ASSESSMENT OF DESIGN PRACTICES AND PERFORMANCE EVALUATION OF RIVER DIVERSION STRUCTURES FOR IRRIGATION: A CASE STUDY OF RASSA AND WODESA SMALL SCALE IRRIGATION SCHEMES IN WONDO GENET WOREDA, SIDAMA REGION, ETHIOPIA.

M.SC. THESIS

HENOK NIGUSSIE ALEMU

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

OCTOBER, 2024

ASSESSMENT OF DESIGN PRACTICES AND PERFORMANCE EVALUATION OF RIVER DIVERSION STRUCTURES FOR IRRIGATION: A CASE STUDY OF RASSA AND WODESA SMALL SCALE IRRIGATION SCHEMES IN WONDO GENET WOREDA, SIDAMA REGION, ETHIOPIA.

HENOK NIGUSSIE ALEMU

A THESIS SUBMITTED TO HAWASSA UNIVERSITY

DEPARTMENT OF HYDROULIC ENGINEERING, FACULTY OF BIO-SYSTEMS AND WATER RESOURCES ENGINEERING, INSTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUDIES HAWASSA UNIVERSITY,

HAWASSA, ETHIOPIA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE

HYDROULIC ENGINEERING

OCTOBER, 2024

HAWASSA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

THESIS EXAMINERS' APPROVAL SHEET

This is to certify that the thesis entitled "*Assessment of Design Practices and Performance Evaluation of River Diversion Structures for Irrigation: A Case Study of Rassa, and Wodesa Irrigation Schemes in Wondo Genet Woreda, Sidama Region, Ethiopia"*, submitted in partial fulfillment of the requirements for the degree of Masters with specialization in Hydraulic Engineering, the graduate program of the Department of Hydraulic Engineering, and has been carried out by Henok Nigussie ID. No GPHydrR/0004/15, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Dr. Tewodros Melese	____________________	____________________
Name of Main Advisor	Signature	Date
____________________	____________________	____________________
Name of Internal Examiner	Signature	Date
____________________	____________________	____________________
Name of External Examiner	Signature	Date
____________________	____________________	____________________
Name of department head	Signature	Date
____________________	____________________	____________________
Name of factuality dean	Signature	Date

Acknowledgement

First of all, I would like to thank the “Almighty God” for giving me the life, patience, audacity, wisdom and who made it possible, to begin and finish this work successfully.

I would like to express my deep heartfelt gratitude and appreciation to my advisor Dr. Tewodros Melese, who helped me a lot from initial to final level. From the beginning, his valuable guidance and interesting discussion to bring solutions for problems, advice, and constructive comments made me able to develop an understanding of the subject. Without his continuous follow-up, correcting the manuscripts and constructive comments the proposal work may not take the current forms.

My special and great thanks to my family that have been providing me with their love, support and encouragement at any cost time and place during my academic careers specially my mother Almaz Tadesse, my darling brothers, George Nigussie who sponsored my research work, my sister, Netsanet Nigussie and all relatives for being spectacular and loving parents to me. If I possess any good qualities, it is all because of you.

Finally, my gratitude goes to all my special lectures in Hydraulic engineering streams for their sharing knowledge and experience and Hawassa Institute of Technology for allow me part of this master’s program.

Declaration

I hereby declare that this Master of Science thesis, titled "Assessment of Design Practices and Performance Evaluation of River Diversion Structures for Irrigation: A Case Study of Rassa and Wodesa Irrigation Schemes in Wondo Genet Woreda, Sidama Region, Ethiopia," is my original work. It has not been submitted for a degree at any other university, and all sources of material used have been duly acknowledged.

Name: Henok Nigussie Alemu

Signature:____________________________

Data: _______________________________

Acronyms and Abbreviations

a.m.s.l	Above Mean Sea Level
DEM	Digital Elevation Model
DHWS	Diversion Head Work Structure
D/S	Downstream
FAO	Food and Agriculture Organization
FS	Factor of Safety
GPS	Geographic Positioning System
Ha	Hectare
HFC	High flood condition
HFL	High flood level
HH	Household
HJ	Hydraulic Jump
IFAD	International Fund for Agricultural Development
IWMI	International Water Management Institute
JHC	Jump height curve
LSIS	Large scale Irrigation scheme
MSIS	Medium Scale Irrigation Scheme
NFC	Normal flood condition
RC	Rating Curve
RF	Rainfall
SSIP	Small scale irrigation projects
Tc	Time of Concentration
TWD	Tail water depth
TWRC	Tail water rating curve
WGWA	Wondo Genet Woreda Administration
WGWANRO	Wondo Genet Woreda Agricultural and Natural Resource Office
WVE	World Vision Ethiopia
^0C	Degree Centigrade
U/S	Upstream
USSCS	United State Soil Conservation Service

Table of contents

Acknowledgement i

Declaration ii

Acronyms and Abbreviations iii

Lists of Tables vii

List of Figures ix

Abstract x

1. INTRODUCTION 1

1.1. Background of the study 1

1.2. Statement of the problem 3

1.3. Objectives of the study 4

1.3.1 General objectives 4

1.3.2 Specific objectives 4

1.4. Research questions 4

1.5. Scope of the study 4

1.6. Significance of the study 5

1.7 Limitation of the Study 5

2 LITERATURE REVIEW 6

2.1. Headwork 6

2.2. Design practice of weir 9

2.2.1 Hydrological analysis 9

2.2.2 Design rainfall frequency analysis 18

2.2.3 Hydraulic analysis 19

2.2.4 Structural analysis 22

2.3. Causes of weir failures 22

2.3.1 Causes of subsurface flow 22

2.3.2 Causes of surface flow 23

2.3.3 Failure due to silt 23

2.4. Concept of performance 24
2.5. Performance standard indicators..25
2.5.1 Hydraulic performance indicators..25
2.5.2 Structural performance indicators 26
3 MATERIALS AND METHOD 28
3.1. Description of the study area 28
3.1.1 Location 28
3.1.2 Population 28
3.1.3 Climate 29
3.1.4 Land use land cover 29
3.1.5 Soils...30
3.2. General description of the schemes 31
3.2.1 Rassa small scale irrigation scheme..31
3.2.2 Wodesa small scale irrigation scheme 32
3.3. Sources and procedure of data collection 33
3.3.1 Primary data and their sources 33
3.3.2 Secondary data, sources and quality assurance..34
3.4. Materials 34
3.5. Methods of data analysis 35
3.5.1 Hydrological analysis..35
3.5.2 Rainfall data quality test 35
3.5.3 Maximum rainfall frequency analysis..39
3.5.4 Stream flow analysis 39
3.5.5 Hydrologic models 41
3.5.6 Hydraulic analysis...42
3.5.7 Seepage analysis 43
3.5.8 Structural analysis 45
3.6. Study design...47

4 RESULTS AND DISCUSSIONS 48
4.1. Results from field visit and observations 48
4.2. Actual dimensions of headwork structures results 48
4.3. Hydrological analysis 49
4.3.1 Analysis of meteorological station 49
4.3.2 Rainfall data frequency analysis results 50
4.3.3 Model sensitivity analysis 53
4.3.4 HEC-HMS model simulation result 54
4.3.5 Design storm analysis result of HEC-HMS model 56
4.4 Hydraulic analysis 60
4.4.1 Adequacy of water way 60
4.4.2 Head of water over the crest 61
4.4.3 Hydraulic jump computation 63
4.4.4. Evaluation of constructed weir components 67
4.4.5 Seepage analysis 74
4.5 Structural analysis 77
4.5.1 Stability analysis of weir body 77
4.5.2 Stability analysis of retaining wall structure 79
5 CONCLUSION AND RECOMMENDATION 83
5.1. Conclusion 83
5.2 Recommendations 84
REFERENCES 86
APPENDIXES 90
Appendix (A) Hydrological data analysis 90
Appendix (B) Hydraulic analysis 98
Appendix (C) Structural analysis 104
Appendix (D) Seepage pressure analysis 114

Lists of Tables

Table 2.1 Descriptions, applications, and limitations of hydrologic models 16

Table 2.2 Comparison of hydrological models 17

Table 3.1 List of rainfall gauge for study 35

Table 3.4 Silt factor of river material 43

Table 3.5 The safe exit gradient based on soil type. 44

Table 4.1 Current status of the two diversion headwork structures 48

Table 4.2 Actual dimensions of headwork structures 49

Table 4.3 Areal coverage of rainfall station from thiessen polygon 49

Table 4.4 Data quality test for average areal rainfall of Wondogenet and Hawassa station ..50

Table 4.5 Determination of maximum and minimum rainfall out liar values........................51

Table 4.6 Summer of design average areal rainfall analysis result. 51

Table 4.7 Fifty (50) Years expected rainfall result 52

Table 4.8 Goodness of fit statistic test results 53

Table 4.9 Parameters sensitivity rank based on objective function. 54

Table 4.10 Optimized parameter values for Rasa watershed input to HEC-HMS model 57

Table 4.11 Optimized parameter values for Wodesa watershed input to HEC-HMS model.57

Table 4.12 Input for design storm simulation of Rasa and Wodesa river 59

Table 4.13 Summary of design storm simulation results for Rasa river 59

Table 4.14 Summary of design storm simulation results for Wodesa river 60

Table 4.15 Classification of jump based on Froude number 64

Table 4.16 Required pile and provided pile for Rasa and Wodesa diversion headwork........69

Table 4.17 Required and provided length of apron for diversion headwork 71

Table 4.18 Required and provided top width for diversion weir with no tension criteria 72

Table 4.19 Required and provided top width for diversion weir with no sliding criteria 72

Table 4.20 Required and provided bottom width for diversion headwork 73

Table 4.21 Required and provided capacity of under sluice 73

Table 4.22 Input data for Rasa and Wodesa weir seepage analysis 74

Table 4.23 Table Summary of corrected pressures at various key points 75

Table 4.24 Under weir pressure of Rasa diversion headwork by Khosla's theory 75

Table 4.25 Under weir pressure of Wodesa diversion headwork Khosla's theory 76

Table 4.26 Calculated uplift pressure of Rasa and Wodesa diversion structures...................77

Table 4.27 Summary of structural analysis for Rasa weir body 78

Table 4.28 Summary of structural analysis for Wodesa weir body 78

Table 4.29 Summary of structure analysis of retaining wall under high flood condition......79

Table 4.30 Summary of structure analysis of retaining wall under normal flow condition...80

Table 4.31 Structural physical performance level of the DHW structures 81

Table 4.32 Comparison of Rasa and Wodesa diversion headworks 82

List of Figures

Figure 2.1 Typical layout of diversion head work. 7

Figure 3.1 Location map of the study area....28

Figure 3.2 Monthly temperature and rainfall variation of Wondogenet 29

Figure 3.3 Wondogenet woreda land use land cover map 2020 30

Figure 3.4 Types of soil in Wondogenet woreda 31

Figure 3.5 Rasa watershed drainage pattern....32

Figure 3.6 Wodesa watershed drainage pattern....33

Figure 3.7 Thiessen polygon for nearest rainfall station 35

Figure 3.8 Average monthly rainfall of two station 36

Figure 3.9 Double mass curves of Hawassa and Wondogenet rainfall stations....37

Figure 3.10 Mean monthly flow hydrograph of Tikur Wuha river 40

Figure 3.11 Double mass curve of Tikur wuha river 41

Figure 4.2 Fitting test of distribution method....52

Figure 4.4 Simulated and observed stream flow after model calibration....54

Figure 4.5 Simulated and observed stream flow after validation....55

Figure 4.6 Daily streamflow model performance statistics result for calibration 55

Figure 4.7 Daily streamflow model performance statistics result for validation 55

Figure 4.8 Layout of HEC-HMS for Rasa watershed 58

Figure 4.9 Layout of HEC-HMS for Wodesa watershed 58

Figure 4.11 Effect of underperformance design of Rasa and Wodesa under sluice 74

Figure 4.12 Uplift pressure distribution of Rasa (a) and Wodesa (b) diversion structures....76

Abstract

Diversion headworks play a vital role in diverting water from rivers or channels for irrigation. However, many such structures face hydrological, hydraulic, structural, operational, and management challenges, leading to reduced functionality and high maintenance costs. Some recently constructed diversion structures have been reported to work properly while others have what could be termed as partial failures; parts of the structures get damaged as time passes. This study focuses on assessing the design practices and performance evaluation of the Rasa and Wodesa diversion headwork structures, which were constructed for irrigation purposes. The research evaluates the current conditions of these structures, analyzes the design practices, and identifies key factors contributing to their underperformance. Primary data, including field observations and measurements, were collected, along with secondary data from sources such as the National Meteorology Agency, the Ministry of Water and Energy, and the Sidama Regional State Irrigation Development Agency. The tools used include GPS, GIS 10.3.1, Excel, DEM, Bentley Flow Master, and HEC-HMS. The hydrological analysis, conducted using the HEC-HMS model, was calibrated and validated with observed streamflow data from the Tukur Wuha River for the periods 1991–2000 and 2001–2007, respectively. The model achieved good performance, with Nash-Sutcliffe Efficiency (NSE) values of 0.671 and 0.672, R^2 values of 0.79 and 0.8, Percent Bias (PBIAS) of 7.26% and 7.41%, and RMSE (Stdev) of 0.6 for both periods. The optimized parameters from the calibrated model were transferred to the ungauged Rasa and Wodesa watersheds using the regionalization technique. Design storm analysis of the model presented the extreme flows of 59.2m^3/s for Rasa and 98.2m^3/s for Wodesa which are beyond the weir design capacity of 44.2m^3/s and 84m^3/s respectively which results in flooding. Poor flood passage capacity and deposition of sediment were also recorded, which hindered efficiency. More specifically, structural evaluations indicated that floor thicknesses of concrete were low, cutoffs inadequate, and weir dimensions too small to provide adequate protection against uplift pressures and piping. Despite these problems, the stability for both weirs in terms of sliding, overturning, overstressing, and contact pressures were determined to be safe. The retaining walls were also safe from a structural perspective although contact pressures were higher than the recommended safe limits and therefore required further assessment and strengthening. This paper points out key aspects that would enable a small-scale diversion weir to function with greater durability and efficiency in irrigation systems.

Keywords: Diversion headwork, Design storm, Performance Evaluation, Rasa, Wodesa

1. INTRODUCTION

1.1. Background of the study

Ethiopia is dependent on rain-fed agricultural production for its livelihood. However, estimated crop production is not sufficient to meet the food requirements of the country. One of the best alternatives for achieving reliable and sustainable food security is expanding irrigation development on various scales, whether small, medium, or large, and utilizing options such as diversion, storage, gravity, and pumped irrigation (Lambisso, 2005).

Small-scale irrigation structures, due to their relatively low investment costs, ease of construction, and simplicity of operation and maintenance, have been a strategic target for Ethiopia in achieving sustainable food security and self-sufficiency. Numerous such schemes have been designed and constructed. While some schemes are performing successfully, various reports have observed that some schemes have failed to serve their intended purpose (Afera, 2004).

In Ethiopia, many irrigation projects have been under construction, and numerous designs have been made for future expansions to ensure food security for the growing population. However, the country has experienced many cases of failure in diversion headworks and under capacity issues for many decades, yet there is no information on the causes of these failures (Awulachew, 2010).

Understanding these problems is crucial to enhancing the effectiveness of irrigation schemes. It is essential to comprehend the various issues that arise at different levels of irrigation scheme development. These challenges compromise the effectiveness of irrigation schemes in generating income for farmers and improving livelihoods (Yami, 2016). As advanced irrigation is relatively new to the country, traditional water management and operation still dominate small-scale production in Ethiopia (Hora, 2016).

Ethiopia faces four key challenges technical, socio-economic, institutional, and environmental that must be overcome to meet its ambitious targets: behind-schedule scheme delivery, low performance of schemes, constraints on scaling up irrigation projects, and protecting the sustainability of irrigation development (Awulachew, 2010).

In addition, it has been observed that some irrigation schemes have failed to serve their intended purposes. Studies have shown that in Ethiopia, most diversion schemes are not operating according to the operational guidelines of their designs (Hailu, 2005).

Performance evaluation is a major component of proper irrigation water management. The most common problem in performance evaluation of irrigation schemes is the lack or unreliability of data. Many believe that irrigation schemes with modern systems will automatically have high performance, but the use of modern field application methods alone does not guarantee high performance. The efficiency of the irrigation system depends as much, if not more, on the capability of the irrigator as on the quality of the system (MoWR, 2002).

Recognizing both the benefits and hazards, assessment and evaluation of irrigation schemes' performance has now become of paramount importance. Not only does it identify where the problems lie, but it also helps to find alternatives that may be both effective and feasible in improving system performance (Temesgen, 2017).

According to the Wondo Genet Woreda Water, Mine, and Energy Office report from 2000-2020 G.C., eight schemes were developed by governmental and non-governmental organizations in the Woreda. Of these schemes, three (37.5%) of the diversion headworks are fully functional, four (50%) are partially functional, and one (12.5%) is damaged and requires serious maintenance. The sustainability and functionality of these irrigation schemes mainly depend on proper designing, implementation, monitoring and controlling, and operation and management of the river diversion headwork structures. However, the follow-up of these structures, especially concerning technical, social, operational and maintenance aspects, water utilization, and economic and financial success, has not been well studied and documented in the study area. Therefore, the proposed study aims to evaluate the performance of the diversion structures of the small-scale irrigation schemes, with an emphasis on the Rasa and Wodesa irrigation schemes, which were implemented in 2016 and 2019 G.C. by IFAD and governmental fund respectively.

In this research work, we primarily aim to evaluate the performance of two selected small-scale irrigation schemes in Wondo Genet Woreda, Sidama region, by identifying the constraints or problems that lead to underperformance. This evaluation will focus on the structural consistency of the diversion headworks and the hydraulic performance of the diversion headwork components based on hydraulic performance indicators.

1.2. Statement of the problem

Diversion headwork structures are critical for managing water resources and supporting small-scale irrigation systems. However, many of these structures in Ethiopia face challenges related to their design, construction, and management. Hydrologic, hydraulic, and structural design limitations, combined with operational inefficiencies, have contributed to reduced functional efficiency and increased maintenance costs. If such issues persist, the reliability of similar engineering solutions may be questioned, potentially undermining their role in addressing food security and improving livelihoods.

Particularly in the study area, various diversion headwork structures have been constructed in recent years. However, field observations and reports reveal that while some structures remain functional, others have encountered significant failures, requiring frequent maintenance. Key issues identified include sedimentation, downstream scouring, seepage, and damage to critical components, all of which hinder operational efficiency and reliability.

These findings highlight the urgent need to evaluate the performance of the Rasa and Wodesa diversion headwork structures and to identify the underlying causes of their underperformance. This research seeks to systematically assess these structures, analyze their design practices, and provide actionable recommendations to enhance their functionality and sustainability.

1.3. Objectives of the study

1.3.1 General objectives

The primary goal of this research is to assess the design practices and evaluate the performance of selected small-scale diversion weirs used for irrigation in Wondo Genet Woreda.

1.3.2 Specific objectives

- To evaluate the current condition of the diversion structures in the study area.
- To analyze the design practices of the selected diversion headworks, focusing on hydrologic, hydraulic, and structural aspects
- To identify the key factors contributing to the underperformance of diversion structures in relation to their design.

1.4. Research questions

The following questions are the main factors that will be dealt in this research work.

- What is the current status of the selected irrigation schemes in the study area?
- Is the failure of the headwork structures due to under estimation of design discharge, unconsidered hydraulic or structural conditions such as uplift, overturning, sliding or tension crack?
- What are the common causes contributing to the underperformance of the selected schemes?

1.5. Scope of the study

This study was conducted on two small-scale diversion weirs in Wondo Genet Woreda and aimed to assess design practices and performance by focusing on: problems concerning their design from hydrologic, hydraulic, and structural analysis perspectives; maintenance indicators; and issues hindering the development of small-scale irrigation schemes, including planning, institutional, operational, and maintenance aspects. This research work also provides remedial measures for the failed structures and offers recommendations to improve performance, a list of best practices from the two weirs for scaling up, and ways to fulfill new requirements. The study covered the main body of the weir, its components and surrounding area.

1.6. Significance of the study

The study provides information to the academicians, stakeholders, policymakers, project managers, and engineers to alleviate mistakes, understand drawbacks and best achievements across the system levels as well before and after the design and construction of small-scale irrigation schemes. This research work primarily helps stakeholders to build new structures or repair the existing ones; this will help to replace the existing ones or protection structures for the current, from minor repairs to complete upgrades or maintain the current functions or meet new requirements. In addition, it is used as a benchmark and entry point for irrigation development projects and future studies.

1.7 Limitation of the study

The primary limitations of this study were encountered during the collection of both primary and secondary data. Geological surveying beneath the weir foundation and determining the subsoil conditions of the riverbed presented challenges, making it difficult to comprehensively assess the entire weir site. Additionally, assessing the condition of the upstream impervious floor and cutoff piles was hindered by the substantial accumulation of granular material and backwater, obstructing detailed observation.

Another limitation was the lack of access to design reports for other small-scale irrigation projects in the region. Only the design documents for the Rasa and Wodesa schemes were available, limiting the ability to compare design procedures and outcomes across multiple schemes.

2 LITERATURE REVIEW

2.1. Headwork

Head works are any hydraulic structures which supplies water to the off taking canal. Head works may be classified as either diversion head works or storage headwork (Asawa, 2008). The works, which are constructed at the head of the canal, in order to divert the river water toward the canal, so as to ensure a regulated continuous supply of silt free water with a certain minimum head into the canal are known as Diversion Head works (Garg, 2005).

Storage head works are an obstruction constructed across the river valley to form the storage reservoir, besides fulfill all the requirements of diversion head works, store excess water when available and release it during period when demand exceeds supplies. The requirement for construction of diversion head works are to rise the water level at the head of the canal, to control the entry of silt into the canal and deposition of silt at the head of the canal and to control the fluctuation of water level in the river during different seasons (Geressu, 2023).

When water is diverted from rivers, depending upon the amount of mean stream flow and demand for the purpose, a dam like structure with or without gates or water inlet mechanism directly off taking from the river may be built across the river where diversion is required. The overall purpose is to confirm that all the flow can be diverted when water levels are low and to ensure the evacuation of sediment from the conveyance system (Afera, 2004).

In recent times, River diversion irrigation projects are highly favored in the country because of the availability of Perennial Rivers flow, suitability of topography, and low investment cost (Awulachew, 2010).

Diversion structures are desired to be safe against any hydraulic structural and geotechnical failures. Site selection of diversions needs to understand these technical considerations apart from design estimation (Baban, 1995).

The design of head work is project specific basically special attention is required on layout and sizing of head works from sediment handling point of view. Several existing head works are facing operational difficulties and loosing significant amount of revenue generation during operation phase due to short comings on head work design and layout (Shrestha, 2006).

The diversion weirs differ from barrages by the mode of raising the water level to the off taking canals. In case of diversion weirs water, the canal’s full supply level requirement is met by raising the height of the diversion structures itself whereas barrages utilize gates (Afera, 2004)

The function of a diversion head work is as follows (Fikru, 2014)

a. It raises the water level on its upstream side.
b. It regulates the supply of water in to the canal.
c. It controls the entry of silts in to the canal.
d. It creates temporary storage up stream of the weir

The weir body, river training work, under sluice, off-taking canal, fish ladder, upstream and downstream apron, upstream and downstream protection work and cut-off pile are all part of the diversion head work (Tadesse, 2019).

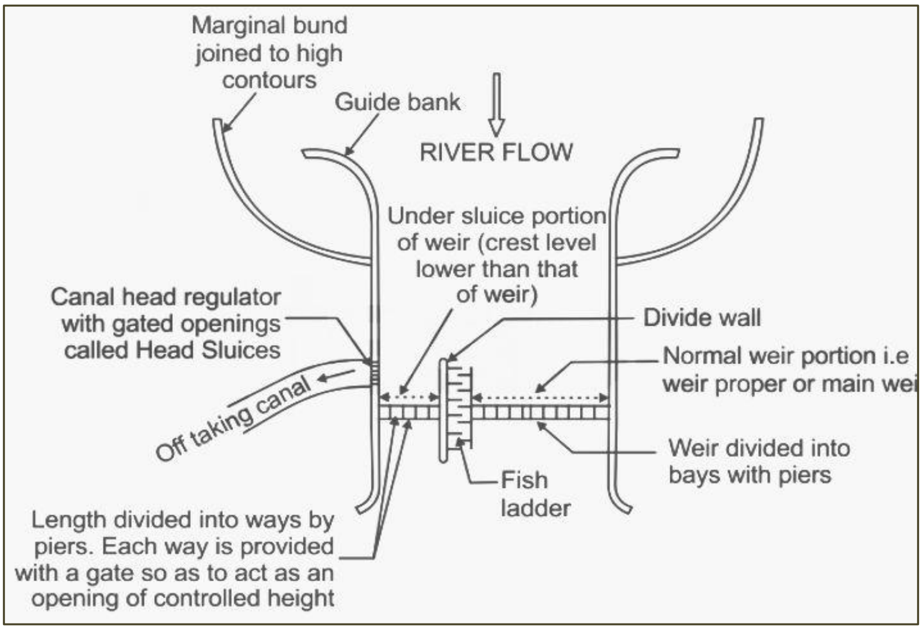


Figure 2.1 Typical layout of diversion head work (Garg, 2005).

Normally the water level of any perennial river is such that it cannot be diverted to the irrigation canal. The bed level of the canal may be higher than the existing water level of the river. In such cases weir is constructed across the river to raise the water level. Surplus water passes over the crest of the weir. Adjustable shutters are provided on the crest to raise the water level to some required height. When the water level on the upstream side of the weir is required to be raised to different levels at different time, barrage is constructed. Barrage is an arrangement of adjustable gates or shutters at different tiers over the weir (Tadesse, 2019).

River training works are required near the weir site in order to ensure a smooth and an axial flow of water and thus to prevent the river from outflanking the works due to a change in its course. The river training works required on a canal head-work are guide banks, marginal bunds, spurs or groynes (Fikru, 2014).

According to (Belango, 2021), the main causes of failure of under sluice components of the diversion headwork structures are the under-sized or improper design of the structures, which did not consider the design standards for under sluice structures. Concerning this, Garg (2005) has set the recommended that the discharging capacity of the under sluice section should be equal to or greater than the greatest of the following: (1) It should be able to ensure sufficient scouring capacity, for which the discharging capacity should be at least twice the full supply discharge of the off-taking canal; (2) It should be greater than the dry-weather flow of the river during winter so that the weir crest gates or shutters are not required to be opened; (3) It should be a substantial portion of the total design discharge so that small floods can be passed over the sluice section, without lifting the weir gates and (4) It should be able to dispose of 10-20% of the high flood discharge during severe floods.

The impervious floor is designed in all cases to reduce the surface flow action that causes scouring due to unbalanced pressure in the hydraulic jump trough. Generally speaking, except very few sites, end sills are not seen at the constructed structures. These could have played significant role in controlling receding jumps and hence reducing erosive power of the flowing water (Lambisso, 2005).

The thickness of aprons the upstream apron, sloping apron and the downstream apron are calculated taking the maximum unbalanced head between the subsurface uplift force and the surface flow for different case of flow, high anticipated flood flow; pond level flood flow and static water condition where the gates are closed with water at pond level upstream (Hora, 2016).

Reduce the exit gradient, i.e. increase the creep length, to reduce the piping phenomenon. Increase the impervious floor length and provide upstream and downstream cutoffs to increase the creep length. Cutoffs are vertical impervious piles installed upstream, downstream, or in the middle of an impervious floor to protect seepage through the weir under bed and reduce uplift force by increasing seepage creep length and to protect the riverbed from erosion, a protection structure and launching apron are provided at the upstream and downstream ends of the impervious floor (Garg, 2005).

2.2. Design practice of weir

There are three elements to the design of a modern diversion weir or barrage Hydrological, Hydraulic and Structural analysis (Afera, 2004).

2.2.1 Hydrological analysis

In the design of hydraulic and irrigation structures in ungauged basins, estimating the peak flow within a specified frequency is of primary importance. To estimate the magnitude of peak floods, hydrologic models may be used depending on the availability of data. Hydrologic models are simplified, conceptual representations of components of the hydrologic cycle. They are primarily used for hydrologic prediction and to enhance understanding of hydrologic processes (Bitew, 2019).

Hydrologic models can generally be classified as either stochastic or deterministic. Stochastic models produce outputs with inherent randomness, while deterministic models do not involve randomness in their predictions. Deterministic hydrologic models are further classified into three major categories. The first category is the lumped model, which assesses the catchment response solely at the outlet without accounting for the response of individual sub-basins. The second category is the semi-distributed model, which allows for spatial variation by dividing the catchment into several sub-basins. The third type is the distributed model, which permits parameters to vary spatially at a resolution determined by the user (Cunderlik, 2003).

