)*, *14*(8). https://doi.org/10.3390/w14081215

भारत सरकार जल शिक्त मं त्रालय जल सं साधन, नदी िवकास और गं गा सं रक्षण िवभाग के द्रीय जल आयोग राट्रीय जल अकादमी, प ु णे के द्रीय जल अिभयां ित्रकी से वा के नव िनय ु क्त अिधकािरयय का इककीसवां प्रवे शन प्रिशक्षण कायर् क्रम िडजाइन और अन ु सं धानModule IV: Design of Weirs, Barrages and Canals. (2019).