Distributed hydrological models, such as the European Hydrological System Model (MIKE-SHE) and Modular Modeling System (MMS), and semi-distributed models like the Hydrological Engineering Center Hydrological Modelling System (HEC-HMS), Soil and Water Assessment Tool (SWAT), Topography Based Hydrological Model (TOPMODEL), Hydrologiska Byråns Vattenbalansavdelning (HBV), and Hydrological Simulation Program-Fortran (HSPF), have been developed for runoff estimation based on data availability and the complexity of hydrological systems (Bitew, 2019).

2.2.1.1 Lumped models

Parameters of lumped hydrologic models do not vary spatially within the basin and thus, basin response is evaluated only at the outlet, without explicitly accounting for the response of individual sub-basins. Parameters of lumped models often do not represent physical features of hydrologic processes and usually involve certain degree of empiricism. The impact of spatial variability of model parameters is evaluated by using certain procedures for calculating effective values for the entire basin. The most commonly employed procedure is an area-weighted average. Lumped models are not usually applicable to event-scale processes. If the interest is primarily in the discharge prediction only, then these models can provide just as good simulations as complex physically based models (Bitew, 2019).

A. Rational method

According to Chaw, (1988) The rational method is that if a rainfall of intensity i begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration tc when all of the watershed is contributing to flow at the outlet. The product of rainfall intensity i and watershed area A is the inflow rate for the system, $iA,$ and the ratio of this rate to the rate of peak discharge Q(which occurs at time tc) is termed the runoff coefficient C (0 < C < 1).

The assumptions associated with the rational method are: (Chaw, 1988).

i. The computed peak rate of runoff at the outlet point is a function of the average rainfall rate during the time of concentration, that is, the peak discharge does not result from a more intense storm of shorter duration, during which only a portion of the watershed is contributing to runoff at the outlet.

ii. The time of concentration employed is the time for the runoff to become established and flow from the most remote part of the drainage area to the inflow point of the sewer being designed.

iii. Rainfall intensity is constant throughout the storm duration

B. United states soil conservation service (SCS) curve number method

United States soil conservation service curve number method is widely used for estimating runoff on small to medium sized un-gauged drainage basins. The SCS approach involves the use of simple empirical formula, tables and curves. The empirical equations require the rainfall and watershed coefficient as inputs. The watershed coefficient called as curve number (CN) is an index that represents the combination of hydrologic soil group, land use and land treatment classes. SCS method enables the hydrologist to stimulate the various design alternatives and compare the result (Chaw, 1988).

For the computation of runoff, using the SCS curve number method estimates precipitation excess as a function of cumulative precipitation, soil type, land use/cover, and antecedent moisture condition expressed by the following equation: (Subramanya, 2008).

$$Q = \frac{(P-Ia)^2}{(P-Ia+S)} \qquad \ldots 2.1$$

Where: Q = accumulated rainfall depth at time t,

P = direct surface run off in depth (mm)

Ia = the initial abstraction (initial loss), Ia = 0.2*S

S = maximum potential retention in mm.

S value is derived from curve number (CN) using following formula:

$$S = \frac{25400}{CN} - 254 \quad \text{...2.2}$$

Where CN = function of watershed hydrologic land use/land cover units hydrologic soil groups antecedent moisture condition. CN values can be obtained for different land uses and hydrologic condition from the standard Table CN values for AMC-I and II can be obtained using the following empirical equation:

$$CN\ (I) = \frac{4.2 * CN\ (II)}{10 - 0.058 * CN\ (II)} \quad \text{...2.3}$$

$$CN\ (III) = \frac{23 * CN\ (II)}{10 + 0.13 * CN\ (II)} \quad \text{...2.4}$$

The SCS method has an index, which is called curve numbers (CN) to represent the combined hydrologic effect of soil, land use; agricultural land treatment class hydrologic conditions and antecedent soil moisture. The SCS method has also developed a soil classification system that consists of four hydrologic groups according to their minimum infiltration rate which is obtained for a bare soil after prolonged wetting. The groups are identified by the letters A, B, C, and D (Subramanya, 2008).

Group A: deep sand, deep loess, aggregated silts, high infiltration rate,>7.6mm/hr;

Group B: shallow loess, sandy loam, moderate infiltration rate (3.8 to 7.6mm/hr);

Group C: clay loams, shallow sandy loams, soils low in organic content and soils high in clay, low infiltration rate (1.3 to 3.8 mm/hr)

Group D: soils that swells significantly when wet, heavy plastic clay, and certain saline soils, very low infiltration rate (<1.3mm/hr).

It is the time at which the entire watershed begins to contribute runoff and calculated as the time taken for runoff to flow from the most hydraulically remote point in the watershed to the outlet. There might be some possible paths to considered in determining the longest travel time, hence the longest flow path which the longest travel time is likely to occur should be identified and will be used for Tc, which can be computed by Kirpich's equation (Belango, 2021).

C. IHACRES model

The IHACRES (Identification of Hydrographs and Component flows from Rainfall, Evaporation, and Streamflow data) hydrological model is a lumped model that treats a catchment as a single unit, assuming uniform hydrological properties across the entire area. This model is designed to transform rainfall and climate data, typically rainfall and temperature or evaporation, into streamflow estimates. By applying both linear and nonlinear techniques, IHACRES can simulate the relationship between rainfall and runoff, allowing for the separation of quick and slow flow responses, such as surface runoff and groundwater baseflow (Jakeman, 1990).

IHACRES is widely used in various hydrological applications, including flood forecasting, drought assessment, and water resource management. Its simplicity makes it particularly suitable for catchments with limited data availability, and it is often used to predict streamflow in ungauged basins by transferring calibrated parameters from similar catchments. However, being a lumped model, IHACRES assumes that hydrological processes are uniform across the catchment, which may not be appropriate for catchments with diverse land use, topography, or soil properties. Additionally, it does not model spatial variability within the catchment, limiting its effectiveness in studies that require detailed spatial analysis (Croke, 2004).

Despite these limitations, IHACRES is valued for its flexibility, as it incorporates both linear and nonlinear components to simulate various hydrological responses. By calibrating the model using observed streamflow data, users can improve its accuracy for future rainfall events, making it a useful tool for hydrological studies, especially in data-scarce regions. However, for catchments requiring detailed spatial variability modeling, more complex distributed models may be more appropriate (Cunderlik, 2003).

2.2.1.2 Semi-distributed models

Parameters of semi-distributed (simplified distributed) models are partially allowed to vary in space by dividing the basin into a number of smaller sub-basins. There are two main types of semi-distributed models: 1) kinematic wave theory models (KW models, such as HEC-HMS), and 2) probability distributed models (PD models, such as TOPMODEL). The KW models are simplified versions of the surface and subsurface flow equations of physically based hydrologic models. In the PD model's spatial resolution is accounted for by using probability distributions of input parameters across the basin (Hussain, 2021).

A. HEC-HMS Model:

The HEC-HMS model is physically based and conceptually semi-distributed model designed to simulate rainfall-runoff processes in a wide range of geographic areas, from large river basin water supplies and flood hydrology to small urban and natural watershed runoffs. It easily operates huge tasks in relation to hydrological studies, including losses, runoff transform, open channel routing, analysis of meteorological data, rainfall-runoff simulation, and parameter estimation (Bitew, 2019). Moreover, the HEC-HMS uses separate models that compute runoff volume, models of direct runoff, and models of baseflow. It has nine different loss methods, some of which are designed primarily for simulating events, while others are intended for continuous simulation. It also has seven different transformation methods. For example, the Snyder Unit Hydrograph (Moges, 2007). and Clark Unit Hydrograph (Daide, 2021). methods have been applied successfully to simulate long-term stream flows elsewhere.

B. Hydrologiska Byråns Vattenbalansavdelning (HBV) Model

The HBV Model is a conceptual, semi-distributed hydrological model initially developed by the Swedish Meteorological and Hydrological Institute (SMHI). It is widely used for simulating rainfall-runoff processes, snowmelt, and the overall water balance in catchments. The primary inputs to the HBV model are precipitation, temperature, and potential evapotranspiration. The model represents the catchment as a series of interconnected reservoirs, each representing distinct hydrological processes, such as snow storage, soil moisture storage, and groundwater storage (Cunderlik, 2003).

The Runoff module handles water that exceeds the soil's storage capacity, routing it toward the catchment outlet using a simple linear reservoir approach. The Groundwater module allows a portion of the infiltrated water to percolate into the groundwater reservoir, thereby contributing to the river system's base flow (Marti, 2021).

The HBV model is frequently used for flood forecasting, water resource management, and climate impact studies. It's effective in both snow-dominated and rainfall-dominated catchments. However, despite its practicality, the model does have limitations. Since it is a semi-distributed model, it does not account well for spatial variability within large or heterogeneous basins. Moreover, the model often requires parameter calibration, and different sets of parameters may provide similar results without necessarily representing real-world processes accurately (Cunderlik, 2003).

C. SWAT (Soil and Water Assessment Tool):

The Soil and Water Assessment Tool (SWAT) is a semi-distributed hydrological model designed to simulate the impact of land management practices on water, sediment, and agricultural chemical yields across large and complex watersheds. SWAT operates by dividing a watershed into sub-basins, which are further divided into Hydrological Response Units (HRUs) based on land use, soil type, and slope (Arnold, 1998).

SWAT can simulate various components of the hydrological cycle, including precipitation, evaporation, surface runoff, infiltration, percolation, and soil water dynamics. It calculates runoff using the SCS-CN (Soil Conservation Service Curve Number) method and models sediment transport with the Modified Universal Soil Loss Equation (MUSLE). Additionally, SWAT incorporates nutrient cycling processes, such as the movement of nitrogen and phosphorus. The model also allows for the simulation of different land management practices, such as conservation tillage and crop rotations, which affect hydrological and sediment processes (Neitsch, 2011). Calibration and validation are crucial for ensuring the accuracy of SWAT predictions. Calibration involves adjusting model parameters to match observed data, while validation uses independent data sets to test the model's performance under various conditions (Cunderlik, 2003).

Despite its strengths, SWAT has limitations. It requires detailed input data, which can be time-consuming to gather and prepare. The model's semi-distributed nature means it may not capture micro-scale variations, and its daily time step may not be suitable for modeling very short-term or long-term events without adjustments. Additionally, some parameters are sensitive and can significantly impact model outputs, leading to potential inaccuracies if not properly calibrated. The model also makes certain simplifications and assumptions, which can affect its precision. For example, the SCS-CN method may not fully represent all runoff processes, and the assumption of homogeneity within HRUs can overlook variability within these units. Despite these challenges, SWAT remains a powerful tool for watershed management and water resource planning when used with careful consideration of its limitations (Cunderlik, 2003).

D. TOPMODEL (Topography-based Hydrological Model):

TOPMODEL (Topography-based Hydrological Model) is a semi-distributed hydrological model that predicts runoff generation in a watershed based on topographic information. It focuses on the relationship between landscape topography and hydrological processes, using the *Topographic Index* (TI) to quantify the tendency of water to accumulate in certain areas of a catchment (Kirkby, 1997).

TOPMODEL simplifies the complex dynamics of catchment hydrology by relating the depth of the water table to the topography, and it simulates subsurface flow based on the topographic gradient, dynamically adjusting the extent of saturated areas in response to changes in water storage. The model is computationally efficient, requiring fewer parameters and less data than fully distributed models, making it particularly useful in topographically dominated catchments. However, it has limitations, such as assuming homogeneous soil and geological properties across the catchment and a shallow water table, which may not be applicable in all regions. Additionally, its reliance on the saturation-excess mechanism may not capture runoff dynamics in areas dominated by infiltration-excess runoff (Cunderlik, 2003).

TOPMODEL is most effective in hilly or mountainous regions where topography strongly controls water movement, and it has been widely applied for flood prediction, land-use change impacts, and assessing climate change effects on hydrology. However, its simplifying assumptions limit its accuracy in catchments with complex soil and geological variability (Kirkby, 1997).

2.2.1.3 Distributed models.

Parameters of distributed models are fully allowed to vary in space at a resolution usually chosen by the user. Distributed modeling approach attempts to incorporate data concerning the spatial distribution of parameter variations together with computational algorithms to evaluate the influence of this distribution on simulated precipitation-runoff behavior. Distributed models generally require large amounts of (often unavailable) data for parameterization in each grid cell. However, the governing physical processes are modeled in detail, and if properly applied, they can provide the highest degree of accuracy (Bitew, 2019).

Table 2.1 Descriptions, applications, and limitations of hydrologic models

Model Type	Description	Applications	Limitations
Lumped Models	Simplify the catchment by considering it as a single unit. These models aggregate all inputs and outputs over the entire catchment.	- Flood forecasting - Hydrologic design - Water resource management	- Do not account for spatial variability within the catchment. - Less accurate for large and heterogeneous catchments.
Semi-Distributed Models	Divide the catchment into several sub-catchments or hydrologically similar units. These models consider spatial variability to some extent.	- Basin-scale hydrologic studies - Watershed management - Intermediate-scale flood prediction	- Require more data than lumped models. - May still oversimplify complex catchment processes.
Distributed Models	Consider spatial variability in detail by dividing the catchment into grid cells or small units. These models simulate hydrological processes at each cell.	- Detailed hydrologic studies - Urban hydrology - Climate change impact assessment	- Require extensive data and computational resources. - Complex to set up and calibrate

Source: (Cunderlik, 2003).

Table 2.2 Comparison of hydrological models

Model	Purpose	Strengths	Limitations	Suitability for Study
HEC-HMS	Simulation of precipitation-runoff processes, flood forecasting	- Versatile for event-based and continuous simulations - Effective for small and medium watersheds - User-friendly with good calibration tools	- Lacks detailed water quality analysis - Less robust groundwater interaction modeling	Suitable for flood analysis and streamflow simulation in watersheds of various sizes
SWAT	Long-term simulation of water quantity and quality	- Excellent for analyzing land-use changes and agricultural practices - Detailed water quality analysis	- Requires extensive input data for soil, land use, and management practices - Less effective for short-term flood forecasting	Better suited for long-term watershed management and assessing agricultural impacts.
MIKE SHE	Physically-based model for surface and groundwater interaction	- Best for groundwater and surface water interaction studies - Comprehensive hydrological cycle modeling	- Requires extensive data and expertise - High computational demand	Ideal for complex hydrological systems with significant groundwater and surface water interactions.
WEAP	Integrated water resource management	- Ideal for planning and water allocation scenarios - Simplified decision-making interface	- Not designed for flood routing - Less robust for hydrological forecasting	Suitable for water allocation, management, and planning scenarios

Source: (Cunderlik, 2003).

2.2.1.4 Model evaluation and statistical analyses

The Nash-Sutcliffe efficiency (NSE) and coefficient of determination (R^2) were used as statistical indices to assess the model performance. R^2 (the coefficient of determination) indicates the degree of linear relationship between simulated and observed data. A R^2 value close to one indicates a better performance. However, it is very sensitive to extremely high values. The Nash-Sutcliffe efficiency (NSE) is one of the most commonly used criteria. This is a normalized statistic, which can be used to determine the goodness of fit. The NSE ranges from $-\infty$ to 1, with 1 indicating a perfect match (Aliye, 2020).

2.2.2 Design rainfall frequency analysis

According to Subramanya (2008), some of the commonly used frequency distribution function for the prediction of extreme maximum rain fall values is

i. Gumbel distribution method.

ii. Log Pearson type III distribution method.

i. Gumbel distribution method

It is one of the widely used probability distribution functions for extreme values in hydrologic and metrological studies for the prediction of flood peaks, maximum rainfalls, wind speed, etc, and expressed by equation (Subramanya, 2008).

$$X_T = X_{mean} + K_T * \delta_X \qquad \text{...2.5}$$

Where X_T = annual maximum rain falls of T years return period,

X_{mean}= mean of the annual maximum daily rainfall and

K_T= frequency factor.

$$K_T = \frac{Y_T - Y_n}{S_T} \qquad \text{...2.6}$$

$$Y_T = -Ln\left(\frac{T}{T-1}\right) \qquad \text{...2.7}$$

Yn =reduced mean, a function of sample size N and obtained from standard table 7.3 (Subramanya, 2008).

Sn =reduced standard deviation, a function of sample size N and obtained from table 7.4 (Subramanya, 2008).

ii. Log Pearson type III distribution method

The Log-Pearson Type III distribution is a statistical technique for fitting frequency distribution data to predict the design flood for a river at some site. Once the statistical information is calculated for the river site, a frequency distribution can be constructed. In this method, the verity is first transformed into logarithmic form (Base 10) and transformed data is then analyzed. If X_T is the verity of random hydrologic series then the series of Y_T varies where Y_T=$\log_{10} X_T$, are obtained, for this Y series for any recurrence interval (Chaw, 1988).

$$Y_T = Y_{mean} + K_T * \delta_Y \text{.........} 2.8$$

Where, K_Y is a frequency factor which is a function of T, the coefficient of skew Cs, and δy= sample standard deviation and Y_{mean} = sample mean,

Which are given in equations 3.11 and attained from standard table 7.6, (Subramanya, 2008)

$$Y_{mean} = \sum_{i=1}^{n} \frac{\log(xi)}{N} \text{.........} 2.9$$

$$\delta_Y = \sqrt{\sum_{i=1}^{n} \frac{(Y_i - Y_{mean})^2}{N-1}} \text{.........} 2.10$$

$$C_S = N * \frac{(Y_i - Y_{mean})^3}{(N-1)(N-2)*\delta_Y{}^3} \text{.........} 2.11$$

2.2.3 Hydraulic analysis

The hydraulic analysis of the weir entails determining the following parameters, which are used to calculate the dimensions of the head work structures waterway length, water level upstream and downstream, energy level upstream and downstream. The depth of scouring, the amount of affluent, and the shape of the weir are all factors to consider. The design discharge is regarded as an inflow upstream of the structures that can safely pass downstream of the weir without causing any damage to the head work apparatus (Kefyalew, 2017).

The estimation of the shape and height of the weir, clear waterway or length of weir crest, discharge and head over the weir, and flood and energy level, afflux and scour depth are all part of hydraulic design. The waterway and afflux are correlated with increase of afflux waterway decreases and vice versa hence a limit placed on maximum afflux shall limit the minimum waterway. A weir with a long crest gives a small linear discharge and hence the required energy dissipation per meter of the crest is smaller than what is needed for a shorter crest length (Afera, 2004).

The amount of the afflux determines the top level of guide banks and marginal bunds, as well as the length of the marginal bunds. The afflux also has an impact on the dynamic action downstream of the weir, as well as the location and parameters of the hydraulic jump.

Although a greater afflux narrows the waterway, it increases the discharge per unit length of weir. As a result, the depth of the scour increases, raising the cost of protection works. Due to possible outflanking, a higher afflux increases the risk of river training structures failing. At the same time, the discharge intensity as a result of scour will increase, necessitating an increase in the length of loose protections upstream and downstream, as well as the depths of pile lines at both ends, all of which will be expensive. As a result, it is often preferable to keep the afflux to a safe value of 1 to 1.2 meters, most commonly 1 meter. However, in a steep reach with a rocky bed, a higher afflux value is allowed (Garg, 2005).

Pond level: The pond level is the water level that must be kept in the under sluices pocket (i.e., upstream of the canal head regulator) in order for the canal to maintain full supply level when full supply discharge is fed into it. The full supply level at the canal's head is determined by the canal's longitudinal section. The pond level is maintained about 1.0 to 1.2 meters above the canal's full supply level to ensure that enough working head is available even though the canal's head reach has silted up or if the canal needs to be fed excess water. If the pond level is limited in certain circumstances, the full supply level is determined by subtracting the working head from the pond level (Asawa, 2008). Seepage theories are used to calculate the uplift pressure at various points under hydraulic structures of complex bed configuration. Generally, there are three theories to analyze subsurface flow: Bligh's, Lane's and Khosla's theory (Novak, 2007).

I. Bligh's theory:

According to Bligh's theory the Percolating water follows the outline of the foundation of the hydraulic structure. Traverse covered by the flowing water through soil is called creep length. It is assumed in this theory that the potential drop is proportional to the length of the creep; there is no difference between horizontal and vertical creep and head loss along the creep length is constant i.e. unit loss=HL/L where HL is the total loss (difference between upstream and downstream water level) and L is the total creep length (Garg, 2005).

Bligh's theory is quite simple and convenient a large number of early irrigation structures were designed using this theory some of these structures are existed even today, but unfortunately a few of them failed. The theory is now rarely used for the design of large, important irrigation structures. However, sometimes it is used for the design of small structures or for the preliminary design of large structures (Afera, 2004).

II. Khosla’s method to analyze subsurface flow:

Khosla’s broke the complicated weir profile into a number of simple common profiles and then studies each profile independently for finding out uplift pressures. He thus made it clear that the loss of head does not take place uniformly in proportion to the length of creep. It actually depends on the profile of the base of the river. Secondly, he also established that the safety against undermining is not obtained by flat hydraulic gradient but the exit gradient should be kept below the critical value (Hora, 2016).

The simple profiles which are most useful are:

a) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end.

b) A straight horizontal floor depressed below the bed but without any vertical cut-off’s piles.

c) A straight horizontal floor of negligible thickness with an intermediate sheet pile.

The key points are the junctions of the floor and the pile lines on either side and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms in to which the complex profile has been broken is valid for the complex profile itself, if corrected for; Correction for the mutual interference of piles, correction for thickness of floor and correction for the slope of the floor (Novak, 2007).

The exit gradient is the hydraulic gradient of the seepage flow under the base of the weir floor. The rate of seepage increases would cause “boiling” of surface soil, the soil being washed away by the percolating water. According to Khosla's theory for standard structure form consisting of floor-length” b” with a vertical depth” d” the exit gradient (GE) at its downstream end is given by:

$$GE = \left(\frac{H}{d}\right) * \left(\frac{1}{\pi\sqrt{\lambda}}\right) \qquad 2.12$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, \alpha = \frac{b}{d} \qquad 2.13$$

Where: H is the total seepage head loss, or residual head at the upstream end, b: is the length of impervious floor, and d: is the length of the d/s pile.

Exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. Factor of safety equal to 4 to 5 is used to keep the structure safe against piping (Novak, 2007).

2.2.4 Structural analysis

The structural analysis of the head work structure general consists of weir proper body and River training work in both static and hydrodynamics condition. Diversion head work stability analysis is to keep compressive stresses under control while preventing the growth of tension stresses in the concrete. The ultimate tension strength of unreinforced concrete or possibly stone masonry is just one-tenth of the ultimate compressive strength. As a result, allowing any tension stresses is considered unwise. The "middle third rule," which maintains the resultant of all forces in the middle third of the structure, is used to achieve this. Forces acting on the structure and possible moments resulting from these forces are included in this analysis, which is done around the external toe of the critical section of a weir (MoA, 2018).

2.3. Causes of weir failures

Failure of hydraulic systems such as a weir or a barrage is caused by a combination of subsurface and surface flow at the site, as well as poor construction quality. Pipes, uplift force, suction caused by standing waves, and scouring on both upstream and downstream of the buildings are examples of such causes. When the hydraulic gradient or exit gradient is greater than the critical value of the soil, the surface soil at the downstream end boils first and is washed away by percolating water. Seepage water forms a channel in the form of a pipe as the process of removing or washing out soil continues. This is known as piping, and it can lead to foundation failure. Similarly, percolating water exerts an uplift force on the floor from the bottom, and if the weight of the floor is insufficient to resist this uplift force, the floor may crack or burst (Garg, 2005).

2.3.1 Causes of subsurface flow

Uplift pressure and piping are the two main causes of such failure (Garg, 2005).

1) Piping or undermining

This happens when water from the upstream side percolates through the foundation's bottom and appears at the weir or barrage floor's downstream end. By scouring at the point of emergence, the force of this percolating water eliminates soil particles. As the soil particles are steadily removed, a depression gradually forms at the bottom of the foundation, extending backwards towards the upstream (Novak, 2007).

2) Uplift pressure

When percolating water exerts upward pressure on the foundation of the weir or barrage, this phenomenon happens. It may fail by rapture if this uplift is not counterbalanced by the structures self-weight. Its distribution is greatest upstream and diminishes as it moves downstream (Garg, 2005).

2.3.2 Causes of surface flow

1) By hydraulic jump

Hydraulic jump occurs when water flows at a high velocity over the crust of the weir or over the gates of the barrage. On the downstream side of the hydraulic jump, a suction pressure or negative pressure acts in the direction of uplift pressure. If the impervious floor is not thick enough, the structure would fail due to rapture (Novak, 2007).

2) By scouring during floods

The barrage's gates are kept open, and the water flows at a high rate. The water may also flow at a very high rate over the weir's crest. In all cases, scouring may occur on both the downstream and upstream sides of the structure. Due to scouring on both the downstream and upstream sides, shearing jeopardizes the structure's stability (Garg, 2005).

2.3.3 Failure due to silt

When a weir is built across a river, it causes progressive retrogression on the downstream side and aggravation on the upstream side. Because the initial flow area calculated for the approach velocity in the upstream side encloses between the upstream high flood level and pond level, upstream aggradations have the propensity to increase the approach velocity in the upstream side of the weir (Novak, 2007).

Downstream retrogression, on the other hand, results in a decrease in downstream river phases, which must be considered during the design process. Increased exit gradients are caused by lower river water levels due to retrogression on the downstream side, putting the structure's safety in jeopardy. As a rule of thumb, when computing the design floor and exit gradient, assume a retrogression of 0.3-0.5m. Retrogression would not be inferred if the downstream river course is solid rock (Garg, 2005).

As a result, silt deposition could impair the soundness of the diversion headwork's operation by clogging gates (under the sluice gate for the weir and the gate for the barrage) and other waterways, diminishing its productivity or causing it to collapse. As a result, the processes listed below are proposed (Novak, 2007).

1) On Rivers with high sediment concentrations, such as those with poor catchments on the upper reach of a river or seasonal rivers, an excluder in the river or an extractor along the canal may be constructed, depending on the magnitude and type of sediment.
2) Because suspended silt is usually carried by the flood on the lower reach, an extractor in the canal would be appropriate. The decision is made based on the current state of the silt, and the designer chooses between the two mechanisms.

2.4. Concept of performance

Performance is the degree to which a system achieves its objectives. But objectives differ for individual systems and may be reset from time to time by a management decision (Fikru, 2014).

There are many reasons to evaluate the performance of irrigation schemes. It can be to improve scheme operations, to assess progress against strategic goals, as an integral part of performance-oriented management, to evaluate the functionality of a scheme, to evaluate impacts of interventions, to better understand the determination of performance, to diagnoses constraints, and to compare the performance of a scheme with others or with the same scheme over time (Lambisso, 2005).

Performance evaluation is the major component of proper irrigation water management; the most common problem in performance evaluation of irrigation schemes is lack or non-reliability of data. In most cases, people believe that irrigation schemes with modern systems will have high performance, but the use of a modern method of field application alone does not guarantee high performance. The efficiency of the irrigation system depends as much or more on the capability of the irrigator as on the quality of life system (MoWR, 2002).

Planning, design and construction of irrigation schemes are mainly dealing with creation of physical infrastructure to facilitate the capturing of water from its source and transportation up to the farm level. These physical facilities need to be properly operated to ensure the capturing, allocation and delivery of water at the right time and adequate quantity. Maintenance activities are designed to ensure the capabilities of physical infrastructure to deliver the intended amount of water over the project life time (Kefyalew, 2017).

2.5. Performance standard indicators

It is known Poor performances of headwork leads to reduced efficiency of the irrigation scheme and causes to substantial economic losses. Performance evaluation of irrigation schemes is mainly becoming an important active field of research where several approaches and methodologies have been developed for assessing irrigation performance from different perspectives (Kefyalew, 2017). According to (Lysne, 2003) underperformance concerns related to headwork design and its components procedure performance standards developed performance indicators are classified as hydraulic and structural performance indicators; the principal terms are described as follows.

2.5.1 Hydraulic performance indicators

Hydraulic performance measures are introduced as functions of defined state variable and are used to indicate the state of system performance relative to the objectives of passage of high floods, water withdrawal, passage of sediments prevalence of seepage and scouring (Lysne, 2003). To address concerns related to hydraulic performance indicators for headwork structures in a systematic way standard developed have been discussed below.

2.5.1.1Passage of floods, including hazard floods

The diversion headwork structures must be in a way that it withstands the maximum flood occurrence is unquestionable. Because the unexpected maximum floods occurring in the area endanger the structure. The method adopted for spilling or passage of floods should be enough to accompany safely the portable amount of flood occurred, without endangering or causing serious damages to the diversion headwork and its appurtenant as a whole. Flash floods due to natural hazards such as overtopping should be handled with some structural damages (Sisay, 2022).

2.5.1.2 Passage of sediments

The design of the headwork must prevent the bed load from approaching the intake and causing clogging of the intake. It needs to facilitate the passage of bed-load through sluice-ways without causing significant structural damages to the headwork structures (Lysne, 2003).

According to (Lambisso, 2005) two methods of obtaining the transportation of sediment with the river flow which are the separation of the sediments before the intake and flushing of sediments from the intake. And for further prevention of the sediments and bed-load in the lower layer of the flow is separated from upstream of the intake at all flow conditions by placing an inlet of the intake above the intake bed.

2.5.1.3 Seepage prevalence

Seepage through the weir structure is common in many old masonry weirs and, in many instances; this is no great consequence and can be ignored. Over a long time, this type of seepage can be problematic with the risk of structural failure i.e. risk on un-bonded masonry blocks being washed away in a flood. In this case, it will probably be necessary to repair the structure by grouting (risk of pollution) or by dismantling and reassembling the structure (Baban, 1995).

The ancillary advantages of masonry kind of weir are it was commonly used in the early days of weir construction, can be very attractive, and used to disguise a concrete structure, simple to construct and economy (Asawa, 2008).

A weir constructed from gabions requires particular attention concerning seepage. Gabions are permeable and unless the boxes are sealed in some way, low flows in the river will tend to pass through the weir rather than over it. As well as running the risk of erosion of the foundations of the weir, flow through the weir will create unfavorable conditions for fish. Gabions can be sealed by a concrete facing/ by use of an impermeable membrane, which must be tried into the bed and banks of the river to avoid being undermined or by past (Afera, 2004).

2.5.2 Structural performance indicators

It has been observed over the years that the diversion weir collapses, initially not because of the unbalanced moment, but mainly due to the foundation scouring. The stability analysis becomes important where the structure and the apron are of two different materials and acts as two independent units, i.e. none monolithically or as particular schemes that the structure is built on alluvial soil for such cases designing the structure based on the surface and sub-surface flow considerations, the structure should have to be checked for stability (Baban, 1995).

Therefore, the diversion headwork as a whole should be structurally safe and stable. At the last step in the design procedure of the diversion weir, the stability of the structure should be checked against various threats such as safety against sliding, safety against overturning checked taking into consideration of every involved and related parameter in each case

Stability analysis of the weir body is checked against sliding and overturning for dynamic and static conditions (Turan, 2004). The following points have to be considered while analyzing the stability of the weir body:

2.5.2.1 Stability against overturning

Overturning failure occurs when the overturning moment exceeds the resisting moment. Therefore, to be on the safer side the resisting moment should exceed the overturning moment. Since unpredictable situations are inevitable to occur at some time (t) of the project lifetime cause the toppling moment to exceed the balance one. A minimum factor of safety of 1.5 is sometimes specified, however if the weir is safe against tension and crushing failures, a factor of safety between, 1.5-2.5 is usually applied and the weir does not fail in overturning. If the resultant strikes just at the middle third point, a factor of safety of 2.0 is available against overturning (Arora, 1996).

Factor of safety against overturning: Occurs when the overturning moment exceeds the resisting moment (Garg, 2005).

2.5.2.2 Stability against shear and sliding

The aprons of the structure and the weir body are considered for stability against shear and sliding. The structure may slide in the flow direction if there are not enough grips between the base and the foundation. To avoid this failure of the weir due to sliding at any horizontal section or the base, the weir should be designed so that the sliding forces do not exceed the resisting force (Sisay, 2022).

The factor of safety against sliding (Fs) should be greater than 1.0. A low value Fs of 1.0 is acceptable because the shear strength has been neglected. The actual factor of safety would be much greater than 1.0 if the shear strength is considered (Arora, 1996).

Factor of safety against sliding: to prevent this problem the weir should be designed so that the sliding forces do not exceed the resisting force (Garg, 2005).
Factor of safety against tension: happen if the resultant force is not, pass through the middle third section of the base, (Garg, 2005).

3 MATERIALS AND METHOD

3.1. Description of the study area

3.1.1 Location

Wondo Genet is one of the Woredas located in the Sidama Regional State located in the great rift valley of Ethiopia that extends between 6^0 59‟N- $7^0$6‟ N and $38^0$37‟- 38^0 43‟ E, and an elevation of 1720-2620 amsl., about 272 km south of the capital city, Addis Ababa, and about 24 km east of Hawassa Town (WGWA, 2016). The woreda covers a total area of 15,145 ha or 151.45 sq. km, which is bordered on the south by Melga woreda, on the west by Hawassa zuria, and on the north and east by Oromia Region.

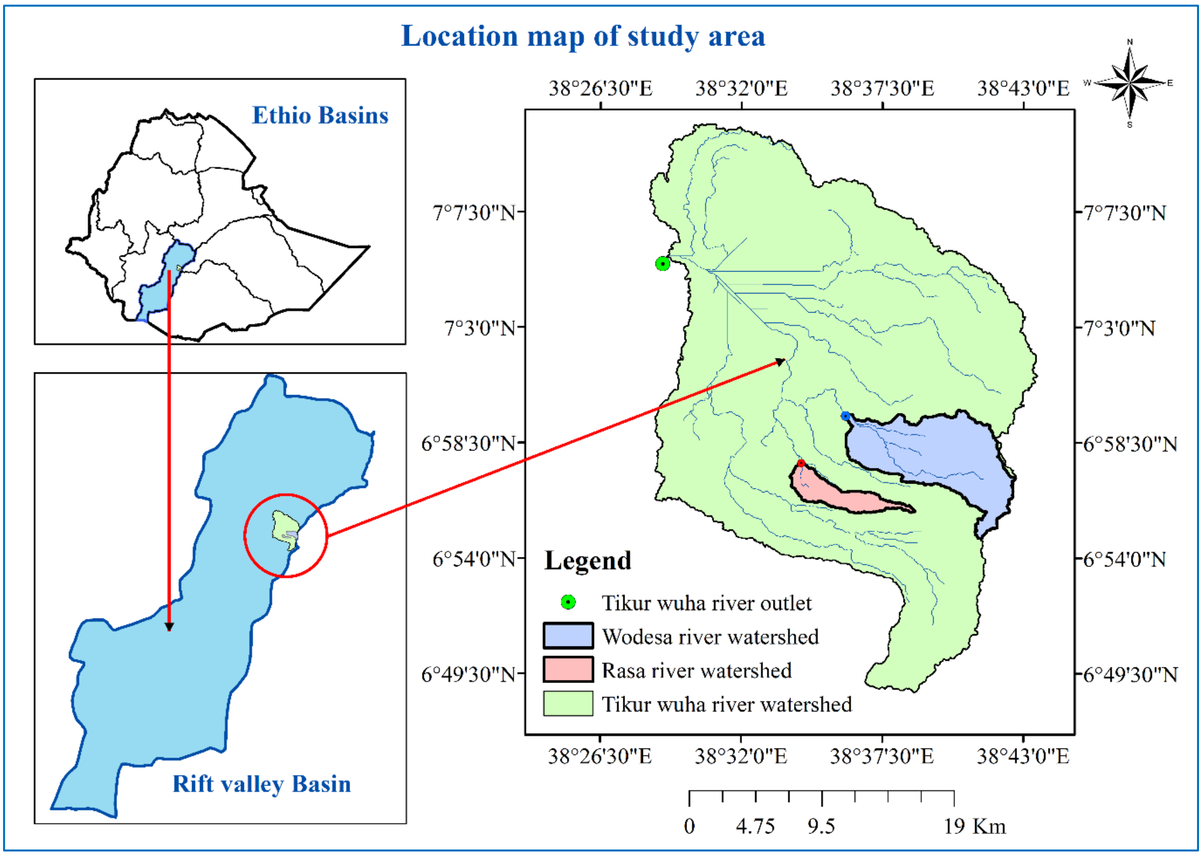


Figure 3.1 Location map of the study area

3.1.2 Population

Based on the 2007 Census conducted by the CSA, the woreda has a total population of 155,715 of whom 79,664 are men and 76,051 women, or 14.85% of its population is urban dwellers (WGWA, 2016).

3.1.3 Climate

Wondo Genet has a tropical wet-dry highland climate characterized by marked seasonal contrasts, unreliable precipitation, and a dry winter season lasting two to four months. It has a bimodal rainfall distribution with two rainy seasons: January to April and a more extensive one from July to September. There is also a long dry season from October to February. The annual average rainfall is about 1200 mm, and the mean temperature throughout the year varies between 16.7°C and 21.9°C. The minimum and maximum temperatures of the study area are 8°C in November and 32.9°C in March, respectively. March is the hottest month with a maximum temperature of 32.9°C, and July is the coldest month with a minimum temperature of 10.4°C. From 1990-2022 the mean annual rainfall and average Temperature for Wondogenet are presented with their respective districts in figure 3.2.

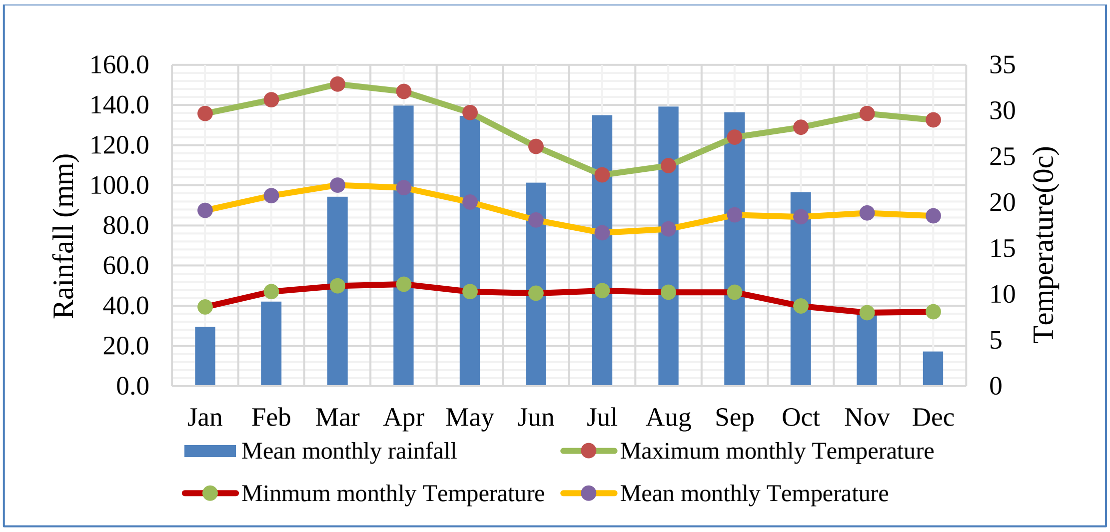


Figure 3.2 Monthly temperature and rainfall variation of Wondogenet

3.1.4 Land use land cover

Land use refers to the management and proper utilization of renewable natural resources, such as soil, forests, water, and other forms of biodiversity. The major factors influencing land use include climate, altitudinal variation, and human influence on resource bases. Climate, as one of the determinant factors of land use activity in a given locality, depends on the amount and intensity of rainfall and temperature patterns. Regarding topographic conditions, some of the major influential factors include slope, altitude, and soil texture, which affect agricultural activities and vegetation cover. Additionally, population pressure and density significantly impact the sustainable utilization of resource bases. Land use can be classified according to its potentiality, capability, and suitability for better and more sustainable agricultural production (Wondrade, 2023).

Wondo genet is in an area where the vegetation cover is relatively better of all land cover type, forest with different densities cover about 28.8% of the area. The forest covers are further classified dense forest, and woody vegetation (moderate forest, and open shrub) with respective coverage of 12.2%, 1% and 16.6%. Perennial crop cultivation and grass land are the first and third major land cover types with coverage of about 35.2% and 13% respectively (WGWANRO, 2024).

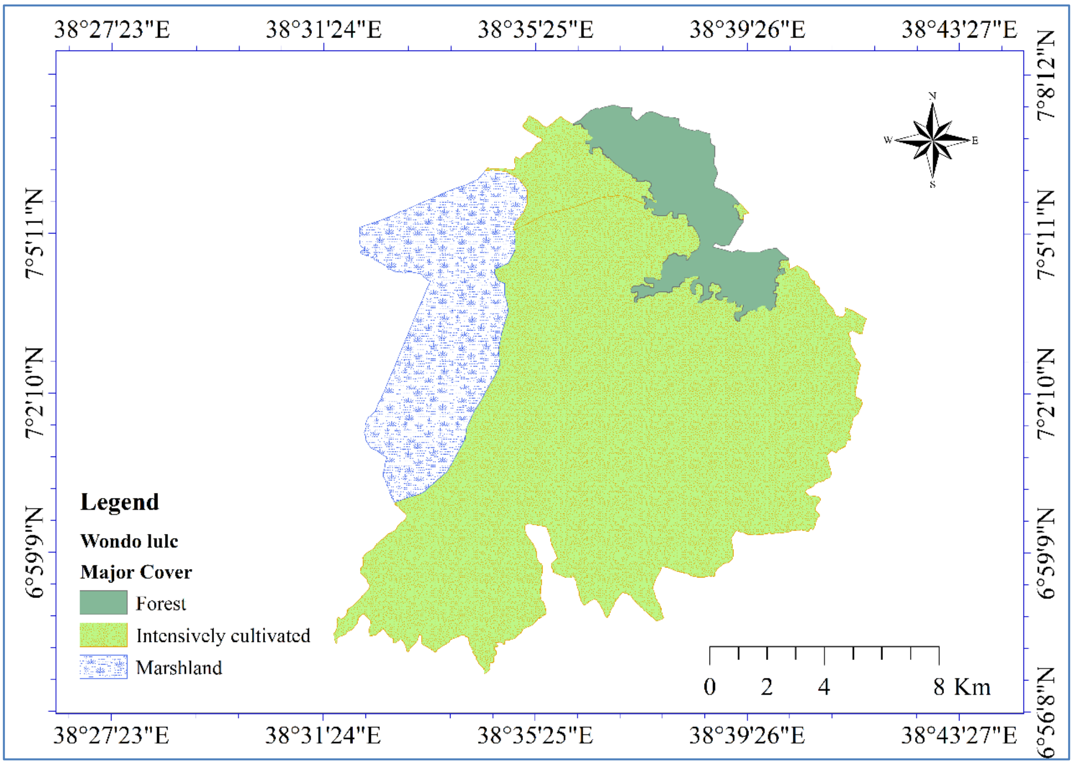


Figure 3.3 Wondogenet woreda land use land cover map 2020

3.1.5 Soils

According to the soils map of Ethiopia 2020, five major soil types have been identified for Wondogenet Woreda: Orthic Luvisols, Chromic Luvisols, Mollic Andosols, Vitric Andosols, and Vertosols. Chromic Luvisols are prominently found in the northern and central regions of the Woreda. Mollic Andosols are located in the southern and southeastern parts of the Woreda. These soils are derived from volcanic ash and are highly fertile, with a dark, organic-rich topsoil layer that supports diverse crop production. Orthic Luvisols are scattered throughout the Woreda, with a significant presence in the eastern areas. These soils are known for their well-developed horizons and moderate fertility, making them suitable for various crops. Vertosols are mainly found in the southwestern region.

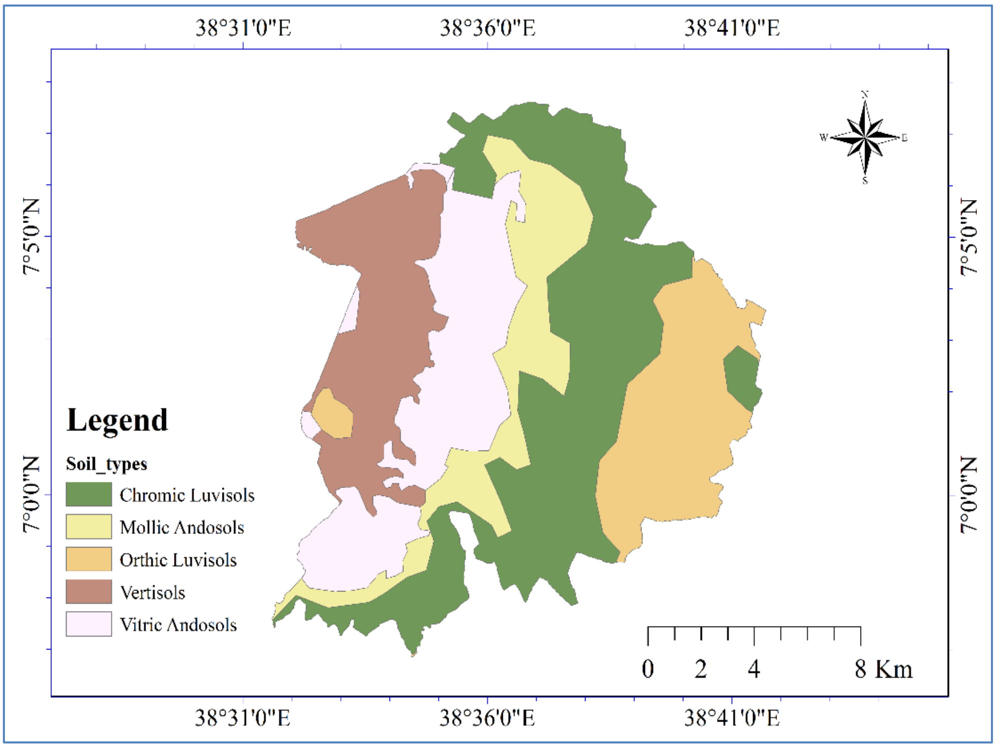


Figure 3.4 Types of soil in Wondogenet woreda (source: Ethio-soil map, 2010)

3.2. General description of the schemes

The irrigation schemes are categorized as small-scale, medium-scale, and large-scale based on their area coverage. In my study, particular emphasis is placed on small-scale irrigation schemes. The two selected schemes feature masonry-type weirs. Field observations revealed that the diversion weirs in both schemes are functional but have failed components.

3.2.1 Rassa small scale irrigation scheme

The Rassa irrigation project is primarily located in Aruma Kebele, Wondogenet District, North Sidama Region. The irrigation development project was undertaken on the Rassa River, with the intake structure specifically situated at an altitude of about 1812.50 meters above sea level and geographical coordinates of 6°57'42.26"N and 38°34'20.6"E. The project site is located 22 km from the regional town of Hawassa and 13 km from the Woreda capital, Chuko. This site is accessible via the Chuko Town to Hawassa asphalt road. From the district capital, the first 8 km is an asphalt road, 3 km is a dry weather road, and the remaining 2 km is a footpath. The implementation of this irrigation scheme project started in 2017 and was completed in 2018 G.C. It is a gravity-type irrigation system. The headwork was designed to divert 220 lit/sec to irrigate 113 hectares of land for 318 household beneficiaries. The main canals have a design discharge of 95 lit/sec over 24 hours and a total length of 1.65 km. The longitudinal slopes of the main canal are 1 m/km and 8 m/km, depending on the existing canal bed level of the selected traditional main canal.

3.2.1.1 Drainage pattern of Rasa river

The Rasa River flows from east to northwest through Melga Woreda, eventually reaching the Cheleleka wetland. From there, it continues to Tikur Wuha before joining Lake Awassa. The following Figure shows the drainage patterns of Rasa watershed.

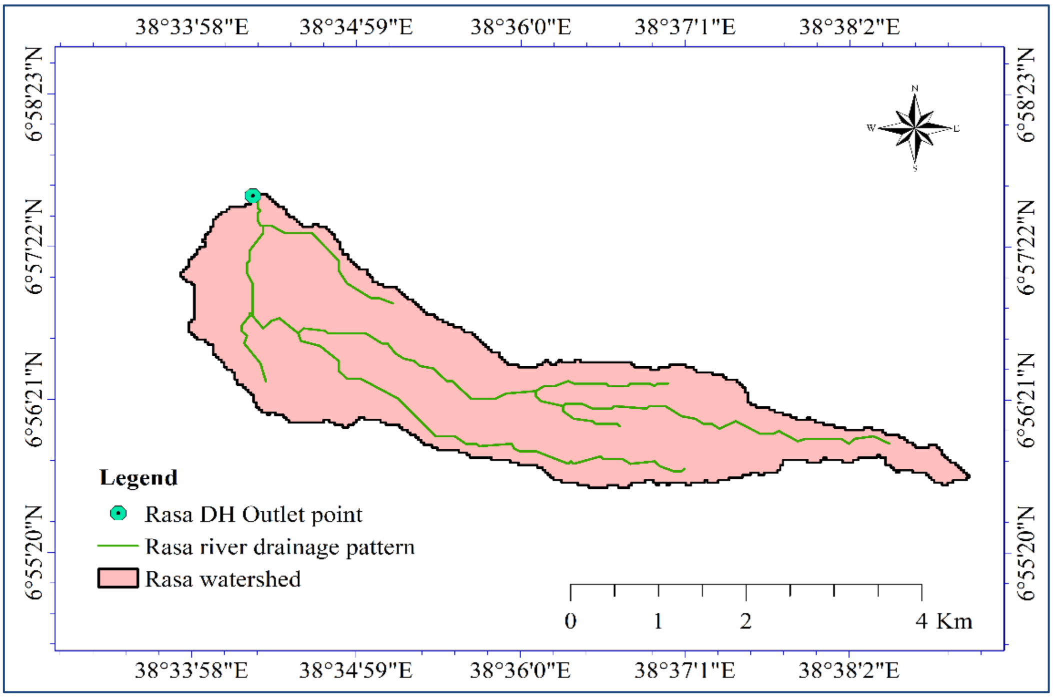


Figure 3.5 Rasa watershed drainage pattern (source: Author, 2024)

3.2.2 Wodesa small scale irrigation scheme

The Wodesa Irrigation development Project is situated in Wondo Genet Woreda, within the Baja Fabrika and Eddo Kebeles of the North Sidama Region. The project is based on the Wodesa River, with the intake structure positioned at an altitude of approximately 1787.28 meters above sea level and geographical coordinates of 6°58'56.26"N and 38°36'20.6"E. It is located 21.1 km from Hawassa, the regional capital, and 6.7 km from Chuko, the woreda capital. Of these distances, the first 19.0 km from Hawassa and 4.6 km from Chuko are on asphalt roads, while the remaining 2.1 km is on an all-weather gravel road. The implementation of this irrigation scheme began in 2019 and was completed in 2020. The system is a gravity type irrigation system, designed to divert 274 liters per second to irrigate 134 hectares of land. Of this area, 64 hectares are allocated for smallholder irrigation beneficiaries, and the remaining 70 hectares are managed by ELFORA Agroindustry PLC. The designed discharge for the main canals is 221 liters per second, with a total length of 1.52 km. The longitudinal slopes of the main canal have been designed to follow the existing canal bed level, with minimal cut and fill based on the current bed level of the selected traditional main canal.

3.2.2.1 Drainage pattern of Wodesa river

The Wodesa River that drains from southeast to northwest direction found in Wondogenet and Melga woreda. The main river that characterizes this watershed is Wodesa River, which encompasses different tributaries. The following Figure shows the drainage patterns of Wodesa watershed.

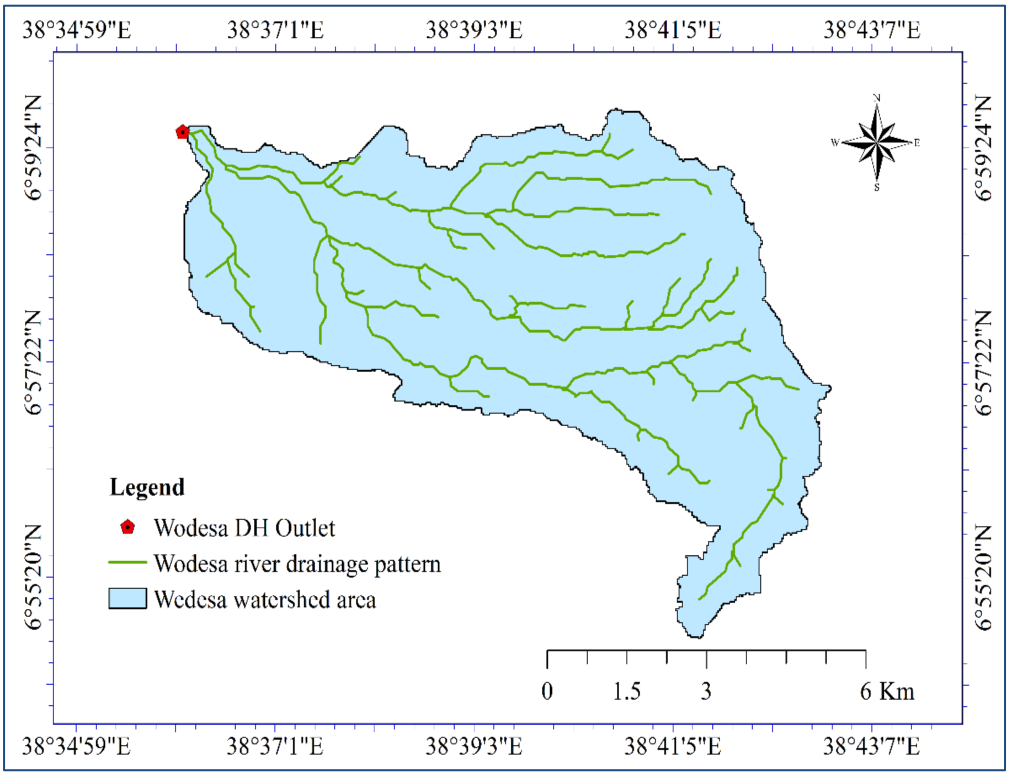


Figure 3.6 Wodesa watershed drainage pattern (source: Author, 2024)

3.3. Sources and procedure of data collection

3.3.1 Primary data and their sources

To achieve the objective of the study primary and secondary data was used. Primary data were collected directly from the field observation, in addition damage of weir components, the problem of flooding upstream and downstream of the structures, erosion of guide bank, under sluices, sedimentation of the head work structures, siltation of the main canal, and other relevant data are collected in the field survey.

a. Field observation

A field visit was conducted to assess the site conditions of the weir, focusing on issues such as siltation, flooding, scouring, damage to the weir and canal components, river bank stability, and the condition of protection structures both upstream and downstream. The visit also evaluated the thickness and length of the upstream and downstream protection works.

Additionally, the operational effectiveness of these structures was reviewed. Photographs of the weir's current condition, taken with a mobile camera during the field observation, are presented in Appendix D, Figure 12.

b. Field measurements

Trips to the sites were made to collect important data for accurately describing the actual ground conditions of the schemes at their locations. The field visits and measurements were undertaken to evaluate the performance of the diversion headwork. Direct measurements of the existing diversion headwork structures, such as weir dimensions, under-sluice dimensions, canal head regulator dimensions, upstream apron and pile lengths, downstream apron and pile lengths, water levels upstream and downstream, and silt levels upstream, were conducted. Geometric data, including river cross-sectional data and river center data, were collected using a total station and GPS. These data were used to design the selected diversion for the study, to identify knowledge gaps, and to suggest alternative designs or proper maintenance and repair.

3.3.2 Secondary data, sources and quality assurance

In addition to primary data collection, secondary data were gathered from various sources. Daily rainfall and temperature data were obtained from the National Meteorology Agency, while streamflow data were sourced from the Ministry of Water and Energy. Design documents for irrigation schemes were collected from the Sidama Regional State Irrigation Development Agency. Land use/land cover (LULC) and soil type data were provided by the Ministry of Water and Energy GIS department, and the digital elevation model (DEM) was obtained from the Shuttle Radar Topography Mission (SRTM) via https://earthexplorer.usgs.gov/. Furthermore, research publications, books, and other relevant documents, both published and unpublished, were reviewed to enhance the study's information base.

3.4. Materials

The tools that were used in this study included a tape meter, leveling instruments, a hand GPS, a phone camera, ArcGIS 10.3.1, HEC-HMS software, Flow master, an Excel spreadsheet, DEM 30x30, a hydrologic soil group map. The data required to accomplish this study included meteorological data, streamflow data, feasibility study design documents, and catchment characteristic data such as LULC and soil type data. These data were collected.

3.5. Methods of data analysis

3.5.1 Hydrological analysis

Rainfall is the primary source of water for river flood discharge in high rainfall regions. There are three stations near the study area. Out of these, two stations were selected for consistency testing using Thiessen polygons in ArcGIS to calculate the design rainfall for estimating river hydrology.

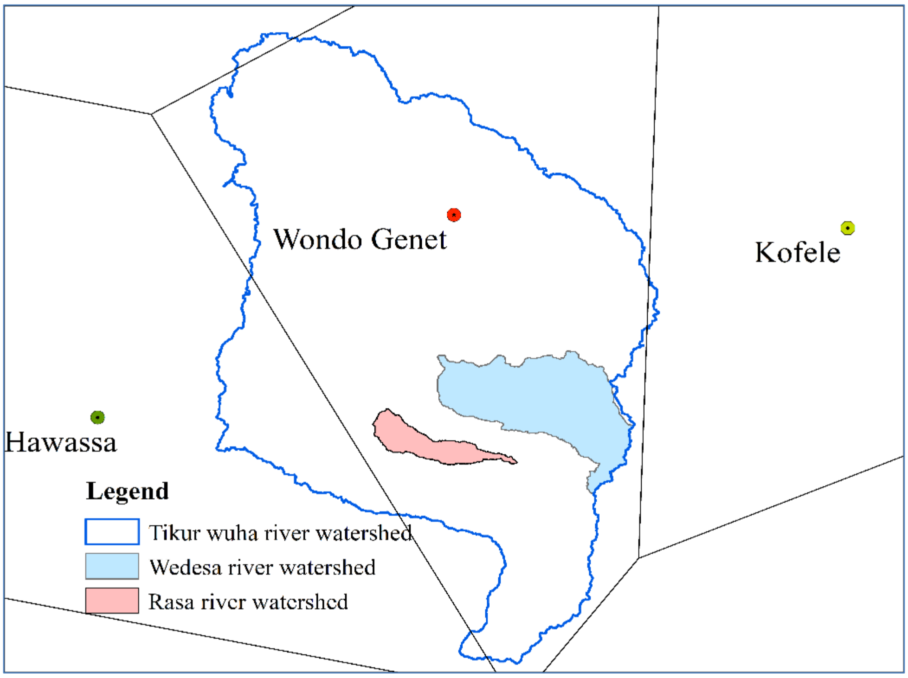


Figure 3.7 Thiessen polygon for nearest rainfall station

Table 3.1 List of rainfall gauge for study

S. N	Station	Latitude (x)	Longitude (y)	Altitude	Data range	Remark
1	Hawassa	38.483	7.065	1694	1990- 2022	In study area
2	Wondo Genet	38.615	7.082	1742	1990- 2022	In study area

3.5.2 Rainfall data quality test

3.5.2.1 Filling missing data

The rainfall data missing from the Hawassa station accounted for approximately 8.9%, while the missing rainfall data from the Wondo Genet station amounted to 10.87% over 33 years of records. The missing rainfall data for the Hawassa station was filled using the Arithmetic Mean method, and the missing data for the Wondo Genet stations was filled using the Invers Distance Method, all utilizing Microsoft Excel 2019.

3.5.2.2 Homogeneity test

To check the homogeneity of the rainfall data, the mean monthly rainfall records from the three nearest selected meteorological stations Hawassa, and Wondogenet were described in

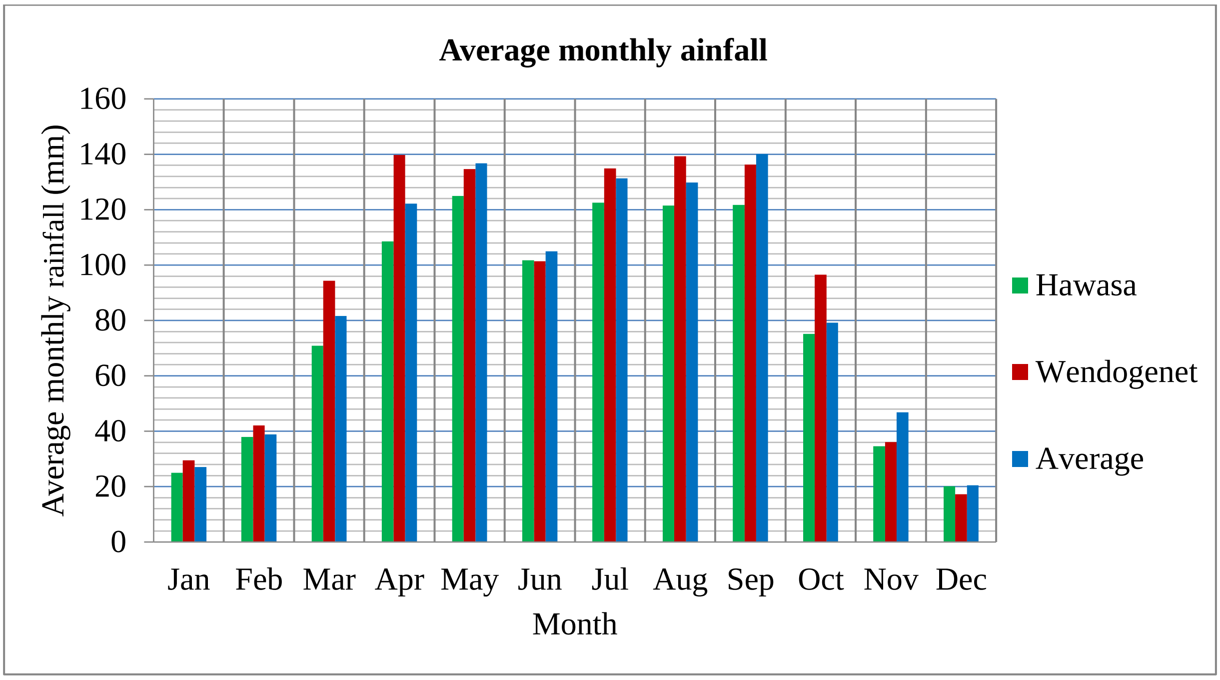


Figure 3.8 Average monthly rainfall of two station

As it was observed from the above figure 3.8 Rainfall data of two station has similar rainfall trend. Therefore, it is possible to use rainfall data for further analysis.

3.5.2.3 Consistency testing of rainfall data

Estimating missing precipitation is one problem that hydrologists need to address. Second problem occurs when the catchment rainfall at rain gages is inconsistent over a period of time and adjustment of the measured data is necessary to provide a consistent record. A consistent record is one where the characteristics of the record have not changed with time. Inconsistency may result from: change in gauge location, exposure, instrumentation, or an observational procedure is not real and on time. To overcome the problem in consistency a technique most widely applied called double mass curve was used. Double-Mass Curve (DMC) analysis is a graphical method for identifying or adjusting inconsistencies in a station record by comparing its time trend with those of other stations nearby (ERA, 2013). Double mass curve plot made for two stations. The curve is a plot of rainfall record of a station and cumulative rainfall collected at a gauge where measurement condition may have changed significantly against the average of the cumulative rainfall for the same period of record collected at several gauges in the same region. The use of the double-mass curve for checking the consistency of precipitation records is explained by the following example in which the annual records of two precipitation stations. First the annual precipitation data for each year are tabulated and then cumulated in chronological order. The cumulative precipitation for each station is then plotted against the cumulative precipitation of the pattern. Double mass curve plot made for Hawassa and Wondogenet metrological stations shown in figure below. From the double mass curve figures the stations were consistent to each other.

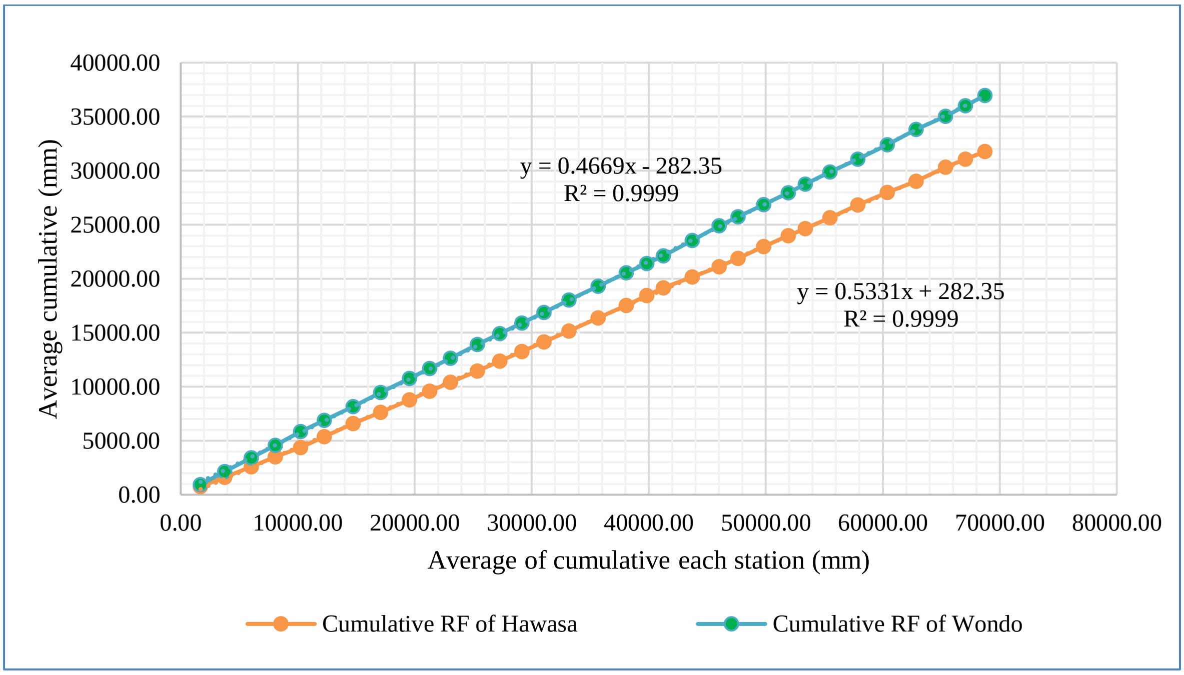


Figure 3.9 Double mass curves of Hawassa and Wondogenet rainfall stations

The mass curve shows a strong direct correlation between the average cumulative rainfall of the two nearest stations and the individual cumulative rainfall. As shown in Figures 3.9 the mass curve analysis revealed a solid direct link between the cumulative rainfall records at the Hawassa and Wondogenet rain gauge stations and the average cumulative rainfall of the two stations. This indicates that the rainfall data at both the Hawassa and Wondogenet stations are reliable. Consequently, there was no significant change in the slope of their respective plots, and the correlation coefficient of the two stations was consistent. Although the Wondogenet station is the nearest to the study area, the average areal rainfall of both Wondogenet and Hawassa was selected to analyze the design rainfall for this study.

3.5.2.4 Outliers

Out liars are data points that depart significantly from the trend of the remaining data. Sometimes the significant change may occur in and around a particular rainy station. Such a change occurring in a particular year might start affecting the rain gauge data being reported from that particular station. After several years, it may be felt that the data of that station is not giving consistent rainfall values. The depletion of high and low out lair can one-sided 10% significantly affect the magnitude of statistical parameter computed data. Especially for small number data to find lower and higher out lairs. Procedures for treating out lairs required judgment involving both mathematical and hydrologic considerations (Subramanya, 2008).

According to the U.S water resource council (1981), if the station skew is greater than +0.4, tests for higher outliers are considered first; if the station skew is less than -0.4, tests for lower out lairs are considered first. Where the station skew is between + 0.4, tests for both high and low outliers should be applied before eliminating any out lairs from the data set.

As ERA (2013), the following equations are used to detect outliers.

Lower outliers (X_L) = X_{mean} - KN * δX…………………………….……………..….3.1

QL = 10^{XL} ………………………………………………….…………………..3.2

Higher outliers (X_H) = X_{mean} + KN * δX……………………………...………...3.3

QH = 10^{XH}……………………………………………………………………3.4

Where, X_{mean}= Mean value of logarithm transferred data.

KN = Factor taken from table which contains one side 10% significance

δX = Standard deviation of logarithmic transferred data.

QL = Lower outlier threshold in log units

QH = Higher outlier threshold in log units

3.5.2.5 Check for variance

After checking the outliers, the data should be checked for variability. For variability the formula used is (ERA, 2013).

$$Relative, \alpha = \frac{\delta}{mean*\sqrt{N}} * 100 < 10\% \quad \text{...3.5}$$

Where, σn-1 = Standard deviation,

N=Number of records, α=standard error

3.5.2.6 Areal rainfall depth calculation

Areal rainfall is the average depth of rainfall over a specific area, usually derived from point rainfall data collected at various rain gauge stations within the area (Chaw, 1988).

There are different methods to calculate areal rainfall, each varying in complexity and accuracy depending on the distribution of gauges and topography of the region for this study

Thiessen Polygon Method was used because this method accounts for the non-uniform distribution of rain gauges by assigning weights based on the area of influence of each rain gauge station (Subramanya, 2008).

Equation: Thiessen Polygon Method

$$\text{Pavg} = \frac{\sum_{i=0}^{n} \text{Pi} * \text{Ai}}{\sum_{i=0}^{n} \text{Ai}} \quad \text{……………………………………………………………….3.6}$$

Where: Pavg = Average areal rainfall, Pi = Rainfall at gauge, Ai = Area of the Thiessen polygon around gauge and n = number of gauges.

3.5.3 Maximum rainfall frequency analysis

The daily maximum rainfall data obtained from the National Meteorological Agency were analyzed using various methods, including Normal, Log Normal, Log Pearson Type III, Pearson Type III, and Gumbel distribution methods. The annual maximum average areal rainfall data series of thirty-three (33) years for the Wondogenet and Hawassa stations were used to predict the possible design flood magnitudes for future 5, 10, 25, 50, and 100 years.

3.5.3.1 Goodness of fit test

The fitted probability distribution for the available sample data was evaluated using the most commonly used goodness-of-fit (GOF) tests. The D-Index test is considered to be one of the best measures of goodness-of-fit in many studies (Chaw, 1988). A higher D-Index value indicates a better fit for the rainfall frequency distribution method. In this study, the D-Index test was used to determine the best statistical distribution for estimating peak rainfall.

According to Subramanya, (2008), the D-index for the comparison of the fit of various distributions is summarized as follows

$$\text{D-Index} = \frac{1}{xm} * (\sum_{i=1}^{6} ABS(Xi - Xi') \quad \text{……………………………...…………..3.14}$$

Where Xi and Xi' are the i^{th} highest observed and computed values for the distribution respectively.

3.5.4 Stream flow analysis

In my study area watershed, there is no gauging station; however, the Tikur Wuha Nr Awassa Bridge gauging station, located downstream of the diversion site, can be used to transfer hydrological parameters for estimating river runoff. Using the rainfall-runoff model (HEC-HMS), the runoff volume of the river at the weir site was estimated. The river flow data time series was used to adjust parameters for catchment water balance calibration and validation, with flow records extending from 1991 to 2007.

3.5.4.1 Filling missing data

To fill the data gaps, the Inverse Distance Method was applied to estimate the missing flow data for the Tikur Wuha station near Awassa Bridge. The missing data accounted for 18.2% of the total records over 15 years.

3.5.4.2 Mean monthly flow of tikur wuha river

The mean monthly flow hydrograph for Tikur Wuha near Awassa Bridge is presented in the figure below.

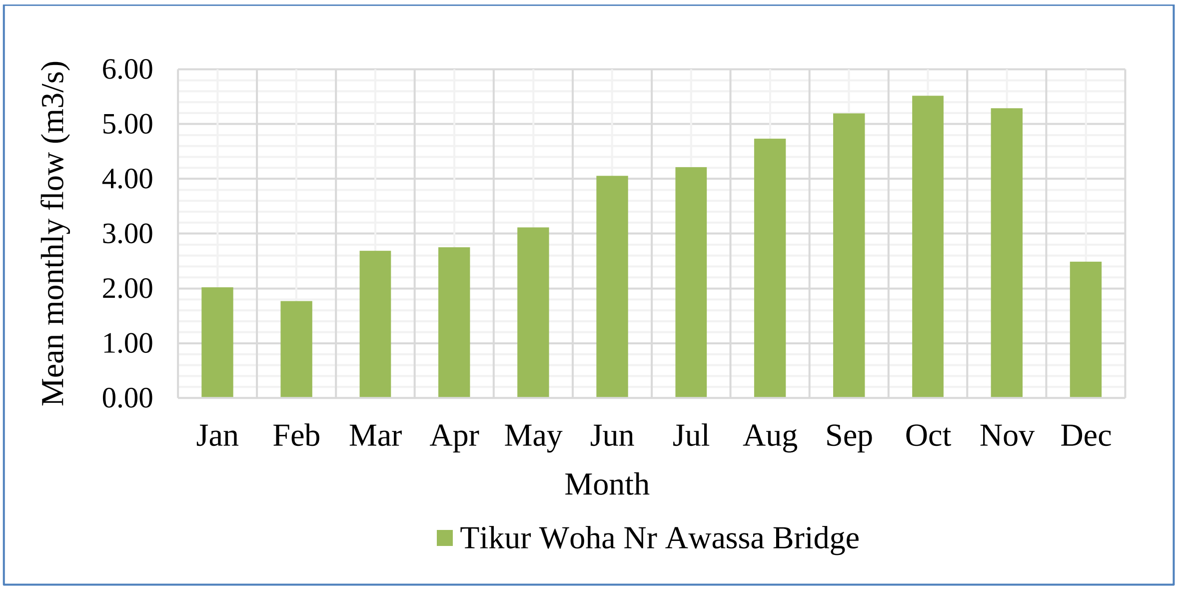


Figure 3.10 Mean monthly flow hydrograph of Tikur Wuha river

3.5.4.3 Consistency testing of streamflow data

When the characteristics of the streamflow record change over time, it is necessary to adjust the measured data to provide a consistent record. This adjustment is crucial because changes in observation procedures, the exposure of the gauge, or the need to relocate the gauge can cause inconsistencies in the record. The double mass curve method was used to check for consistency (Chaw, 1988).

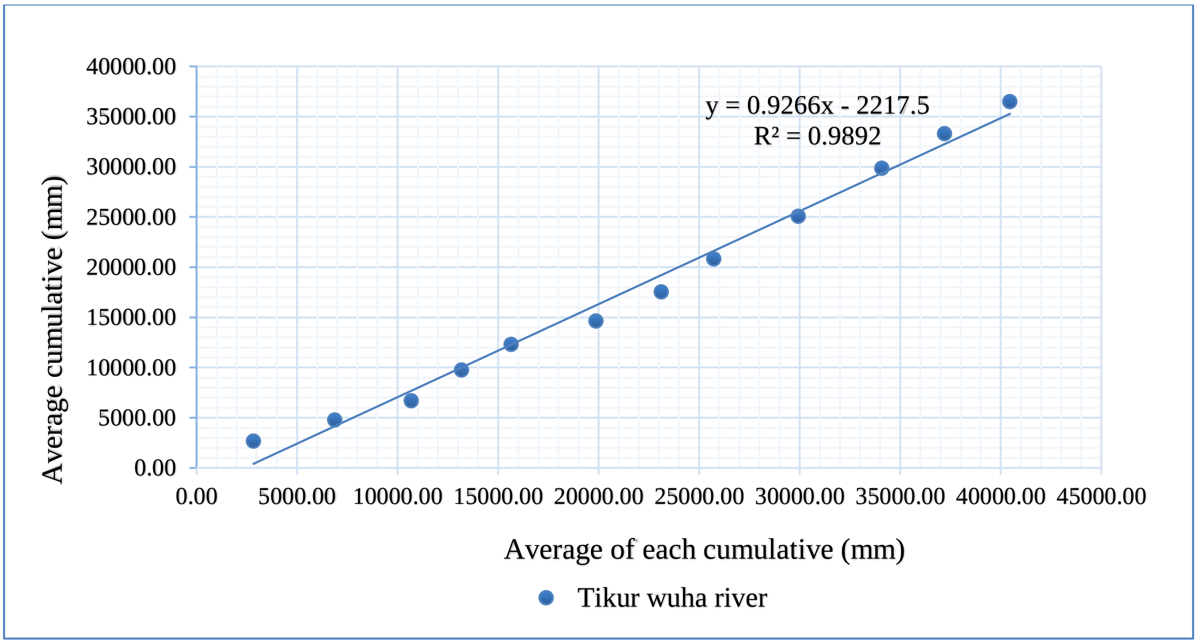


Figure 3.11 Double mass curve of Tikur wuha river

The mass curve analysis shows a strong correlation between the average cumulative rainfall and cumulative flow at Tikur Wuha river stations, as illustrated in Figure 3.11. The high R^2 values confirm the consistency of flow data, indicating no significant changes in measurement procedures or gauge locations over time. This consistency ensures reliable hydrological analyses, supporting the suitability of the data for modeling and decision-making. The double mass curve analysis further confirms the homogeneity of streamflow records, validating their use for hydrological studies in the region (Subramanya, 2008).

3.5.5 Hydrologic models

In the design of hydraulic and irrigation structures in ungauged basins, estimating the peak flow within a specified frequency is of primary importance. To estimate the magnitude of peak floods, hydrologic models may be used depending on the availability of data. Hydrologic models are simplified, conceptual representations of components of the hydrologic cycle. They are primarily used for hydrologic prediction and to enhance understanding of hydrologic processes (Bitew, 2019).

In this study, HEC-HMS model was used to determine the peak direct discharge of the river. Due to its semi-distributed nature, its structure is more physically-based than that of a lumped model and is less demanding on input data compared to a fully distributed model. The discharge obtained from this analysis was used in the Hydraulic analysis of the Rasa and Wodesa diversion weirs.

3.5.5.1 Methodology for using HEC-HMS model

The HEC-HMS model was selected to simulate the rainfall-runoff processes in the Tukur Wuha River watershed. The optimized parameters obtained from the calibration of this gauged watershed were subsequently transferred to the ungauged Rasa and Wodesa River watersheds, which are sub-basins of the Tukur Wuha river watershed, using the regionalization technique.

The model setup incorporated observed average areal rainfall data from the Awasa and Wondogenet stations, streamflow gauge data from the Tukur Wuha River, and physical watershed characteristics derived from Digital Elevation Models (DEM) and land use maps. The SCS-CN method was employed to estimate runoff, with Curve Number (CN) values determined based on soil type and land use classifications. Routing was carried out using the Muskingum routing method, while SCS Unit Hydrograph was used for runoff transformation.

The model calibration process involved optimizing key hydrological parameters to minimize the differences between observed and simulated streamflow. Parameters such as Curve Number, Initial Abstraction, Time of Concentration, and Muskingum routing coefficients were adjusted iteratively using manual and automated optimization techniques provided within the HEC-HMS framework (Aliye, 2020).

A sensitivity analysis was conducted to identify the most influential parameters affecting model outputs. Key parameters analyzed included Curve Number, Initial Abstraction, and Time of Concentration. The analysis demonstrated that CN values and Time of Concentration had the most significant impact on simulated results, making them critical during the calibration process (Cunderlik, 2003).

The model's performance was evaluated during both the calibration and validation periods using established metrics, including Root Mean Square Error (RMSE), Nash-Sutcliffe Efficiency (NSE), Coefficient of Determination (R^2), and Percent Bias (PBIAS) (Bitew, 2019).

3.5.6 Hydraulic analysis

The hydraulic analysis includes surface and subsurface hydraulic analysis to determine the weir shape and height, as well as its clear waterway, discharge over the weir, head over the weir, circumference, flood and energy levels, afflux, scour depth, and seepage flow characteristics. To design a weir, all external forces acting on the weir were calculated (Ertiro, 2017). Water flow length or water way is calculated from Lacey's perimeter formula: $L = 4.75\sqrt{Qd}$………………………..……………………………….…..……3.14

Where; L is length of water way in meter and Q_d is design discharge in (m^3/s) Scouring depth of the flow (R): $R=1.35(\frac{q^2}{f})1/3$………………………………..………....…………3.15

Where; q is the discharge per unit width of the river, *f* is the silt factor.

Table 3.4 Silt factor of river material

Type of Reach	**Mean value of "f"**
Large boulders	24
Medium boulders	15
Small boulders	12
Heavy gravel	9
Coarse gravel	4.75
Coarse bajri and sand	2.75
Fine bajri and sand	1.75
Coarse sand	1.50
Medium sand	1.25
Standard silt	1.00

Source: - (MoA, 2018)

The capacity for passage of peak flood of the diversion headwork will be done using equation 3.16 (Arora, 1996).

$$Qd = C_{d*}(L - n*K*H_e) *H_e^{1.5} \text{………………………………………………………}3.16$$

Where: Cd=coefficient of discharge, L= net length of the weir, Qd=discharge over the weir, n=number of end contractions, K=coefficient of end contraction≈ 0.1and He=total head over the weir.

3.5.7 Seepage analysis

The hydraulic gradient of seepage under the weir foundation can be determined using Bligh's creep theory, Lane's weighted creep theory, and Khosla's theory. Among these, Khosla's theory is preferred for calculating the creep length and checking the exit gradient at the downstream apron. This is because it provides a more detailed and realistic analysis of seepage behavior under hydraulic structures. Khosla's theory accounts for intermediate cutoffs, the thickness of impervious floors, and pressure variations, offering a robust method for calculating the exit gradient, which is crucial in preventing piping and uplift failure. Therefore, Khosla's approach is more suitable for modern and complex hydraulic designs, such as diversion weirs (Garg, 2005).

The exit gradient for given floor length, b with vertical cut of depth d is given as

$$GE=\left(\frac{H}{d}\right)*\left(\frac{1}{\pi\sqrt{\lambda}}\right) \quad \text{...} 3.17$$

$$\lambda=\frac{1+\sqrt{1+\alpha^2}}{2}, \alpha=\frac{b}{d} \quad \text{...} 3.18$$

The thickness of the impervious floor was determined by uplift pressures, which can be calculated using the method suggested by Khosla's for various stages of flow, the thickness of the floor in the jump through should be explored. However, for other portions of the downstream floor, the maximum uplift would occur when the water level on the upstream side was at pond level and there was no tail-water (or flow) on the downstream side (Asawa, 2008).

Table 3.5 The safe exit gradient based on soil type.

Soil type	**Safe exit gradient**
Shingle	1/4 to 1/5 (0.25 to 0.2)
Coarse sand	1/5 to 1/6 (0.2 to 0.17)
Fine sand	1/6 to 1/7 (0.17 to 0.14)

Source: (Garg, 2005)

The critical exit gradient of the weir foundation was calculated as the following equation (Khassaf, 2008).

$$icr=\frac{\gamma sat-\gamma w}{\gamma w} \quad \text{...} 3.19$$

The factor of safety against uplift pressure due to piping of water through weir foundation was given as follow

$$FS=\frac{icr}{GE} \quad \text{...} 3.20$$

Where the value of Fs is equal or greater than 3 the weir floor was safe against uplift pressure due to piping of water under foundation of weir (MoA, 2018).

3.5.8 Structural analysis

In the structural analysis of the weir, the stability of both the weir body and the retaining wall was assessed against key failure mechanisms, including overturning, sliding, overstressing, and excessive contact pressure on the foundation. Each of these stability checks was performed to ensure the structure's safety under various loading conditions, with calculations based on the following critical points:

i. Overturning

When the overturning moment exceeds the resisting moment, overturning failure happens. As a result, overturning failure the weir is normally preceded by tension or crushing failure. If no tension at any point in the weir body or retaining wall it meets the maximum compressive stress does not exceed the permissible limit, a weir may be considered safe against overturning.

The factor of safety against overturning is one of the measures for stability of the structure.

$$F_{ot} = \frac{\sum M_r}{\sum M_O} \geq 1.5 \qquad (3.21)$$

Where; Fot is factor of safety against overturning, Mr is the resisting moment and Mo is disturbing or overturning moment (Garg, 2005).

ii. Sliding

When the weir and retaining wall lied's over its base or when part of the weir above the horizontal plane slides over that plane, the sliding failure happens. The weir should be designed so that the sliding forces do not surpass the resisting force at any horizontal section or at the base to prevent this failure (Ertiro, 2017).

The safety factor against the sliding is given as

$$F_S = \frac{\mu \sum V}{\sum H} \geq 1.5 \qquad (3.22)$$

Where, μ is sliding coefficient usually varies from 0.6 to 0.75, V is vertical force on the structure H is horizontal force on the structure (Garg, 2005).

iii. Over-stress;

$$e \leq \frac{B}{6} \qquad (3.23)$$

$$e = \left| X - \frac{B}{2} \right| \qquad (3.24)$$

$$X = \frac{\sum Mn}{\sum V} \ldots\ldots 3.25$$

Where: ΣM_n= Net moment (=ΣM_R-ΣM_{OT}), e= Eccentricity developed and B is base width of weir or retaining wall (Garg, 2005).

iv. Stability against contact pressure:

The contact pressure (Stress) on the foundation at the toe or heel of the weir body should be less than the allowable bearing capacity of the foundation material. Maximum and minimum compressive stress developed at toe of the weir is given by:

$$\sigma_{maz} = \frac{\sum F_v}{B} \times \left(1 + \frac{6e}{B}\right) \ldots\ldots 3.26$$

$$\sigma_{min} = \frac{\sum F_v}{B} \times \left(1 - \frac{6e}{B}\right) \ldots\ldots 3.27$$

3.6. Study design

The study has been conducted by using both descriptive and experimental study design. This study was used to know the design practice and performance of constructed diversion weir and recommend the design procedures based on national and international guide line values of design assumption.

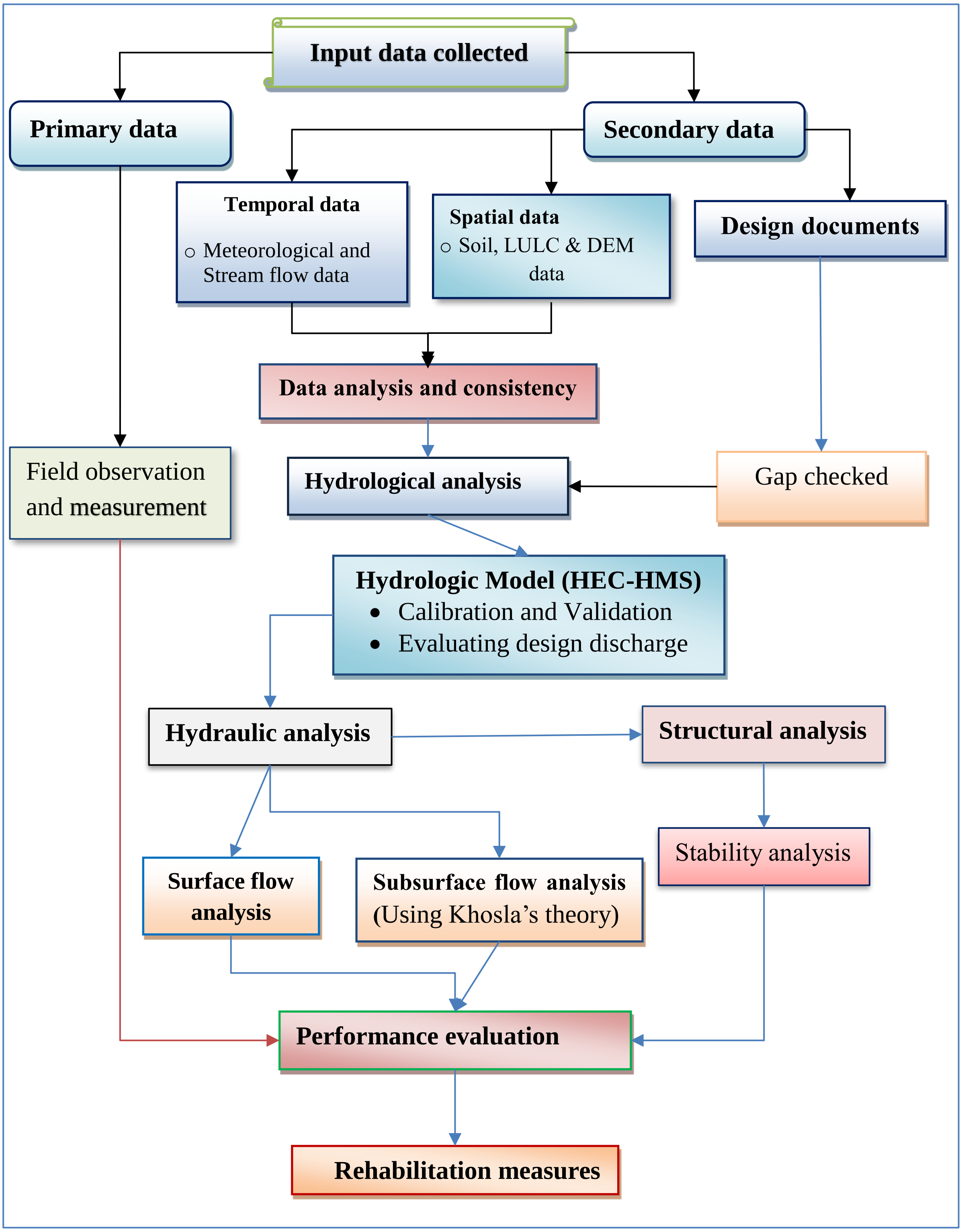


Figure 3.12 Conceptual frame work adopted in the study

4 RESULTS AND DISCUSSIONS

4.1. Results from field visit and observations

In general, based on field observations, the selected diversion headwork structures are currently facing significant structural and managerial issues, as outlined in Table 4.1.

Table 4.1 Current status of the two diversion headwork structures

S/N	Current status of DHWSs	Acquired DHWs
1	Soured downstream	Rasa
2	Seepage problems	Rasa
3	Sedimentation problems	Rasa and Wodesa
4	Damaged u/s & d/s aprons	Rasa
5	Damaged u/s & d/s cutoff piles	Rasa
6	Damaged intake & under sluice gates	Rasa
7	Damaged retaining wall	-
8	Collapse weir proper	-
9	Main canal siltation	Rasa and Wodesa
10	Weir alignment problems	Rasa

The results in Table 4.1 indicate that the current assessment of Diversion Head Work Structures (DHWSs) at Rasa and Wodesa reveals several issues. One out of the two structures are experiencing problems such as scouring downstream, seepage, damaged aprons, cutoff piles, intake gates, and alignment issues. Sedimentation head work and siltation in the main canal are significant concerns, affecting both Rasa and Wodesa. However, the weir structure and retaining walls at both sites are intact, with no reported issues.

4.2. Actual dimensions of headwork structures results

The actual dimension results of the existing diversion headwork structures are presented in Table 4.2 as follows.

Table 4.2 Actual dimensions of headwork structures

S/no	Parameters	Units	Rasa DHWS	Wodesa DHWS
1	Weir dimensions			
1.1	Height (H)	m	0.4	1
1.2	Length (L)	m	7	10
1.3	Top width(b)	m	1	1.5
1.4	Bottom width (B)	m	1.8	3.6
2	Under sluice section			
2.1	Width(w)	m	0.5	1
3	Impervious length			
3.1	Total impervious length	m	8.8	14.8
3.2	D/s concrete floor length	m	4	8
4	Cutoff piles			
4.1	U/s cutoff pile	m	1	1
4.2	D/s cutoff pile	m	1.75	2
5	Wall Dimension			
5.1	Top Width	m	0.5	0.5
5.2	Bottom width	m	1.75	1.7
5.3	U/s Height of wing wall	m	3	4.4
5.4	D/s Height of wing wall	m	2.5	2.9
5.5	Foundation depth	m	1	1

4.3. Hydrological analysis

4.3.1 Analysis of meteorological station

From the Thiessen polygons in Section 3, Figure 3.7, it is evident that there are two stations near the study area. The corresponding areal coverage is shown in the table below. Therefore, the average areal rainfall from these two stations was used for this study. Detailed analytical results are presented in Appendix A, Table 1.

Table 4.3 Areal coverage of rainfall station from thiessen polygon

S/n	Station name	Area (hectares)
1	Awasa	7,100
2	Wondogenet	61,500

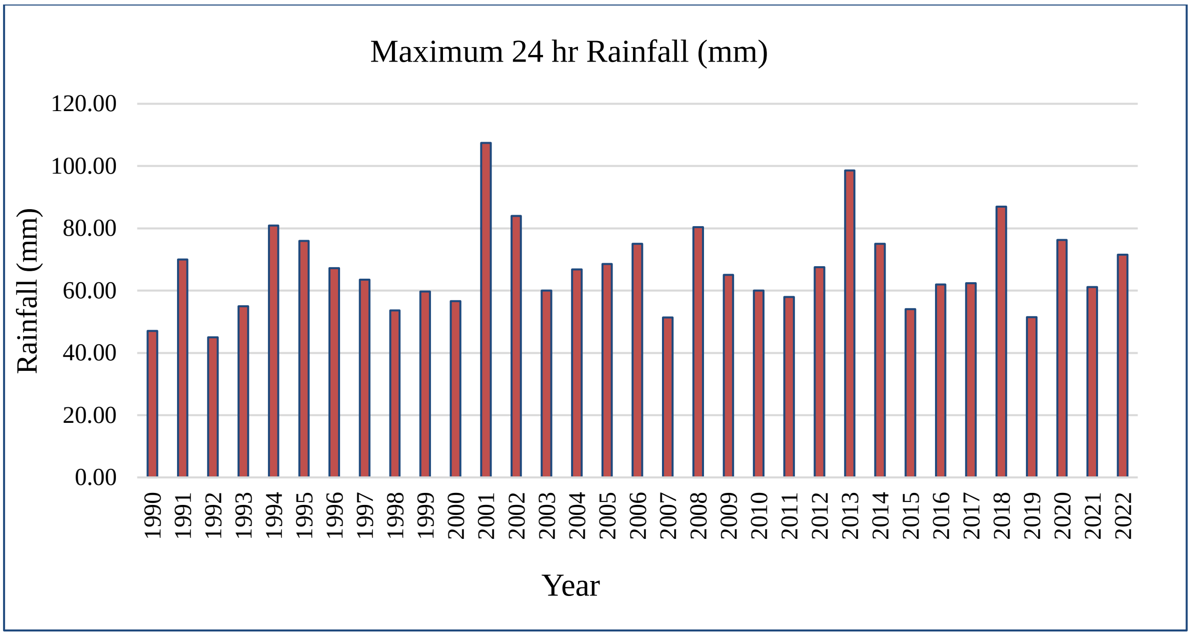


Figure 4.1 Annual maximum average areal rainfall of Wondogenet and Hawassa station

4.3.2 Rainfall data frequency analysis results

Table 4.4 Data quality test for average areal rainfall of Wondogenet and Hawassa station

S/n	Parameters	Average areal rainfall
1	Number of data (N)	33
2	Sum	2240.9
3	Mean (X)	67.91
4	Minimum	45
5	Maximum	107.4
6	Standard Deviation (SD)	15.24
7	Skewness Coefficient	1.018
8	Standard error of mean	2.65
9	Relative standard (RSD)	3.91%

Based on Table 4.3 The relative standard deviation of the average areal rainfall is less than 10%, indicating that the data set is fairly consistent with low variability. This consistency makes the data reliable for use in further hydrological studies, such as design rainfall calculations or flood forecasting.

Table 4.5 Determination of maximum and minimum rainfall out liar values

Outlier Test results	
Kn	2.6
Higher Limit: YH=Ymean+Kn*бy	2.06
Higher Limit	115.46
Lower Limit: YL=Ymean- Kn*бy	1.58
Lower Limit	38.17
Maximum data from time series	107.4
Minimum data from time series	45

Based on the outlier test results, the higher limit for the rainfall data is 115.46 mm, while the lower limit is 38.17 mm. The actual maximum and minimum values from the time series data are 107.4 mm and 45 mm, respectively. Since both the maximum and minimum data points fall within the calculated limits, there are no outliers in the dataset. This confirms that the rainfall data series is consistent and reliable for further analysis.

Table 4.6 Summer of design average areal rainfall analysis result.

Method of analysis	Expected design rain fall year				
	5	10	25	50	100
Gumbel's method	81.07	91.32	104.27	113.87	123.41
Gumbel's Extreme value method	78.87	87.79	99.06	107.42	115.72
Log Pearson Type 3 method	78.98	88.13	99.81	108.39	117.49
Log Normal method	78.98	88.13	99.81	106.29	117.49
Pearson Type 3 Distribution	80.36	88.20	97.01	103.06	108.37
Normal	80.73	87.44	94.60	99.22	103.37

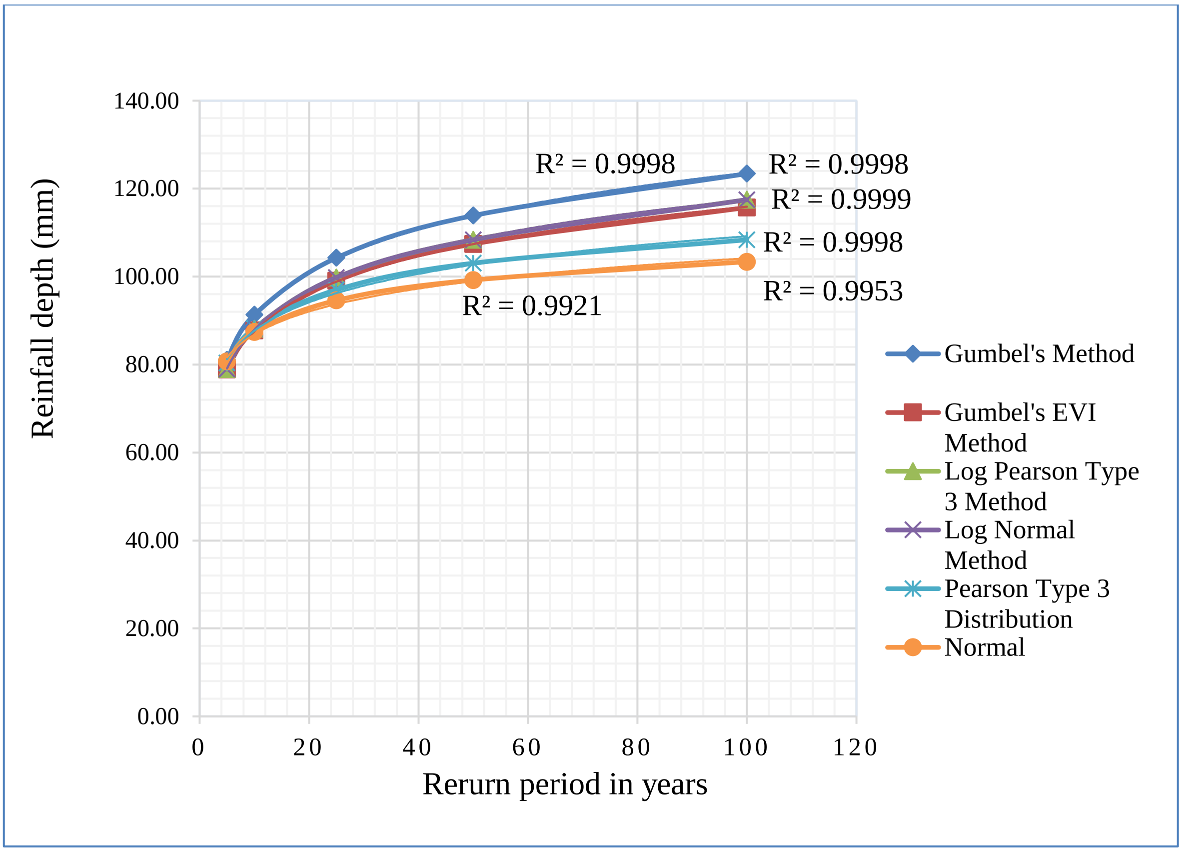


Figure 4.2 Fitting test of distribution method

Table 4.7 Fifty (50) Years expected rainfall result

Name	Method of analysis	50-year return period design rain fall (mm)
Average areal rainfall of Wondogenet and Hawassa station	Gumbel's Method	113.87
	Gumbel's EVI Method	107.42
	Log Pearson Type 3 Method	108.39
	Log Normal Method	106.29
	Pearson Type 3 Distribution	103.06
	Normal	99.22

Table 4.8 Goodness of fit statistic test results

Rank	XI	Normal	Log Pearson Type III	Log Normal	Pearson Type III	Gumbel extreme value	Gumbel
		XI -'XI'	XI -'XI'	XI -'XI'	XI -'XI'	XI -'XI'	XI -'XI'
1	107.40	10.69	3.93	8.19	10.09	4.62	2.63
2	98.60	11.84	8.92	11.01	11.51	9.24	3.80
3	87.00	10.09	9.08	10.11	9.90	9.25	10.83
4	84.00	1.00	1.18	1.56	0.91	1.26	24.23
5	80.00	0.10	1.08	1.03	0.09	1.10	28.93
6	76.30	1.66	0.11	0.48	1.63	0.13	34.21
Sum		35.38	24.31	32.38	34.12	25.60	104.63
Sum/Mean		0.52	0.36	0.48	0.50	0.38	**1.54**
Point rainfall		99.22	108.39	108.29	103.06	107.42	**113.87**
Design of areal rainfall				**113.87**			

Based on Table 4.8 and Figure 4.2, a higher D-Index value signifies a better fit for the rainfall frequency distribution method. Thus, the most reliable method identified in this study was the Gumbel method. The analysis from Table 4.7 shows that the Gumbel method produced a design rainfall value of 113.87 mm for a 50-year return period. This value was then used to calculate the areal design rainfall for the hydrological analysis of the Rasa and Wodesa small-scale irrigation diversion weirs.

4.3.3 Model sensitivity analysis

The sensitivity analysis of the HEC-HMS model results used to simulate the hydrological behavior of the Tikur Wuha River watershed. Key parameters analyzed include the SCS unit hydrograph lag time, curve number, initial abstraction, and Muskingum K and X values for routing within sub-reaches. These parameters, ranked by their sensitivity and influence on model performance, are summarized in Table 4.9.

Table 4.9 Parameters sensitivity rank based on objective function.

Parameter name	Description	Min	Max	Rank
SCS. Lag (min)	SCS unit hydrograph lag time	0.1	30,000	1
Curve number (AMC_II)	SCS curve number	35	99	2
Initial abstraction (min)	SCS Curve number initial abstraction	0.001	500	3
Muskingum, K (hr.)	Flood wave travelling time	0.001	150	4
Muskingum, X	Weighted coefficient of discharge	0.001	0.5	5
Muskingum reaches	Muskingum sub reaches	1	100	6

4.3.4 HEC-HMS model simulation result

The calibration and validation of the HEC-HMS model for the Tukur Wuha River Basin were performed by comparing the simulated streamflow with the observed flow at the main gauging station (Nr Awassa Bridge station). The graphical comparison of measured and simulated flows showed a good match during both the calibration and validation periods (Figure 4.4 and Figure 4.5). This indicates that the model accurately captured the observed streamflow trends in both periods. However, the total runoff volume and peak flow were slightly overestimated. This discrepancy may be attributed to the routing coefficients or the use of an hourly time step for simulation. Based on the calibration and validation results, the model's performance was evaluated using statistical indicators (Figure 4.6 and Figure 4.7).

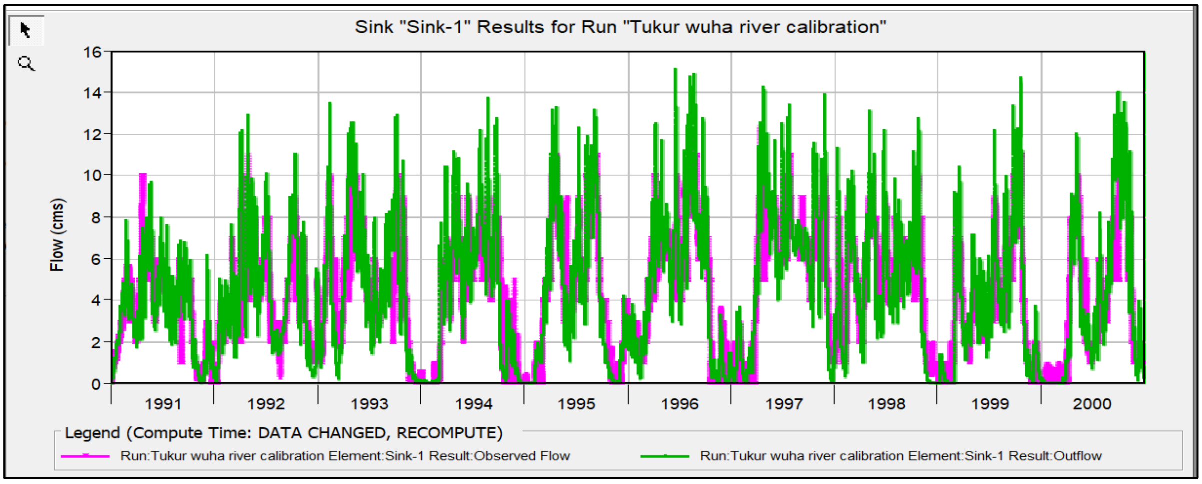


Figure 4.4 Simulated and observed stream flow after model calibration

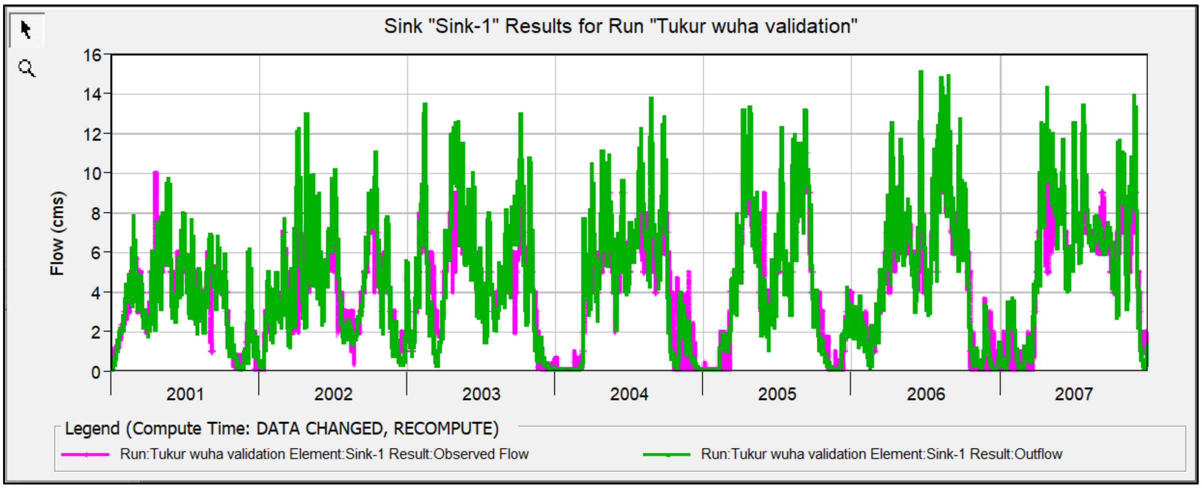


Figure 4.5 Simulated and observed stream flow after validation

Project: Tikurwuha river   Simulation Run: Tukur wuha river calibration

Start of Run: 01Jan1991, 00:00
End of Run: 31Dec2000, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: Basin model - Calb
Meteorologic Model: Meteorological mo - Calb
Control Specifications: Control specification - Calb

Show Computation Points: CP: Sink-1   Sorting: Hydrologic

Computation Point	RMSE Stdev	Nash Sutcliffe	Percent Bias	R^2
Sink-1	0.6	0.671	7.26	0.79

Figure 4.6 Daily streamflow model performance statistics result for calibration

Project: Tikurwuha river   Simulation Run: Tukur wuha validation

Start of Run: 01Jan2001, 00:00
End of Run: 31Dec2007, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: Basin model - Vald
Meteorologic Model: Meteorological mo - Vald
Control Specifications: Control specification - Vald

Show Computation Points: CP: Sink-1   Sorting: Hydrologic

Computation Point	RMSE Stdev	Nash Sutcliffe	Percent Bias	R^2
Sink-1	0.6	0.672	7.41	0.80

Figure 4.7 Daily streamflow model performance statistics result for validation

The result in figure 4.6 and figure 4.7 indicated that the calibration and validation results of the HEC-HMS model applied to the Tikur Wuha River demonstrate good performance overall. During the calibration period (1991–2000), the model produced a root mean square error (RMSE) standard deviation of 0.6, indicating a low deviation between observed and simulated streamflow. The Nash-Sutcliffe Efficiency (NSE) value of 0.671 suggests a good model fit, with values closer to 1 indicating better performance. The Percent Bias (PBIAS) was 7.26%, indicating a slight overestimation of the total streamflow volume, while the R^2 value of 0.79 shows that 79% of the variance in observed streamflow was explained by the model.

In the validation period (2001–2007), the model's performance remained consistent, with an RMSE of 0.6. The NSE value slightly improved to 0.672, maintaining stable performance across both periods. The PBIAS slightly increased to 7.41%, suggesting a marginal overestimation in streamflow, but this value remains within acceptable limits. The R^2 value also improved to 0.80, indicating that the model was able to explain 80% of the variance in observed data during the validation period.

Overall, the model demonstrates reliable performance in both calibration and validation phases, with strong NSE and R^2 values, low RMSE, and minimal percent bias. This indicates the model is suitable for hydrological predictions in the Tikur Wuha River watershed, though there is a slight overestimation of the flow.

4.3.5 Design storm analysis result of HEC-HMS model

In this study, the HEC-HMS model was employed to optimize parameters for the gauged basin of the Tukur Wuha River. These optimized parameters were subsequently transferred to the ungauged Rasa and Wodesa River watersheds, which are sub-basins of the Tukur Wuha River, using the regionalization technique. By selecting basins with similar physical characteristics, the reliability of the transferred parameters is enhanced, making them suitable for hydrological analyses, specifically for flood prediction in the Rasa and Wodesa Rivers.

The optimized parameters for the sub-catchment in the SCS Unit Hydrograph Transform Method were the time of concentration (Tc) and lag time (tlag). For the SCS Curve Number Loss Method, the parameters include initial abstraction, weighted soil curve number, and percentage of impervious areas, as shown in Tables 4.10 and 4.11 for both the Rasa and Wodesa River watersheds. According to the SCS method, the unit hydrograph (UH) lag time for ungauged catchments can be estimated as $t_{lag}=0.6\times T_c$, where Tc is the time of concentration.

Table 4.10 Optimized parameter values for Rasa watershed input to HEC-HMS model

Sub basin	Area (Km²)	Loss			Transformation method	
		Initial abstraction	CN	% Impervious	Tc (hr.)	Lag time(hr.)
S1	2.7208	16.75	81.27	3.24	1.55	0.93
S2	5.1006	16.48	62.84	2.75	1.88	1.13
S3	3.8504	16.48	77.56	0.43	0.89	0.53

Table 4.11 Optimized parameter values for Wodesa watershed input to HEC-HMS model

Sub basin	Area (Km²)	Loss			Transformation method	
		Initial abstraction	CN	% Impervious	Tc (hr)	Lag time(hr)
S1	15.62	16.75	68.37	3.24	2.32	1.39
S2	6.31	16.48	75.82	2.75	1.14	0.68
S3	9.86	17.48	63.23	0.43	2.07	1.24
S4	22.25	17.11	73.46	3.47	2.72	1.63
S5	3.63	16.66	71.54	1.78	1.15	0.69

The reach routing method used was the Muskingum routing method, which is applied for streamflow routing in river reaches. It simulates how water flows through a river channel over time by considering changes in storage, inflow, and outflow along the channel. In the HEC-HMS model, the Muskingum method relies on two key parameters: K (Storage Time Constant) and X (Weighting Factor). The Muskingum K value represents the travel time or storage effect of water as it moves through the river. A higher K value indicates longer storage and slower movement through the channel. X is a dimensionless weighting factor, ranging between 0 and 0.5, that defines the relative influence of flow on storage levels.

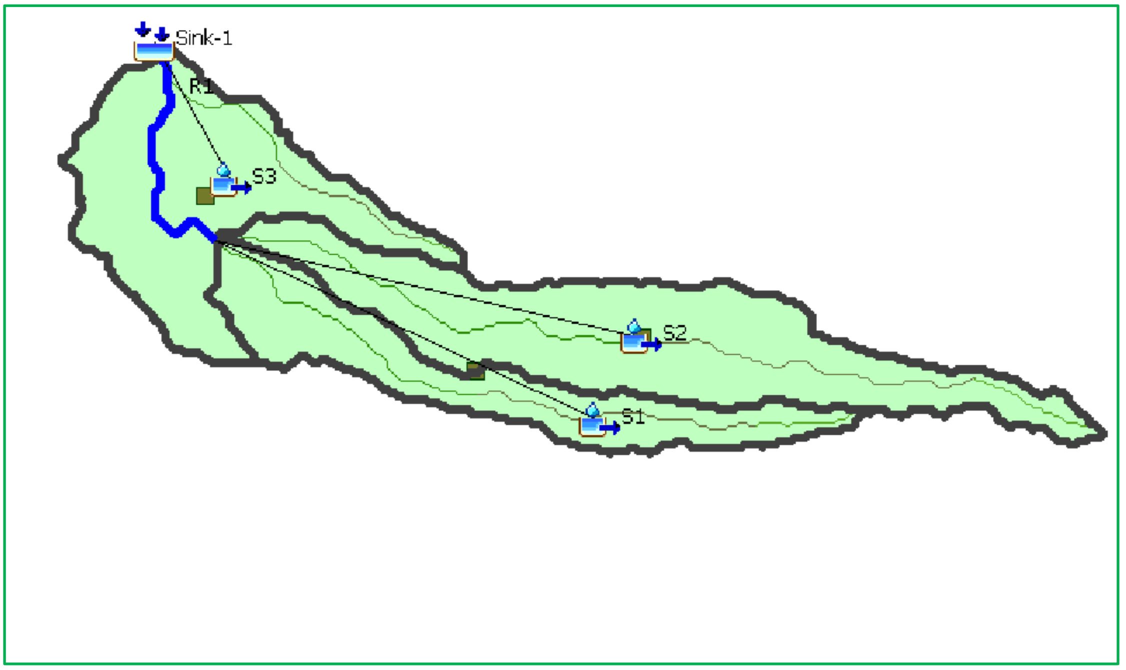


Figure 4.8 Layout of HEC-HMS for Rasa watershed

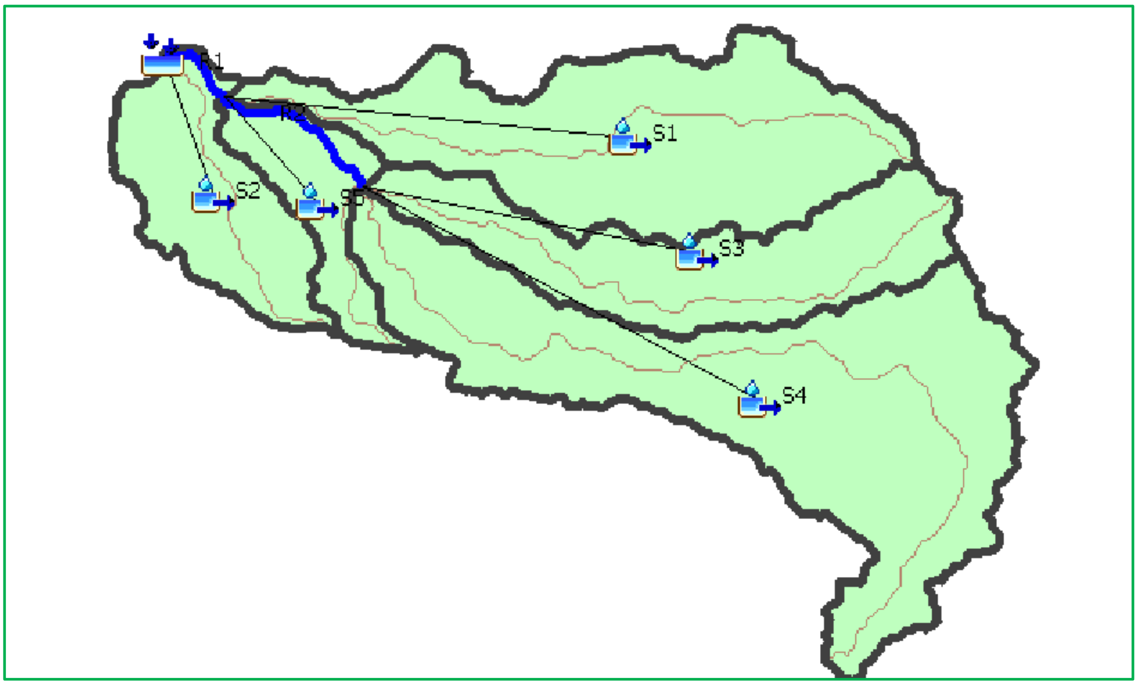


Figure 4.9 Layout of HEC-HMS for Wodesa watershed

In this study, the design storm analysis using the HEC-HMS model was conducted by inputting maximum rainfall data for a 50-year return period. The incremental depth of precipitation was arranged sequentially using the alternate block method see (table 4.12). The analysis results of the event-based simulation are presented in Table 4.13 and Table 4.14 for both the Rasa and Wodesa rivers, respectively.

Table 4.12 Input for design storm simulation of Rasa and Wodesa river

	Return period (year)		
	min	hr.	50 year
	5	0.083	17.37
	10	0.167	28.99
	15	0.25	37.38
Design Rainfall:	30	0	52.96
XT = 10Y	60	1	67.76
	120	2	80.18
	180	3	86.28
	360	6	95.19
	720	12	102.87
	1440	24	113.87

Table 4.13 Summary of design storm simulation results for Rasa river

Project: Rasa DHW Simulation Run: 50 yr frequency storm
Sink: Sink-1

Start of Run:	12Jun2024, 00:00	Basin Model:	Basin model
End of Run:	13Jun2024, 00:00	Meteorologic Model:	Meteorological model
Compute Time:	20Jun2024, 23:45:42	Control Specifications:	Control specification

Volume Units: MM 1000 M3

Computed Results

Peak Discharge: 59.2 (M3/S) Date/Time of Peak Discharge 12Jun2024, 13:16
Volume: 757.8 (1000 M3)

The HEC-HMS model analysis result for Rasa diversion headwork 50 year returning period of frequency storm at the weir site or at outlet junction was 59.2 cubic meters per second (m^3/s) and the base flow of Rasa river was 80l/sec which was the taken from Wondogenet woreda water mine and energy office; therefore, the total discharge of the river was 59.2m3/s.

Table 4.14 Summary of design storm simulation results for Wodesa river

Project:	Wodesa DHW	Simulation Run:	Frequency storm of wodesa
		Sink:	Sink-1
Start of Run:	12Jun2024, 00:00	Basin Model:	Basin model
End of Run:	14Jun2024, 00:00	Meteorologic Model:	Meteorological model
Compute Time:	20Jun2024, 21:25:16	Control Specifications:	Control specification
Volume Units:	◉ MM ○ 1000 M3		
Computed Results			
Peak Discharge:	98.2 (M3/S)	Date/Time of Peak Discharge	12Jun2024, 15:00
Volume:	52.28 (MM)		

The HEC-HMS model analysis for the Wodesa River, considering a 50-year return period frequency storm at the weir site or outlet junction, estimated a peak discharge of 98.2 cubic meters per second (m^3/s). Additionally, the base flow of the Wodesa River, recorded at 150 liters per second (l/s), was obtained from the Wondogenet Woreda Water, Mines, and Energy Office. Therefore, the total discharge of the river during peak conditions was calculated to be 98.2 m^3/s. Detailed analysis, including the HEC-HMS simulation hydrograph and outlet discharge data for both rivers over time, is presented in Appendix A, Figures 1 and 2, and Tables 8 and 9.

4.4 Hydraulic analysis

4.4.1 Adequacy of water way

The weir crest length (P) should be adequate to pass the design flood safely. The minimum stable width of an alluvial channel was usually determined from the Lacey's wetted perimeter (P). Lacey calculated a series of regime flow equations. One of these describes the regime perimeter of a river in alluvial material in terms of its dominant flow (Ertiro, 2017).

Water flow length or water way is calculated from Lacey's perimeter formula.

I. *For Rasa diversion head work*

$L = 4.75\sqrt{Q} = 4.75\sqrt{59.2} = 36.55 \times 0.5 = 18.3m$

Design flood magnitude = 59.2m3/s.

Constructed weir crest length = 7m

Width of under sluice = 0.5m

II. *For Wodesa diversion head work*

$L = 4.75\sqrt{Q} = 4.75\sqrt{98.2} = 47.07 \times 0.5 = 23.5m$

Design flood magnitude = 98.2m3/s.

Constructed weir crest length = 10m and Width of under sluice = 1m

In most cases, the calculated waterway was wider than the actual river regime. According to the MoA (2018), a reduction in the waterway width was inevitable due to the looseness factor, defined as the ratio of the actual provided waterway to the calculated waterway. The recommended looseness factor is between 0.5 and 1. However, the constructed waterway widths of 7m for Rasa and 10m for Wodesa were significantly less than the required minimum widths of 18.3m and 23.5m, respectively. This indicates the weirs are inadequate to safely pass the high floodwaters of the rivers.

The MoA guidelines recommend a maximum energy level of 2m over the crest of diversion weirs. Based on the broad-crested weir discharge formula, the maximum allowable discharges were 44.2 m^3/s for Rasa and 84 m^3/s for Wodesa. These are below the design flood discharges of 59.2 m^3/s and 98.2 m^3/s, as calculated by the HEC-HMS model, indicating insufficient capacity to manage peak flood events.

The energy head levels for the design floods were calculated as 2.4m for Rasa and 2.9m for Wodesa, both exceeding the recommended 2m limit. Consequently, the waterway capacities of the Rasa and Wodesa weirs are inadequate to handle design floods with a 50-year return period. This inadequacy highlights the need for modifications or reinforcements to ensure the structures can withstand extreme flood events.

4.4.2 Head of water over the crest

During the field observation the upstream floor level of the constructed weir of Rasa river was 1,812.50m.a.s.l, the height of weir was 0.4m the crest level of weir was 1812.90m.a.s.l and the top width of weir was 1m. For wodesa diversion headwork river bad level was 1,787.28m.a.s.l, the height of weir was 1m the crest level of weir was 1788.28m.a.s.l and the top width of weir was 1.5m. The computation of depth of water was calculated by try and error with velocity head equation given as.

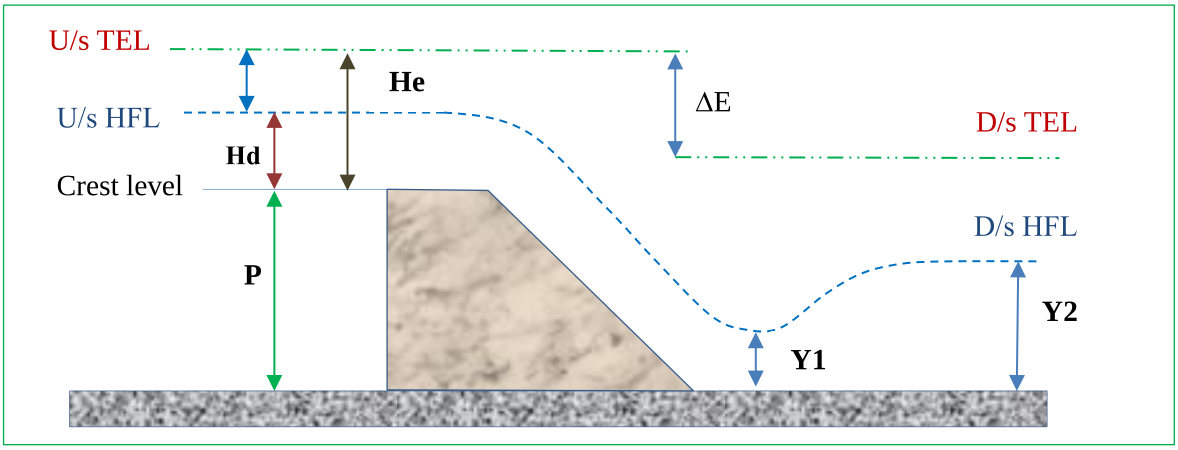


Figure 4.10 Weir profile

- Discharge over the weir is generally expressed as:

$$Q = CL_oH_e^{3/2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.1$$

$$H_e = (Q / (C.L))^{2/3}$$

Where: - Q = Peak Discharge

C = Discharge Coefficient (C = 1.7)

H_e= Total Energy Head (m)

L_o= Crest Length of Weir (m)

- The approach velocity (Va):

$$Va=(Q/A) =\left(\frac{Q}{L(Hd+P)}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.2$$

- Approach velocity head (Ha):

$$Ha = He - Hd;\ where\ Ha = \frac{Va^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.3$$

Then the Substitute the value of Ha

$$Ha = H_e\text{-}H_d = \left(\frac{Va^2}{2g}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.4$$

Substitute the value of Va into Equation 4.4

$$H_e\text{-}H_d = \left(\frac{\left(\frac{Q}{L(Hd+P)}\right)^2}{2g}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.5$$

The values of H_d can be obtained by trial and error method

I. For Rasa diversion headwork

Q = Peak Discharge, 59.2m^3/s

C = Discharge Coefficient (C = 1.7)

L = Crest Length of Weir = 7m

P = Height of weir = 0.4m

T = U/s Height of wing wall = 3m

H_e= Total Energy Head = 2.91m (Calculated by using equation 4.1)

The value of H_d is determined by using equation 4.5

$$H_e\text{-}H_d = \left(\frac{\left(\frac{Q}{L(Hd+P)}\right)^2}{2g}\right) = 2.91\text{-}H_d = \left(\frac{\left(\frac{59.2}{7(Hd+0.4)}\right)^2}{2*9.81}\right) = \frac{3.645}{(Hd+0.4)^2} \qquad \text{……………4.6}$$

The values of H_d can be obtained from equation above by trial and error method the depth of water over weir crest was 2.47m. The extreme depth over the weir measured from retaining wall top to weir crest was 2m which was 80.87% the designed water depth over the weir to pass the peak flood of the study. This indicate the underperformance of weir crest and upstream retaining wall constructed for Rasa diversions weir.

II. For Wodesa diversion headwork

Q = Peak Discharge, 98.2m^3/s

C = Discharge Coefficient (C = 1.7)

L = Crest Length of Weir = 10m

P = Height of weir = 1m

T = U/s Height of wing wall = 3.6m

H_e= Total Energy Head = 3.22m (Calculated by using equation 4.1)

The value of H_d is determined by using equation 4.5

$$H_e\text{-}H_d = \left(\frac{\left(\frac{Q}{L(Hd+P)}\right)^2}{2g}\right) = 3.22\text{-}H_d = \left(\frac{\left(\frac{98.2}{10(Hd+1)}\right)^2}{2*9.81}\right) = \frac{4.915}{(Hd+1)^2} \qquad \text{…………………………………….4.7}$$

The values of the head (Hd) were obtained using the trial-and-error method based on the provided equation, with the depth of water over the weir crest measured at 2.9 meters.

The extreme depth from the top of the retaining wall to the weir crest was 3 meters, representing 100% of the designed water depth required to pass the peak flood in the study. This indicates that both the weir crest and the upstream retaining wall of the Wodesa diversion weir were constructed to effectively manage peak flood events.

4.4.3 Hydraulic jump computation

A hydraulic jump occurs when a stream of water with high velocity and low depth (supercritical flow) meets another stream with low velocity and high depth (subcritical flow), causing a sudden rise in the water surface. The type of hydraulic jump is characterized by the Froude number, and the appropriate protection structures are determined based on the jump's characteristics (Garg, 2005).

Table 4.15 Classification of jump based on Froude number

Types of Jump	Froude number	Jump Characteristics	Energy dissipation (%)
Strong jump	Fr> 9	Rough jump, lots of energy dissipation	5
Steady jump	4.5 <Fr< 9	Considerably energy losses	20
Oscillating jump	2.5 <Fr< 4.5	Unstable oscillating jump; production of large waves of irregular period	20-40
Weak jump	1.7 <Fr< 2.5	Little energy loss	40-70
Undular jump	1.0 <Fr< 1.7	Free-surface undulations d/s of the jump; negligible energy loss	70-85

Source: (Garg, 2005).

Calculation of hydraulic jump stated from continuity equation where the energy level before jump equal to after jump as follow.

$$P + He = Y_1 + \frac{V^2}{2g} \qquad \text{.......... 4.8}$$

Where, P= height of Weir, He= Total head over weir crest V_1= velocity at section 1

a) *For Rasa diversion headwork*

Q = Peak discharge = 59.2m^3/s

H_e= Total energy head = 2.91m

P = Height of weir = 0.4m

L = Crest length of weir = 7m

By using equation 4.8

$$P + He = Y_1 + \frac{V_1{}^2}{2g} \;;\; 0.4 + 2.91 = Y_1 + \frac{V_1{}^2}{2g}$$

$$3.31 = Y_1 + \frac{V_1{}^2}{2g} \qquad \text{.......... 4.9}$$

$$V_1 = \frac{Q}{A} = \frac{Q}{L*Y_1} = \frac{59.2}{7*Y_1}; = \frac{8.46}{Y_1} \qquad \text{.......... 4.10}$$

Substituting eq 4.10 into eq 4.9

$$3.31 = Y_1 + \frac{\left(\frac{8.46}{Y_1}\right)^2}{2g} ; = Y_1 + \frac{71.57}{2*9.81*Y_1^{\,2}} = Y_1 + \frac{3.65}{Y_1^{\,2}} ;$$

$$Y_1^{\,3} = 3.31Y_1^{\,2} - 3.65 \ldots\ldots 4.11$$

The values of Y1 can be obtained from equation 4.11 by trial and error method. Hence, the value of Y_1=0.98m; Critical depth (Y_C) must be greater than y1

$$Yc = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} ; = \left(\frac{8.46^2}{9.81}\right)^{\frac{1}{3}} = 1.94m$$, and Velocity at section 1 become

$$V_1 = \frac{8.46}{Y_1} = \frac{8.46}{0.98} = 8.63 \text{ m/s}$$

The downstream apron was flat, the conjugate depth of Y2 is calculated by

$$Y_2 = \frac{Y_1}{2}\left[\sqrt{1+8F_1^{\,2}}-1\right] \ldots\ldots 4.12$$

Where: Froude number (F_1)= $\frac{V}{\sqrt{gY_1}}$; = $\frac{8.63}{\sqrt{9.81*0.98}}$ = 2.78

$$Y_2 = \frac{0.98}{2}\left[\sqrt{1+8*2.78^2}-1\right] = 3.4m$$> Tail water depth, y_3 = 2.02m

Based on the above (table 4.15) description the characteristics of jump was unstable oscillating jump; production of large waves of irregular period and the type of Oscillating jump. Where about 20-40% of the energy needs to reduce or dissipate to avoid scoring of the downstream river bed material; but there was no provided energy dissipater at down floor of the Rasa diversion headwork this case the high scoring of the downstream river and constructed floor.

b) For Wodesa diversion headwork

Q = Peak discharge = 98.2m^3/s

H_e= Total energy head = 2.9m

P = Height of weir = 1m

L = Crest length of weir = 10m

By using equation 4.5

$$P + He = Y_1 + \frac{V_1^{\,2}}{2g} ; \; 1+ 2.9 = Y_1 + \frac{V_1^{\,2}}{2g}$$

$$3.9 = Y_1 + \frac{V_1^{\,2}}{2g} \ldots\ldots 4.13$$

$$V_1 = \frac{Q}{A} = \frac{Q}{L*Y_1} = \frac{98.2}{10*Y_1}; = \frac{9.82}{Y_1} \qquad \text{.........4.14}$$

Substituting eq 4.14 into eq 4.13

$$3.9 = Y_1 + \frac{\left(\frac{9.82}{Y_1}\right)^2}{2g}; = Y_1 + \frac{96.43}{2*9.81*{Y_1}^2} = Y_1 + \frac{4.92}{{Y_1}^2};$$

$${Y_1}^3 = 3.4{Y_1}^2 - 3.65 \qquad \text{.........4.15}$$

The values of Y1 can be obtained from equation 4.12 by trial and error method. Hence, the value of Y_1=1.40m; Critical depth (Y_C) equals to:

$$Yc = \left(\frac{q^2}{g}\right)^{\frac{1}{3}}; = \left(\frac{9.82^2}{9.81}\right)^{\frac{1}{3}} = 2.14\text{m, and Velocity at section 1 become}$$

$$V_1 = \frac{9.82}{Y_1} = \frac{9.82}{1.4} = 7.07 \text{ m/s}$$

Where Froude number: $F_1 = \frac{V}{\sqrt{gY_1}} = \frac{7.07}{\sqrt{9.81*1.28}} = 1.9$

The downstream apron was flat, the conjugate depth of Y2 is calculated by

$$Y_2 = \frac{Y_1}{2}[\sqrt{1 + 8{F_1}^2}-1] = \frac{1.4}{2}[\sqrt{1 + 8 * 1.9^2}-1] = 3.12\text{m} > Y_3 = 2.16\text{m}$$

The hydraulic jump observed at the Wodesa diversion headwork, as described in Table 4.15, was characterized as a weak jump. In this type of jump, approximately 45-70% of the energy needs to be dissipated to prevent scouring and damage to the downstream riverbed material. Without adequate energy dissipation structures, the downstream bed material becomes vulnerable to erosion. This highlights the critical need for effective energy dissipation measures to ensure the protection of the riverbed from potential damage caused by the hydraulic jump.

The downstream water depth, based on the rating curve, was measured at 2.02 m for the Rasa River and 2.16 m for the Wodesa River. Both downstream depths are lower than the conjugate depth, indicating that the hydraulic jump occurs further downstream, beyond the impervious floor. This condition necessitates the design and construction of an energy-dissipating structure to control where the hydraulic jump takes place and ensure it occurs within the protected section of the riverbed, thereby preventing potential scouring. However, in this study, no such energy-dissipating structures were present, leaving the downstream riverbed vulnerable to scouring from the high-energy jump.

According to the Ministry of Agriculture (MoA, 2018), the difference between the conjugate depths (Y_2 and Y_3) should ideally be zero for the best hydraulic conditions. However, a general range of 0.2m to 0.4m is acceptable. In the case of the Rasa and Wodesa diversion headworks, Y_2 is greater than Y_3, which indicates the need to depress the riverbed to improve hydraulic performance and minimize erosion.

4.4.4. Evaluation of constructed weir components

4.4.4.1 River training work

River training work was used to protect the outflow of water due increasing of water depth after construction of weir and used to narrowing down the natural river bank (Tadesse, 2019).

In the case of the Rasa diversion headwork, the average natural riverbank width measured upstream of the weir is 14 meters, while the width between the right and left ends of the upstream structure narrows to 10 meters. However, there is no river training structure in place, and the left-side wing wall at the headwork was not constructed. As a result, during heavy rainfall seasons, the river water tends to outflank the structure on the side of the off-taking canal, leading to potential flooding and erosion. In contrast, the Wodesa diversion headwork has well-maintained river training works that effectively protect the riverbanks and prevent similar issues.

4.4.4.2 Upstream and downstream cut-off structure

The cut off structure is structure used to reduce uplift pressure and prevent or reduce flow of water seepage underneath of foundation of the weir by increasing flow path or creep length of seepage. Usually constructed from RCC concrete at upstream end and downstream end of impervious floor; sometimes provided at middle of impervious floor of weir (MoA, 2018).

In the case of the Rasa and Wodesa diversion headworks, the upstream and downstream cutoff structures were constructed to a depth of 1m for the upstream cutoff and 1.75m for the downstream cutoff for Rasa, and 1m for the upstream cutoff and 2m for the downstream cutoff for Wodesa, respectively.

To fix bottom levels of cut-offs, the scour depth, R is multiplied by a factor of safety ranging from 1.25 to 1.5 and 1.75 to 2.0 for upstream cut-off and downstream cut-off depths respectively. By using U/S HFL-1.25R for upstream and D/S HFL-1.75R for downstream pile level (MoA, 2018).

Based on the geological description of the study area from the design document, the river bed material at the weir site for the Rasa River consists of coarse gravel with silty clay deposits. For the Wodesa River, the river bed is composed of sand, gravel, and rock block materials. The mean silt factor values are 4.75 for the Rasa River and 2.75 for the Wodesa River, as indicated in Table 3.4. Assuming that the foundation material is uniform for both the upstream and downstream piles, these silt factor values will inform the design considerations for the weir.

A. Cut off depth determination for Rasa diversion headwork:

The depth of scour can be calculated using Lacey's equation

$$R = 1.35 \times \left(\frac{q^2}{f}\right)^{\frac{1}{3}} \quad \ldots\ldots 4.16$$

Where: R = Hydraulic mean depth

q = discharge per meter length = 59.20 m^3/s / 7.0m = 8.46m^3/s /m; Where 59.20 m^3/s is Q_{50} design flood and 7.0m average river width at proposed intake site

f = Lacey's silt factor, assumed to be 4.75 for course gravel with silty clay

$$R = 1.35 \times \left(\frac{8.46^2}{4.75}\right)^{\frac{1}{3}} = 3.33$$

- *Upstream cutoff depth:*

  Highest flood level at the upstream = 1815.37m

  River bed level =1812.5m

  Required depth of scour below HFL at upstream = 1.5*R = 1.5*3.33 = 4.99m

  Upstream cutoff level = U/S HFL- (1.5 × R) =1815.37 – 4.99 = 1810.38m. amsl

  U/S depth of Cutoff = River bed level – 1810.38 = 2.12m

- *Downstream cutoff depth:*

  Highest flood level at the Downstream =1815.9m

  River bed level = 1812.5m

  Required depth of scour below HFL at Downstream = 1.75 R = 5.83m

  Downstream cutoff level = D/S HFL- (1.75 × R) =1815.9 – 5.89 =1810.01m

  D/S depth of cutoff = River bed level- 1810 = 2.49 m, say 2.5m

B. Cut off depth determination for Wodesa diversion headwork:

The depth of scour can be calculated using Lacey's equation.

Where: R = Hydraulic mean depth

q = discharge per meter length = 98.20 m^3/s / 10.0m = 9.82m^3/s /m;

where 98.2 m^3/s is Q_{50} design flood for wodesa river and 10.0m average river width at proposed intake site

f = Lacey's silt factor, assumed to be 2.75 for sand, gravel and rock block

$$R = 1.35 \times \left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35 \times \left(\frac{9.82^2}{2.75}\right)^{\frac{1}{3}} = 4.42$$

- *Upstream cutoff depth:*

  Highest flood level at the upstream = 1791.18m

  River bed level =1787.28m

  Required depth of scour below HFL at upstream = 1.5*R = 1.5*4.42 = 6.63m

  Upstream cutoff level = U/S HFL- (1.5 × R) =1791.18 – 6.63 = 1784.55m. amsl

  U/S depth of Cutoff = River bed level – 1784.55= 2.73m ≅ 2.75m

- *Downstream cutoff depth*

  Highest flood level at the downstream =1790.40m

  River bed level = 1787.28m

  Required depth of scour below HFL at downstream = 1.75 R = 7.74m

  Downstream cutoff level = D/S HFL- (1.75 × R) =1790.49 – 7.74 =1782.75m

  Downstream depth of cutoff = River bed level – 1782.75 = 4.53 m, say 4.5m

Table 4.16 Required pile and provided pile for Rasa and Wodesa diversion headwork

Diversion weirs	Design discharge (m3/s)	Cut off pile	Silt factor (f)	Scouring depth (R)	Required pile depth (m)	Provided pile depth (m)
Rasa	59.2	Upstream	4.74	3.33	2.12	1
		Downstream	4.74	3.33	2.5	1.75
Wodesa	98.2	Upstream	2.75	4.42	2.75	1
		Downstream	2.75	4.42	4.5	2

The result in section 4.4.4.2 in table 4.16, indicated the designed upstream and downstream cutoff piles for Rasa diversion headwork is 1m and 1.75m for Wodesa diversion headwork was 1m and 2m respectively which are much less than the required upstream cutoff piles 2.12m and 2.75m, and downstream cutoff piles 2.5m and 4.5m for Rasa and Wodesa respectively, which are 40% and 33.33% below the required upstream piles and 70% and 44.44% below the required downstream piles for Rasa and Wodesa diversion headwork structures respectively.

4.4.4.3 Upstream and downstream impervious floor

Impervious floor or apron is provided to protect the main body of the weir from the scouring effect and ultimate failure. The total floor length of impervious floor includes the downstream basin/floor length, glacis, weir crest, and upstream floor. The impervious floor in conjunction with the downstream cutoff should result in safe exit gradient. The purpose of impervious apron in the weir structure is to resist uplift pressures and to dissipate the incoming energy over the weir. The length of the apron is determined considering the hydraulic jump length and the available exit gradient at the end of the apron and the apron varies in thickness from maximum at the start of the hydraulic jump stilling basin/apron to a minimum at the end is recommended. In general, impervious apron of sufficient length must be provided on the upstream and downstream side of the weir. Similarly, the thickness of the apron especially of the downstream apron should be sufficient to resist the hydraulic jump as well as the uplift pressure (Garg, 2005).

A. Determination of apron length for Rasa diversion headwork

The apron length is fixed by Lane's method as detailed below. According to this method, the total creep length provided which is equal to sum of total vertical creep lengths and one third of horizontal creep distance must be greater than CH (Novak, 2007).

$$L > C*H \qquad \text{……4.17}$$

Where, L= Total percolation distance (m) = Horizontal length + 1/3 vertical length

C = percolation co-efficient = 4.75, i.e. C for course gravel mix with silty clay boulder

H = max head (m)

For Dynamic case: H = U/s HFL - D/s HFL = 1815.37 – 1813.37 = 2m

For Static case: H= weir crest level minus -river bed = 1812.90 – 1812.5 = 0.4m

Therefore H = 2m is the maximum head; Thus, L > 4.74*2 = 9.5m

- Length of downstream impervious floor without crest shutters (Ld/s) is obtained from:

$$L_{d/s} = 2.21C\sqrt{\frac{H}{10}} = 2.21*4.75\sqrt{\frac{2}{10}} = 4.7m \qquad \text{……4.18}$$

- Length of upstream impervious floor (Lu/s):

$$Lu/s = L - (L_{d/s} + \text{Bed width} + 2*d_{u/s} + 2*d_{d/s}) \qquad \text{……4.19}$$

$$= 9.5 - (5.00 + 1.8 + 2*2.5 + 2*5.0) = -12.3m.$$

The negative sign indicates that no need to provide u/s impervious floor, but adopt the nominal length, 3.0m was constructed its safe.

B. Determination of apron length for Wodesa diversion headwork

The total creep length given is must exceed the sum of vertical creep lengths and horizontal creep distance C= percolation coefficient = 9, i.e. C for Sand mixed with gravel and boulder.

For Dynamic case: H = U/s HFL - D/s HFL = 1791.18 – 1789.44 = 1.74m

For Static case: H= weir crest level minus -river bed = 1788.28 – 1787.28 = 1m

Therefore H = 1.74m is the maximum head; Thus, L > 1.74*9 = 15.66m

- Length of downstream impervious floor (Ld/s) is obtained using equation 4.18

$$L_{d/s} = 2.21C\sqrt{\frac{H}{10}} = 2.21*9\sqrt{\frac{1.74}{10}} = 8.32\text{m. Say 9m}$$

- Length of upstream impervious floor (Lu/s) :

$L_{u/s} = L – (L_{d/s} + \text{Bed width} + 2*d_{u/s} + 2*d_{d/s})$ ……………………………………4.19

$= 15.66 – (9.00 + 3.6 + 2*3 + 2*4.5) = -11.34\text{m}.$

The negative sign indicates that there is no need to provide an upstream impervious floor. However, a nominal length of 3.2 meters was constructed, which is considered safe.

Table 4.17 Required and provided length of apron for diversion headwork

Diversion weirs	Foundation	C*H	Provided	Required		Cutoff(m)	
			Ld/s	Ld/s	Lu/s	U/s	D/s
Rasa	Coarse sand	9.5	4	4.7	3	1	1.75
Wodesa	Gravel sand	15.66	8	8.32	3.8	1	2

The constructed downstream impervious floor lengths for the Rasa and Wodesa diversion headworks were 4 meters and 8 meters, respectively, which correspond to 80% of the design length for Rasa and 88.9% for Wodesa. Both of these values are below the required design specifications, as indicated in the table above.

4.4.4.4 Upstream and downstream protection work

The upstream and downstream impervious floor were protected by protection structure constructed from stone or concrete block at end to protect the scouring of river bed due to jump of flood; however, in case of Rasa diversion weir there is no protection structure for upstream and downstream end of impervious floor. These were increase the failure of the weir river bed impervious floor due to scouring. But for wodesa diversion headwork upstream and downstream protection structure was good standing to protecting the scouring of river bad.

4.4.4.5 Weir body dimension

The weir dimension includes the bottom and top width which was stable under condition of maximum stress. The top width of broad crested with vertical upstream face was designed for the following three cases and taken the maximum value (Novak, 2007).

i. *On the consideration of no tension criteria*

$$a = \frac{Hd}{\sqrt{G}} \quad \text{...4.20}$$

Where Hd is the depth water on top of weir and G is specific gravity of material where weir constructed (For masonry G = 2.25).

Table 4.18 Required and provided top width for diversion weir with no tension criteria

Diversion weirs	Water depth over the weir crest (Hd)	Specific gravity (G)	Required top width (a)	Constructed top width (m)
Rasa	2.47	2.25	1.65	1
Wodesa	2.9	2.25	1.93	1.5

ii. *On the consideration of no sliding criteria*

$$a = \mu \frac{Hd}{G} \quad \text{...4.21}$$

Where µ is coefficient of friction usually taken 3/2

Table 4.19 Required and provided top width for diversion weir with no sliding criteria

Diversion weirs	Water depth over the weir crest (Hd)	Specific gravity (G)	Required top width (a)	Constructed top width (m)
Rasa	2.47	2.25	1.65	1
Wodesa	2.9	2.25	1.93	1.5

iii. *For crest shutter condition criteria*

a= s+1...4.22

In case of Rasa and Wodesa diversion headwork there is no shutter over crest of the weir body so the top width for both schemes becomes is 1m under this condition.

The bottom width (B) of the weir may be obtained by providing suitable side slopes for upstream vertical faced weir. The bottom width of the weir should not be less than (Hd+p)/(G-1) for no flow, for high flow and for high flow when weir is submerged condition (Garg, 2005).

Table 4.20 Required and provided bottom width for diversion headwork

Diversion weirs	Height of weir (m)	Water depth over the weir crest (Hd)	Specific gravity (G)	Required bottom width (a)	Constructed bottom width (m)
Rasa	0.4	2.47	2.25	2.30	1.8
Wodesa	1	2.9	2.25	3.12	3

The results in Section 4.4.4.5 indicated that the top width of the weir for the design flood at the Rasa and Wodesa diversion headworks was 1m and 1.5m, respectively significantly below the required top widths of 1.65m and 1.93m. This represents a reduction of 60.60% and 77.72% below the required top widths for the Rasa and Wodesa diversion headwork structures, respectively. Similarly, the bottom widths of the weirs, which were 1.8m and 3m, fell short of the required dimensions of 2.30m and 3.12m, respectively. This constitutes a 78.26% and 96.15% reduction from the required bottom widths for the Rasa and Wodesa diversion headworks. These deficiencies could significantly impact the structural integrity and flood-handling capacity of the weirs.

4.4.4.6 Scouring sluice gate

According to (MoA), small scale irrigation design guide line the size of under scouring sluice gate should be designed and constructed to flushing of 10 to 20% of design flood discharge during expected design floods so as to reduce flood height over the structure and hence wing wall height, i.e. it should be designed to ensure sufficient scouring capacity to dispose of the above range of the peak flood.

Table 4.21 Required and provided capacity of under sluice

Diversion weirs	Design discharge (m3/s)	Recommend discharge of scouring (m3/s)	Sluice capacity $Q = C \cdot A \cdot \sqrt{2g \cdot \Delta H}$ (m3/s)	Remark
Rasa	59.2	11.84	3.15	Less value
Wodesa	98.2	19.64	8.35	Less value

The results in Table 4.21 indicate that the designed scouring sluice gate width for the Rasa diversion headwork is 0.5 m, and for the Wodesa diversion headwork, it is 1 m. These widths are 26.60% and 42.52% below the required capacity for the Rasa and Wodesa diversion headwork structures, respectively. The scouring sluice gate is primarily designed to remove sediment from the upper side of the weir and discharge it downstream without causing damage. However, during the field visit, significant problems were observed, including the accumulation of silt, gravel, and boulder material, as well as damage to the under sluice gates and clogging of the sluice gates at both diversion headworks.

These issues hinder the proper functioning of the sluice gates and increase sedimentation, affecting the performance of the diversion structures.

Figure 4.11 Effect of underperformance design of Rasa and Wodesa under sluice

4.4.5 Seepage analysis

Seepage analysis was carried out by using Khosla's theory for two condition of the water at upstream of the weir. These two conditions were water at pond level (water at crest of weir) and water at high flood level. The water level at pond the tail water depth was considered as zero where the water level at high flood level the tail water depth becomes downstream high flood level.

Table 4.22 Input data for Rasa and Wodesa weir seepage analysis

S/n	Description of data	Rasa	Wodesa	Remark
1	Weir crest level (m)	1812.9	1788.28	Measured
2	U/s High flood water level(m)	1815.37	1791.18	Calculated
3	River bed level (m)	1812.5	1787.28	From design guide line
4	Thickness of floor(m)	0.2	0.3	From design guide line
5	D/s High flood level (m)	1814.5	1789.4	Calculated
6	U/s cut-off depth	1	1	From design guide line
7	D/s cut-off depth (m)	1.75	2	From design guide line
8	Length of impervious floor (m)	8.8	14.8	Measured

4.4.5.1 Seepage pressure analysis's using Khosla's theory

Table 4.23 Table Summary of corrected pressures at various key points

Diversion headwork	U/s pile Nr. 1	% pressure	D/s pile Nr. 2	% pressure
Rasa	ΦE1	100	ΦE2	35.29
	ΦD1	78.48	ΦD2	26.50
	ΦC1	73.49	ΦC2	0
Wodesa	U/s pile Nr. 1	% pressure	D/s pile Nr. 2	% pressure
	ΦE1	100 %	ΦE2	29.81%
	ΦD1	83.83%	ΦD2	22.34%
	ΦC1	80.40%	ΦC2	0%

The head of water over the floor during high flood condition and pond level for Rasa diversion weir was 2.87m and 0.4m and for wodesa diversion weir is 3.9m and 1m respectively. The thickness of impervious floor required in high flood condition and pond level condition at upstream pile inner corner and downstream pile inner corner were calculated as following table.

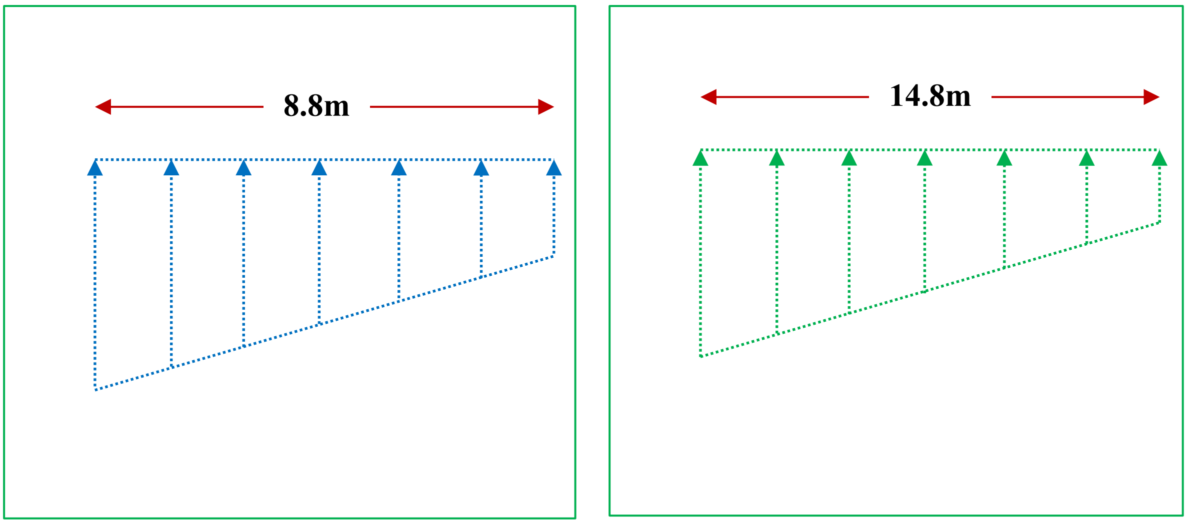
Table 4.24 Under weir pressure of Rasa diversion headwork by Khosla's theory

Rasa diversion headwork	Height of water above river bed	Height of sub soil H.G line above the datum d/s pile			
		U/s pile line		D/s pile line	
		ΦC		ΦE	
		73.49%		35.29%	
		Pressure head (m)	Required impervious floor thickness(m) (t=1.33h/G-1)	Pressure head (m)	Required impervious floor thickness(m) (t=1.33h/G-1)
HFC	2.87	0.35	0.33	0.17	0.16
NFC	0.4	0.29	0.27	0.14	0.15
Provided thickness			0.2		0.2

Table 4.25 Under weir pressure of Wodesa diversion headwork Khosla's theory

Wodesa diversion headwork	Height of water above river bed	Height of sub soil H.G line above the datum d/s pile			
		U/s pile line		D/s pile line	
		ΦC		ΦE	
		80.40%		29.81%	
		Pressure head (m)	Required impervious floor thickness(m) (t=1.33h/G-1)	Pressure head (m)	Required impervious floor thickness(m) (t=1.33h/G-1)
HFC	3.9	0.63	0.6	0.23	0.22
NFC	1	0.80	0.75	0.30	0.29
Provided thickness		0.3		0.3	

As indicated in table 4.24 and table 4.25 the constructed concrete floor thickness of Rasa and Wodesa diversion headwork structures from scoured section were much less than the required concrete floor thickness that cannot contour balance the uplift pressure under the structures. For the Rasa diversion headwork provided concrete floor thickness at upstream and downstream was 0.2m which is 57.1% of below the required average thickness of impervious floor 0.35m. In case of wodesa diversion headwork provided concrete floor, thickness was 0.3m which is 64.52% of below the require average thickness of impervious floor 0.47m.



(a) (b)

Figure 4.12 Uplift pressure distribution of Rasa (a) and Wodesa (b) diversion structures

Table 4.26 Calculated uplift pressure of Rasa and Wodesa diversion structures

Diversion headwork	Parameters	ΦE1	ΦD1	ΦC1	ΦE2	ΦD2
Rasa	% Pressure	100	78.48	73.49	35.29	26.5
	Pressure head (h)	0.47	0.37	0.35	0.17	0.13
	Uplift pressure (ϒω*h)	0.47ϒω	0.37ϒω	0.35ϒω	0.17ϒω	0.13ϒω
Wodesa	% Pressure	100	83.83	80.40	29.81	22.34
	Pressure head (h)	0.78	0.65	0.63	0.23	0.17
	Uplift pressure (ϒω*h)	0.78ϒω	0.65ϒω	0.63ϒω	0.23ϒω	0.17ϒω

Based on Equation 3.17, the exit gradient for the Rasa and Wodesa diversion headworks was calculated to be 0.29 and 0.30, respectively. The critical gradient of the foundation soil, as determined by Equation 3.19 from the methodology section, was 0.83 for Rasa and 0.94 for Wodesa. The factor of safety (Fs) against uplift pressure due to piping through the foundation of the Rasa and Wodesa diversion weirs was 2.66 and 3.13, respectively. A factor of safety of 2.66 is considered inadequate for the safe performance of the Rasa diversion weir, as it falls below the recommended minimum of 3. In contrast, the Wodesa diversion weir, with a factor of safety of 3.13, is considered adequately safe against piping.

4.5 Structural analysis

Structural analysis of this study including stability analysis of weir proper body and appurtenance structure such as retaining wall and wing wall. In this study the stability of diversion weir was analyzed under static case and dynamic case. Mode of failures are overturning, sliding and stress also checked for bearing capacity of the foundation.

The force acting on the weir body in static and dynamic case are self-weight of weir, uplift pressure, water pressure at upstream and downstream side. The other force such as wave, wind and earth quake are not considered in case of Rasa diversion headwork because the weir height was too short. However, for Wodesa diversion headwork the impact of earth quake is also considered since the study area is located in seismic zone.

4.5.1 Stability analysis of weir body

The stability of diversion weir was analyzed under pond level or no flow condition (static case) and for high flood level or overflow condition (dynamic case) and analyzed in detail in appendix-C in table 15, 16,17, and 18, and the result is presented as the summary of factors of safety against overturning, sliding, and tension developed for the normal and worst case of the two-diversion weir are presented in table 4.27 and table 4.28 respectively.

Table 4.27 Summary of structural analysis for Rasa weir body

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	1.66	8.72	Should be > 1.5	Safe
2	Stability against sliding	0.08	0.02	Should be < 0.65	Safe
3	Stability against tension				
	$X_{average}$	0.92	0.93		
	e	0.02	0.03		
	B/6	0.3	0.3		
	Overstress/Tension	ok	ok	e < B/6	Safe
4	Stability against contact pressure			Should be<100 KN/m^2	
	Contact pressure maximum (KN/m^2)	18.25	23.73		Safe
	Contact pressure minimum (KN/m^2)	15.70	19.42		Safe

Table 4.28 Summary of structural analysis for Wodesa weir body

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	1.52	2.86	Should be > 1.5	Safe
2	Stability against sliding	0.24	0.18	Should be < 0.65	Safe
3	Stability against tension				
	$X_{average}$	1.32	1.36		
	e	0.18	0.14		
	B/6	0.5	0.5		
	Overstress/Tension	ok	ok	e < B/6	Safe
4	Stability against contact pressure			Should be < 88 KN/m^2	
	Contact pressure maximum (KN/m^2)	38.98	41.88		Safe
	Contact pressure minimum (KN/m^2)	18.10	23.91		Safe

The results in Table 4.27 and Table 4.28 indicate that the stability analysis of the weir body for both the Rasa and Wodesa diversion headworks shows stability against sliding, overturning, overstressing, and contact pressure under normal flow conditions as well as high flood levels. Based on the soil texture analysis from laboratory results, the Rasa diversion weir site is predominantly composed of sandy soil with a mixture of silt and clay, classifying it as sandy loam. In contrast, the Wodesa diversion weir site is dominated by clay, with a mixture of sand and silt, classifying it as clay loam.

According to the Ministry of Agriculture (MoA), the allowable bearing pressure for sandy loam soil is between 100 and 200 kN/m², while for clay loam soils, it ranges from 75 to 150 kN/m². The contact pressure (stress) on the foundation at the toe or heel of the weir body was found to be less than the allowable bearing capacity of the foundation material, indicating that the weir is safe against contact pressure for both headwork.

4.5.2 Stability analysis of retaining wall structure

The stability analysis of the retaining wall structures for the upstream and downstream sections was conducted for both empty and high flood conditions. During site observations, it was noted that the retaining wall for the Rasa diversion weir consists of only the right-side upstream and downstream walls, with a maximum backfill soil depth of 2 meters on the inclined surface of the retaining wall. For the Wodesa diversion weir, the maximum backfill soil depth on the inclined surface is 2.2 meters. The stability of the upstream and downstream retaining walls was evaluated, and detailed analysis results are presented in Appendix C for both Rasa and Wodesa diversion headwork retaining walls. The results are summarized in Tables 4.29 and 4.30, which provide an overview of the structural analysis under both conditions.

Table 4.29 Summary of structure analysis of retaining wall under high flood condition

HFC	Rasa				Wodesa			
Factor of safety	U/S	Check	D/S	Check	U/S	Check	D/S	Check
Overturning >1.5	2.92	Safe	5.00	Safe	2.17	Safe	3.17	Safe
Sliding < 0.65	0.10	Safe	0.04	Safe	0.18	Safe	0.01	Safe
Overstress < B/6	0.05	Safe	0.19	Safe	0.13	Safe	0.35	Safe
B/6	0.33		0.33		0.41		0.41	
Pmax	93.33	safe	140.06	Unsafe	137.53	Unsafe	217.63	Unsafe
Pmin	67.94	Safe	29.55	safe	50.09	safe	-22.35	Unsafe

The results in Table 4.29 indicate that the structural analysis of the retaining wall under high flood conditions indicates that the wall is stable against overturning, sliding, and overstressing. However, concerns arise with contact pressure. At the Rasa site, the maximum contact pressure at the downstream foundation exceeds the allowable bearing capacity (100kN/m²), while at the Wodesa site, both upstream and downstream pressures surpass the safe limits (88kN/m²), indicating potential risk to the foundation. Despite these concerns, the overall structure remains stable, though the high contact pressures should be addressed to ensure long-term safety.

Table 4.30 Summary of structure analysis of retaining wall under normal flow condition

NFC	Rasa				Wodesa			
Factor of safety	U/S	Check	D/S	Check	U/S	Check	D/S	Check
Overturning >1.5	8.83	Safe	8.83	Safe	13.62	Safe	13.62	Safe
Sliding < 0.65	0.01	Safe	0.01	Safe	0.04	Safe	0.04	Safe
Overstress < B/6	0.6	Unsafe	0.6	Unsafe	0.16	Safe	0.16	Safe
B/6	0.33		0.33		0.41		0.41	
Pmax	208.95	Unsafe	208.95	Unsafe	126.82	Unsafe	126.82	Unsafe
Pmin	-72.37	Unsafe	-72.37	Unsafe	35.61	Safe	35.61	Safe

The results in Table 4.30 indicate that, under normal flow conditions, the retaining wall is stable against overturning and sliding, with safety factors well above the required levels. However, overstress is a concern at Rasa, exceeding the safe limit, while Wodesa remains within safe limits. Contact pressures exceed the allowable limits at both Rasa (208.95 kN/m²) and Wodesa (126.82 kN/m²), suggesting potential issues. Minimum pressures are acceptable at all sites. Overall, the retaining wall is stable under both high flood and normal flow conditions regarding overturning and sliding. Nevertheless, contact pressures surpass safe limits in both scenarios, indicating a need for further investigation and potential reinforcement. Additionally, the overstress conditions at Rasa under normal flow should be addressed.

The overall summary of the field observation survey and the assessment of the structural physical performance of the two diversion headwork structures, evaluating their current condition in terms of design practices, stability, durability, and adherence to design specifications, is presented in Table 4.31. The main components of the diversion headworks assessed include guide banks, intake gates, weir proper, sluice gates, retaining walls, aprons, cutoff piles, and protection works. The condition of each structure was classified as either Good, Poor, Failed, or Not Constructed.

Table 4.31 Structural physical performance level of the DHW structures

S/N	Diversion headwork structures	Rasa	Wodesa
1	Guide banks	Not constructed	Good
2	Intake gate	Failed	Good
3	Weir proper	Good	Good
4	Under sluice gate	Failed	Good
5	Intake head regulator	Failed	Good
6	Retaining walls	Good	Good
7	U/s apron	Good	Good
8	U/s cutoff pile	Good	Good
9	D/s apron pile	Poor	Good
10	D/s cutoff pile	poor	Good
11	Divide wall	Not constructed	Good
12	Marginal bunds	Not constructed	Good
13	Under sluice tunnels	Failed	Good
14	D/s protection works	Not constructed	Not constructed
15	U/s protection works	Not constructed	Good
16	Stilling basin/Dissipator	Not constructed	Not constructed

Based on the field observation and assessment of the two-diversion headworks, the structural performance of the Rasa and Wodesa diversion headworks exhibits significant contrasts. The Rasa diversion headwork is in a critical state, with only 25% of its structures rated as being in good condition, such as the weir proper, retaining walls, upstream apron, and upstream cutoff pile. These components are functioning according to design specifications and provide stability and durability to the system. However, several key structures have failed, including the intake gate, under sluice gate, intake head regulator, and under sluice tunnels, which severely compromise the operational safety and efficiency of the diversion. Additionally, components like the downstream apron pile and cutoff pile are in poor condition, further raising concerns regarding the overall structural integrity. Most alarmingly, 37.5% of the critical components, such as guide banks, the divide wall, and marginal bunds, were not constructed, which greatly affects the ability of the diversion headwork to manage high-flow conditions and protect downstream areas.

In contrast, the Wodesa diversion headwork is in significantly better shape, with 87.5% of its structures rated as being in good condition. These include the intake gate, weir proper, under sluice gate, retaining walls, upstream and downstream aprons, and cutoff piles. The good condition of these components ensures that the Wodesa headwork is stable, durable, and adhering to design specifications. Although some structures, such as the downstream protection works and stilling basin/dissipator, were not constructed, the overall impact on the system's performance is minimal in comparison to Rasa. Wodesa robust structural condition ensures its effective operation, with minimal maintenance required to maintain its high-performance level.

Table 4.32 Comparison of Rasa and Wodesa diversion headworks

Diversion Headwork	Good (%)	Poor (%)	Failed (%)	Not Constructed (%)
Rasa	25%	12.50%	25%	37.50%
Wodesa	87.50%	0%	0%	18.75%

From the table above, it is clear that the Wodesa diversion headwork is in far better condition, with the majority of its components functioning as designed. In contrast, the Rasa diversion headwork shows significant deficiencies, with only 25% of its components in good condition and a large percentage of failed or non-constructed elements.

5 CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The evaluation of the Rasa and Wodesa diversion weirs in Wondo Genet Woreda revealed critical structural, operational, and management challenges that affect their performance. The Wodesa diversion weir displayed better structural integrity, with key components such as the weir proper, intake gate, sluice gates, and cutoff walls in good condition, allowing for stable operation. In contrast, the Rasa weir exhibited severe deficiencies, including failed intake and under-sluice gates, and the absence of crucial structures like guide banks, marginal bunds, and the left-side wing wall. These omissions have significantly reduced the weir's ability to manage water flow, resist hydraulic forces, and ensure reliable water supply for irrigation.

Hydrological analysis using the HEC-HMS model, calibrated with streamflow data from the Tukur Wuha gauging station, demonstrated reliable predictive performance. The model provided peak flow rates of 59.2 m^3/s for the Rasa diversion and 98.2 m^3/s for Wodesa during a 50-year return period storm. However, the constructed weirs were designed for lower capacities of 44.2 m^3/s and 84 m^3/s, respectively. This discrepancy indicates that both weirs are undersized for handling peak flows, making them vulnerable to being overwhelmed during extreme flood events. Furthermore, the reduced crest length of both weirs compromises their ability to effectively pass the design flood, further increasing their vulnerability during high-flow periods.

The under-sluice sections at both weirs were found to have capacities 26.60% and 42.52% below the required levels for Rasa and Wodesa, respectively. This under-sizing has led to excessive sediment accumulation in the sluice pockets, clogging of under-sluices, and siltation in the intake gates and irrigation canals. Field inspections revealed significant issues, including the buildup of silt, gravel, and boulders, and damaged under-sluice gates, further reducing the effectiveness of the sluice systems.

In terms of structural adequacy, the concrete floor thicknesses at both weirs were found to be much less than required to counterbalance uplift pressures. For Rasa, the floor thickness of 0.2m was 57.1% below the required 0.35m, while for Wodesa, the 0.3m floor thickness was 64.52% below the required 0.47m. The exit gradients of 0.29 for Rasa and 0.30 for Wodesa, combined with critical gradients of 0.83 and 0.94, result in a factor of safety (Fs) of 2.66 for Rasa, which is below the recommended minimum of 3. This makes the Rasa weir unsafe against piping. Conversely, Wodesa's Fs of 3.13 indicates adequate safety.

The evaluation of weir dimensions showed that the top and bottom widths of the weirs at both sites were inadequate. The top widths for Rasa (1m) and Wodesa (1.5m) were significantly below the required dimensions of 1.65m and 1.93m, with deficiencies of 60.60% and 77.72%. Similarly, the bottom widths, 1.8m for Rasa and 3m for Wodesa, fell short of the required 2.30m and 3.12m, representing reductions of 78.26% and 96.15%. Despite these dimensional inadequacies, both weirs remain structurally stable against sliding, overturning, overstressing, and contact pressure under both normal and flood conditions.

The constructed retaining walls at both weirs were found to be stable against overturning and sliding under high flood and normal flow conditions. However, the contact pressures at both sites exceeded safe limits, necessitating further investigation and potential reinforcement. Additionally, overstress conditions observed at the Rasa weir under normal flow highlight the need for immediate attention to avoid potential structural failure.

In summary, while the Wodesa weir shows relative structural integrity, the Rasa diversion weir faces significant challenges that compromise its operational efficiency and safety. To ensure the long-term sustainability of both weirs, addressing the issues of under-sizing, inadequate structural dimensions, uplift pressures, and sediment management is critical. Proper maintenance and reinforcement of critical components are necessary to enhance the reliability and performance of these small-scale irrigation schemes.

5.2 Recommendations

Based on the findings of this study, the following recommendations are proposed to improve the performance and sustainability of the small-scale diversion weirs in the Rasa and Wodesa irrigation schemes:

1. **Structural reinforcement and repair**
    - Critical repairs are needed for the Rasa weir's intake gate, under-sluice gates, guide banks, and other missing components like marginal bunds and wing walls.
    - Reinforce retaining walls to address high contact pressures and correct overstress conditions in the Rasa weir.
2. **Adequate sizing of weir dimensions**
    - Revise the crest and width dimensions of both weirs to meet design standards, ensuring sufficient capacity to pass design floods. Future projects should consider long-term flood risks and operational needs.

3. **Sediment management and sluice gate maintenance**
   - Implement regular desilting of gates and canals to maintain operational efficiency. Resize the under-sluice sections to improve sediment passage and flood handling.
   - Establish routine maintenance schedules to prevent sediment buildup and ensure structural longevity.
4. **Increased concrete floor thickness**
   - The concrete floor thicknesses for both the Rasa and Wodesa weirs should be increased to meet the required values for resisting uplift pressure. In particular, the Rasa diversion weir, with a floor thickness 57.1% below the required standard, needs urgent modification to prevent failure due to uplift forces.
   - Future designs must ensure that the concrete floor thickness is adequate to counterbalance uplift pressure and prevent issues related to piping.
5. **Improved hydrological modeling and design storm consideration**
   - Conduct design storm analysis early in project planning to ensure weirs can handle peak discharges predicted by models like HEC-HMS. Adopt conservative flood estimation methods to add safety margins for extreme events.
6. **Regular monitoring and evaluation**
   - Regularly inspect the weirs for structural stability, sediment accumulation, erosion, and gate functionality. Develop a hydrological and structural monitoring program to guide timely interventions
7. **Training and capacity building**
   - Train local operators on proper weir operations, sluice gate handling, and sediment management to address issues proactively.
   - Provide continuing education programs for engineers and technicians to enhance design practices and performance assessment methods

Through the adoption of these recommendations, it is believed that there will be improvement in the performance, durability and safety of the small-scale diversion weirs in Rasa and Wodesa irrigation schemes to bring about sustainable water delivery for irrigation and also to minimize the instances of structural failure during extreme hydrological events.

REFERENCES

Afera, A. (2004). Appraisal of current design practices of river diversion structures in Amhara region. A thesis submitted to School of Graduate Studies, Addis Ababa University in Partial Fulfillment for the Degree of, 1-102.

Aliye, M. A. (2020). Evaluating the Performance of HEC-HMS and SWAT Hydrological Models in Simulating the Rainfall-Runoff Process for Data Scarce Region of Ethiopian Rift Valley Lake Basin. Open Journal of Modern Hydrology, 1-19. doi:DOI: 10.4236/ojmh.2020.104007

Arnold, J. S. (1998). Large area hydrologic modeling and assessment. Part I. Journal of the American Water Resources Association, 34(1), 73-89.

Arora, K. (1996). Irrigation water power and water Resource Engineering. Delhi: Slandered Publishers and distributions.

Asawa, G. (2008). Irrigation and water resources engineering. New Delhi: New age international.

Awulachew, S. B. (2010). Irrigation potential in Ethiopia. Addis Ababa: International Water Management Institute.

Baban, R. (1995). Design of Diversion Weirs: Small Scale Irrigation in Hot Climates. Wiley.

Belango, B. (2021). Performance Evaluation of Diversion Head Work Structures: Case study on Agega-1, Agega-2, and Sana Irrigation Schemes In Kembata Tembaro Zone, SNNPR, Ethiopia. MSc. Thesis Hawassa University, 1-158.

Beven, K. J. (1997). A physically based, variable contributing area model of basin hydrology. Hydrological Sciences Journal,, 24(1), 43-69.

Bitew, T. (2019). Application of HEC-HMS Model for Flow Simulation in the Lake Tana Basin: The Case of Gilgel Abay Catchment, Upper Blue Nile Basin, Ethiopia. journal/hydrology, 1-27. doi:10.3390/hydrology6010021

Braja, M. D. (2007). Principles of Foundation Engineering, SI Seventh Edition.

Chaw, V. T. (1988). Applied Hydrology. McGraw-Hill International editions.

Croke, B. F. (2004). A Catchment Moisture Deficit Module for the IHACRES Rainfall-Runoff Model. Environmental Modelling & Software , 19(1), 1-5.

Cunderlik, M. (2003). Hydrologic Model Selection Fort the CFCAS Project: Assessmentof Water Resources Risk and Vulnerability to Changing Climatic Conditions.

Daide, F. (2021). Beht Watershed (Morocco) Rainfall-Runo Simulation with the HEC-HMS Hydrological Model. Ecological Engineering & Environmental Technology. Retrieved from https://doi.org/10.12912/27197050/142942

Das, B. M. (2007). Principles of Foundation Engineering, SI Seventh Edition.

ERA, E. R. (2013). Drainage Design Manual. Federal Democratic Republic of Ethiopia.

Ertiro, F. (2017). Evaluation of Failures and Design Practices of River Diversion Structures for Irrigation: A Revisit of Two SSI Schemes in Ethiopia. International journal of Earth science and engineering, 10. doi:DOI: 10.21276/ijee.2017.10.0305

Fikru, H. (2014). Performance assessment of diversion head works implemented for irrigation. Case of Fentale irrigation based development project. MSc, Thesis. Addis Ababa University, Ethiopia.

Garg, S. (2005). Irrigation engineering and Hydraulic structures. New Delhi: Khanna publishers,.

Geressu, R. T. (2023). Optimised multi-objective design of weir structures. ICE Virtual Library, 173 - 185. Retrieved from https://doi.org/10.1680/jwama.22.00002

Hailu, D. (2005). Design and Operation of Diversion Systems. Nile Basin Capacity Building, 1-16.

Hora, M. W. (2016). Assessment of Design Practices and Performance of Diversion Weir in Small Scale Irrigation (Case Study from Projects in Western Oromia, Ethiopia). A Thesis submitted to the department Water Resource and Irrigation Engineering, Arba Minch University.

Hussain, F. (2021). Application of Physically Based Semi-Distributed Hec-Hms Model for Flow Simulation in Tributary Catchments of Kaohsiung Area Taiwan. Journal of Marine Science and Technology. Retrieved from https://jmstt.ntou.edu.tw/journal/vol29/iss1/4

Jakeman, A. J. (1990). Computation of the Instantaneous Unit Hydrograph and Identifiable Component Flows with Application to Two Small Upland Catchments. Journal of Hydrology, 117(1-4), 275-300.

Kefyalew, D. (2017). Performance Evaluation of Tibila Irrigation Based Development Project Diversion Headwork, Ethiopia. MSc. thesis Arba Minch University, 1-98.

Khassaf, S. I. (2008). Seepage Analysis Underneath Diyala Weir Foundation. Proceedings of the Thirteen International Water Technology Conference, IWTC, 1-12. doi:DOI: 10.13140/RG.2.2.15546.90565

Kirkby, M. J. (1997). A physically based, variable contributing area model of basin hydrology. Hydrological Sciences Journal,, 24(1), 43-69.

Lambisso, R. (2005). Assessment of Design Practices and Performance of Small Scale. World Vision Ethiopia, 1-57.

Lysne, D. G. (2003). Hydraulic design Trondheim, Norwegian University of Sciences and Technology, department of hydraulics and Environmental Engineering.

Marti, T. S. (2021). Applied Modeling of Hydrological Systems in Central Asia. Zurich, Switzerland. Retrieved from https://tobiassiegfried.github.io/HydrologicalModeling_CentralAsia/,

MoA. (2018). National Guidelines for Small Scale Irrigation Development in Agricultural Growth program. Addis Ababa, Ethiopia: Ministry of agriculture.

Moges, S. (2007). Application of semi-distributed conceptual hydrological model for flow forecasting on upland catchments of Blue Nile River Basin, a case study of Gilgel Abay catchment.

MoWR. (2002). Water sector development program. Addis Ababa, Ethiopia.: Ministry of water resource.

Neitsch, S. A. (2011). Soil and Water Assessment Tool (SWAT) Input/Output Documentation. Texas Water Resources Institute.

Novak, P. (2007). Hydraulic Structures Fourth Edition. Taylor & Francis 2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN.

Shrestha, P. T. (2006). Small Hydropower and head works.

Sisay, A. B. (2022). Assessment of design practice and performance of Small-scale Diversion weir construction for irrigation Structures: Case study of ashar, and upper kuashini Irrigation Schemes in Awi Zone, Dangla woreda, Amara region. MSc. Thesis Bahirdar University, Ethiopia, 1-114.

Subramanya, K. (2008). Engineering hydrology third edition. Indian institute of technology.

Tadesse, S. (2019). Investigation on the causes of failure of Tana Beles Weir. Master of Science Thesis Addis Ababa University, 1-113.

Temesgen, D. K. (2017). Performance Evaluation of Tibila Irrigation Based Development . MSc Thesis. Arba Minch University, Ethiopia., 1-98.

Turan, K. H. (2004). Software program for Design of Diversion Structures. MSc Thesis. Middle East technical university, 1-332.

WGWA. (2016). Socio-Economic profile of Wondo Genet Woreda. Unpublished.

WGWANRO. (2024). Wondo Genet Woreda Agriculture and Natural Resource Office. Unpublished.

Wondrade, N. (2023). Integrated use of GIS, RS and USLE model for LULC change analysis and soil erosion risk mapping in the Lake Hawassa Watershed, Southern Ethiopia. GEOCARTO INTERNATIONAL, 1-26. Retrieved from https://doi.org/10.1080/10106049.2023.2210106

Yami, M. (2016). Irrigation projects in Ethiopia: what can be done to enhance effectiveness under ‘challenging contexts’? International Journal of Sustainable Development & World Ecology.

APPENDIXES

Appendix (A) Hydrological data analysis

Table 1: - Daily heaviest rainfall for Awasa and Wondogenet meteorology stations

Year	Wondogenet station (mm)	Awassa station (mm)	Average areal Rainfall (mm)
1990	46.40	51.60	46.94
1991	71.15	60.00	70.00
1992	44.32	50.90	45.00
1993	56.26	44.10	55.00
1994	80.90	92.60	72.66
1995	77.80	60.40	76.00
1996	69.87	44.10	67.20
1997	63.75	61.30	63.50
1998	56.60	58.70	56.82
1999	60.50	52.66	59.69
2000	55.56	65.70	56.61
2001	108.55	97.45	107.40
2002	85.60	70.90	84.08
2003	58.80	70.50	60.01
2004	54.70	67.10	55.98
2005	68.85	65.57	68.51
2006	73.46	88.30	75.00
2007	50.50	61.00	51.59
2008	81.26	73.10	80.42
2009	66.23	55.00	65.07
2010	60.75	53.60	60.01
2011	59.60	51.70	58.78
2012	69.94	46.60	67.52
2013	99.58	90.10	98.60
2014	76.50	65.60	75.37
2015	55.02	45.30	54.01
2016	63.25	51.28	62.01
2017	62.60	60.40	62.37
2018	88.27	76.80	87.08
2019	50.20	63.20	51.55
2020	75.10	87.70	76.40
2021	61.10	26.40	57.51
2022	72.40	64.20	71.55

Table 2: - Areal coverage of rainfall station from thiessen polygon

Station	Area (km^2)
Wondogenet	615
Awasa	71

Table 3: - Areal daily heaviest rainfall for rainfall frequency analysis

S. No.	Year	Max. RF	Descending Order	Rank	Logarithmic Value/Yo/
1	1990	47.00	107.40	1	2.031004281
2	1991	70.00	98.60	2	2.015359755
3	1992	45.00	87.0	3	1.993876915
4	1993	55.00	84.00	4	1.939519253
5	1994	72.66	80.00	5	1.924279286
6	1995	76.00	76.30	6	1.905256049
7	1996	67.20	76.00	7	1.882524538
8	1997	63.50	75.00	8	1.880813592
9	1998	53.60	75.00	9	1.875061263
10	1999	59.70	72.66	9	1.861295394
11	2000	56.60	71.50	11	1.854306042
12	2001	107.40	70.00	12	1.84509804
13	2002	84.00	68.50	13	1.835690571
14	2003	60.00	67.50	14	1.829303773
15	2004	66.80	67.20	15	1.827369273
16	2005	68.50	66.80	16	1.824776462
17	2006	75.00	65.00	17	1.812913357
18	2007	51.40	63.50	18	1.802773725
19	2008	80.40	62.40	19	1.79518459
20	2009	65.00	62.00	20	1.792391689
21	2010	60.00	61.10	21	1.78604121
22	2011	58.00	60.40	22	1.781036939
23	2012	67.50	60.00	23	1.77815125
24	2013	98.60	59.70	24	1.775974331
25	2014	75.00	58.00	25	1.763427994
26	2015	54.00	56.60	26	1.752816431
27	2016	62.00	55.00	27	1.740362689
28	2017	62.40	54.00	28	1.73239376
29	2018	87.00	53.60	29	1.72916479
30	2019	51.50	51.40	30	1.710963119
31	2020	76.30	51.40	30	1.710963119
32	2021	61.10	47.00	32	1.672097858
33	2022	71.50	45.00	33	1.653212514
SUM			2240.90		60.13
MEAN			67.91		1.82
STANDARD DEVATION			15.24		0.09
SKEWNESS COEFICIENT			1.02		0.48

Table 4: - Data Check for variability

Data quality Check	
Number of data	33
Standard deviation	15.24
Standard error of mean	2.65
Relative standard (α)	**3.91 %**

3.91<10% Acceptable.

Table 5: - Determination of maximum and minimum rainfall out liar values

Outlier Test	
Kn	2.60
Higher Limit; YH=Ymean+Kn*бу	2.06
Higher Limit	115.46
Lower Limit; YL=Ymean- Kn*бу	1.58
Lower Limit	38.17
Minimum data from time series	45.00
Maximum data from time series	107.40

Outliers are considered based on the value of the skewness coefficient, which indicates the symmetry of the data distribution. If the skewness coefficient falls between -0.4 and +0.4, the distribution is approximately symmetrical, and both higher and lower outliers should be examined. However, if the skewness coefficient is less than -0.4 (indicating a left-skewed distribution), the lower outliers should be considered more carefully. Conversely, if the skewness coefficient is greater than +0.4 (indicating a right-skewed distribution), the focus should be on the higher outliers.

In this study, both lower and higher outliers were examined based on the skewness coefficient, but no data points were identified as outliers. Therefore, all the data can be considered valid for analysis.

Table 6: - Design storm analysis for areal rainfall

6.1	**Method**			**Gumbel's**		
	Return period T yrs	5	10	25	50	100
	Standard variate, Sn	1.12	1.12	1.12	1.12	1.12
	Standard mean, Yn	0.54	0.54	0.54	0.54	0.54
	yt	1.50	2.25	3.20	3.90	4.60
	Frequency factor, KT	0.86	1.54	2.39	3.02	3.64
	Y=Xmean + Z*XStd	81.07	91.32	104.27	113.87	123.41
6.2	**Method**			**Gumbel's extreme value**		
	Return period T yrs	5	10	25	50	100
	Standard variate, Sn	1.12	1.12	1.12	1.12	1.12
	Standard mean, Yn	0.54	0.54	0.54	0.54	0.54
	yt	1.50	2.25	3.20	3.90	4.60
	Frequency factor, KT	0.72	1.30	2.04	2.59	3.14
	Y=Xmean + Z*XStd	78.87	87.79	99.06	107.42	115.72
6.3	**Method**			**Log Pearson Type 3**		
	Design Period, T	5	10	25	50	100
	Probability, P	0.20	0.10	0.04	0.02	0.01
	K=(Cs/6)	0.08	0.08	0.08	0.08	0.08
	W=(Ln(1/P2))0.5	1.79	2.15	2.54	2.80	3.03
	Frequency Factor, KT	0.84	1.28	1.75	2.05	2.33
	Standard Normal Variance, Z	0.82	1.33	1.92	2.31	2.69
	Y=Ymean + Z*Y	1.90	1.95	2.00	2.03	2.07
	Design Rainfall, X= 10Y	78.98	88.13	99.81	108.39	117.49
6.4	**Method**			**Log Normal**		
	Design Period, T	5	10	25	50	100
	Probability, P	0.2	0.1	0.04	0.02	0.01
	K=(Cs/6)	0.08	0.08	0.08	0.08	0.08
	W=(Ln(1/P2))0.5	1.79	2.15	2.54	2.80	3.03
	Frequency Factor, KT	0.84	1.28	1.75	2.05	2.33
	Standard Normal Variance, Z	0.82	1.33	1.92	2.31	2.69
	Y=Ymean + Z*σy	1.90	1.95	2.00	2.03	2.07
	Design Rainfall, X= 10Y	78.98	88.13	99.81	106.29	117.49
6.5	**Method**			**Pearson Type 3 Distribution**		
	Design Period, T	5	10	25	50	100
	Probability, P	0.20	0.10	0.04	0.02	0.01
	K=(Cs/6)	0.08	0.08	0.08	0.08	0.08
	W=(Ln(1/P2))0.5	1.79	2.15	2.54	2.80	3.03
	Z	0.84	1.28	1.75	2.05	2.33
	Kt	0.82	1.33	1.91	2.31	2.65
	X=Xmean + KT*σx	80.36	88.20	97.01	103.06	108.37
6.6	**Method**			**Normal**		
	Design Period, T	5	10	25	50	100
	Probability, P	0.20	0.10	0.04	0.02	0.01
	K=(Cs/6)	0.17	0.17	0.17	0.17	0.17
	W=(Ln(1/P2))0.5	1.79	2.15	2.54	2.80	3.03
	KT	0.84	1.28	1.75	2.05	2.33
	X=Xmean + KT*σx	80.73	87.44	94.60	99.22	103.37

Table 7: - Goodness to fitness

7.1 Normal

Rank	XI	PR	w	KT	XI'	XI-'XI'
1	107.40	0.03	2.66	1.89	96.71	10.69
2	98.60	0.06	2.38	1.57	91.76	11.84
3	87.00	0.09	2.20	1.35	88.51	10.09
4	84.00	0.12	2.07	1.19	86.00	1.00
5	80.00	0.15	1.96	1.05	83.90	0.10
6	76.30	0.18	1.86	0.93	82.06	1.66
Sum						**35.38**
Sum/Xm						**0.52**

7.2 Log Normal

Rank	XI	PR	w	KT	Y	XI'	XI-'XI'
1	107.40	0.03	2.66	1.89	2.00	99.21	8.19
2	98.60	0.06	2.38	1.57	1.97	92.59	11.01
3	87.60	0.09	2.20	1.35	1.95	88.49	10.11
4	84.00	0.12	2.07	1.19	1.93	85.44	1.56
5	80.40	0.15	1.96	1.05	1.92	82.97	1.03
6	76.30	0.18	1.86	0.93	1.91	80.88	0.48
Sum							**32.38**
Sum/Xm							**0.48**

7.3 Log person Type III

PR	w	K	Z	KT	y	XI'	XI-'XI'
0.03	2.66	0.08	1.89	2.09	2.01	103.47	3.93
0.06	2.38	0.08	1.57	1.67	1.98	94.68	8.92
0.09	2.20	0.08	1.35	1.41	1.95	89.52	9.08
0.12	2.07	0.08	1.19	1.21	1.93	85.82	1.18
0.15	1.96	0.08	1.05	1.05	1.92	82.92	1.08
0.18	1.86	0.08	0.93	0.91	1.91	80.51	0.11
Sum							**24.31**
Sum/Xm							**0.36**

7.4 Person Type III

PR	w	K	Z	KT	XI'	XI-'XI'
0.03	2.66	0.02	1.89	1.93	97.31	10.09
0.06	2.38	0.02	1.57	1.59	92.09	11.51
0.09	2.20	0.02	1.35	1.36	88.70	9.90
0.12	2.07	0.02	1.19	1.19	86.09	0.91
0.15	1.96	0.02	1.05	1.05	83.91	0.09
0.18	1.86	0.02	0.93	0.93	82.03	1.63
Sum						**34.12**
Sum/Xm						**0.50**

7.5 Gumbel extreme value

PR	KT	XI'	XI-'XI'
0.03	2.29	102.78	4.62
0.06	1.74	94.36	9.24
0.09	1.41	89.35	9.25
0.12	1.17	85.74	1.26
0.15	0.98	82.90	1.10
0.18	0.83	80.53	0.13
Sum			25.60
Sum/Xm		0.38	

7.6 Gumbel

PR	YT	KT	XI'	XI-'XI'
0.04	3.24	2.42	104.77	2.63
0.03	3.43	2.59	107.40	3.80
0.03	3.58	2.72	109.43	10.83
0.02	3.71	2.84	111.23	24.23
0.02	3.83	2.95	112.93	28.93
0.02	3.96	3.06	114.61	34.21
Sum				**104.63**
Sum/Xm				**1.54**

7.7 D-Index’s

Rank	XI	Normal XI -'XI'	Log Pearson Type III XI -'XI'	Log Normal XI -'XI'	Pearson Type III XI -'XI'	Gumbel extreme value XI -'XI'	Gumbel XI -'XI'
1	107.40	10.69	3.93	8.19	10.09	4.62	2.63
2	98.60	11.84	8.92	11.01	11.51	9.24	3.80
3	87.60	10.09	9.08	10.11	9.90	9.25	10.83
4	84.00	1.00	1.18	1.56	0.91	1.26	24.23
5	80.00	0.10	1.08	1.03	0.09	1.10	28.93
6	76.30	1.66	0.11	0.48	1.63	0.13	34.21
Sum		35.38	24.31	32.38	34.12	25.60	104.63
Sum/Mean		0.52	0.36	0.48	0.50	0.38	**1.54**
Point rainfall		108.39	108.39	108.39	103.06	107.42	**113.87**
Design of areal rainfall					**113.87**		

Table 8: - Out let discharge of Rasa diversion head work after calibration and validation with in time step

Project: Rasa DHW Simulation Run: 50 yr frequency storm
Sink: Sink-1

Start of Run: 12Jun2024, 00:00
End of Run: 13Jun2024, 00:00
Compute Time: 20Jun2024, 23:45:42

Basin Model: Basin model
Meteorologic Model: Meteorological model
Control Specifications: Control specification

Date	Time	Inflow from R1 (M3/S)	Inflow from S3 (M3/S)	Total Inflow (M3/S)
12Jun2024	13:08	46.6	12.1	58.7
12Jun2024	13:09	46.7	12.1	58.8
12Jun2024	13:10	46.8	12.1	58.9
12Jun2024	13:11	46.9	12.1	59.0
12Jun2024	13:12	47.0	12.1	59.1
12Jun2024	13:13	47.0	12.1	59.1
12Jun2024	13:14	47.1	12.1	59.2
12Jun2024	13:15	47.1	12.1	59.2
12Jun2024	13:16	47.1	12.1	59.2
12Jun2024	13:17	47.2	12.1	59.2
12Jun2024	13:18	47.1	12.1	59.2
12Jun2024	13:19	47.1	12.0	59.2
12Jun2024	13:20	47.1	12.0	59.1
12Jun2024	13:21	47.0	12.0	59.1
12Jun2024	13:22	47.0	12.0	59.0
12Jun2024	13:23	46.9	12.0	58.9
12Jun2024	13:24	46.8	12.0	58.8
12Jun2024	13:25	46.7	12.0	58.7
12Jun2024	13:26	46.6	12.0	58.6
12Jun2024	13:27	46.5	12.0	58.5
12Jun2024	13:28	46.4	11.9	58.3
12Jun2024	13:29	46.2	11.9	58.2
12Jun2024	13:30	46.1	11.9	58.0

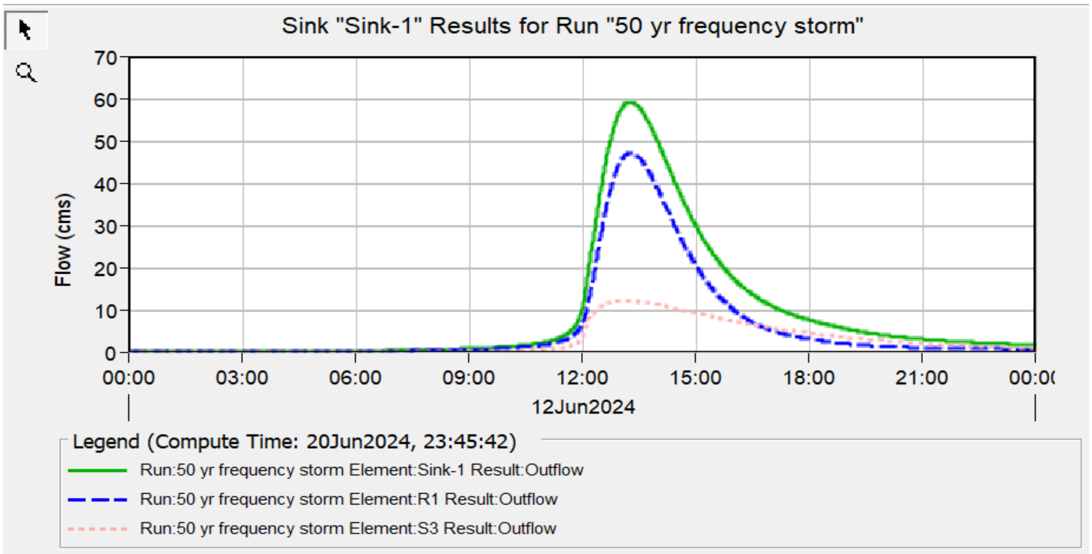


Figure 1:-HEC-HMS simulation hydrograph of Rasa diversion headwork

Table 9: - Out let discharge of Wodesa diversion head work after calibration and validation with in time step

Project: Wodesa DHW   Simulation Run: Frequency storm of wodesa
Sink: Sink-1

Start of Run: 12Jun2024, 00:00   Basin Model: Basin model
End of Run: 14Jun2024, 00:00   Meteorologic Model: Meteorological model
Compute Time: 20Jun2024, 21:25:16   Control Specifications: Control specification

Date	Time	Inflow from R1 (M3/S)	Inflow from S2 (M3/S)	Total Inflow (M3/S)
12Jun2024	12:00	3.3	2.1	5.4
12Jun2024	12:30	10.8	10.7	21.5
12Jun2024	13:00	33.5	12.4	46.0
12Jun2024	13:30	59.2	12.7	72.0
12Jun2024	14:00	75.6	12.4	88.0
12Jun2024	14:30	84.0	11.9	95.8
12Jun2024	15:00	87.0	11.2	98.2
12Jun2024	15:30	86.6	10.4	97.0
12Jun2024	16:00	84.0	9.6	93.6
12Jun2024	16:30	80.0	8.9	88.9
12Jun2024	17:00	75.3	8.1	83.5
12Jun2024	17:30	70.3	7.4	77.7
12Jun2024	18:00	65.1	6.8	71.9
12Jun2024	18:30	60.1	6.2	66.3
12Jun2024	19:00	55.2	5.6	60.8
12Jun2024	19:30	50.5	5.0	55.5
12Jun2024	20:00	46.1	4.6	50.6
12Jun2024	20:30	42.0	4.1	46.1
12Jun2024	21:00	38.2	3.7	41.9
12Jun2024	21:30	34.7	3.4	38.1

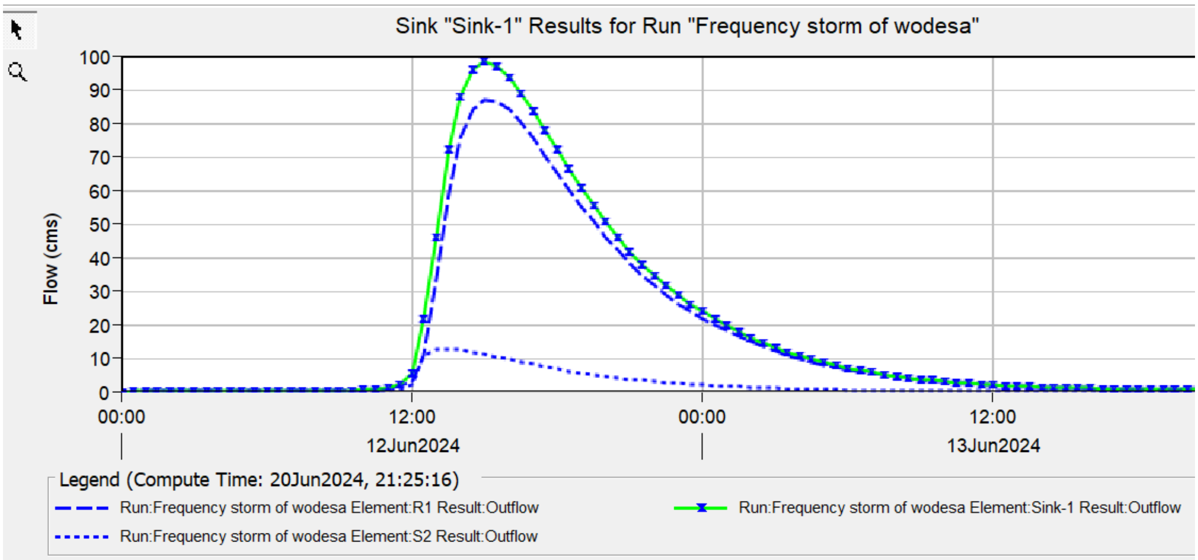


Figure 2:-HEC-HMS simulation hydrograph of wodesa diversion headwork

Appendix (B) Hydraulic analysis

Table 10: - Rivers longitudinal profile determination

Rasa river				
Change	Distance (m)	Elevation(m)	Elevation difference	Slope (m/m)
0	0	1812.5	-	-
5	5	1812.36	0.14	2.80%
5	10	1811.92	0.44	4.40%
5	15	1811.67	0.25	1.67%
5	20	1811.1	0.57	2.85%
5	25	1810.82	0.28	1.12%
5	30	1810.12	0.7	2.33%
5	35	1809.58	0.54	1.54%
5	40	1809.5	0.08	0.20%
Average slope of Rasa river			**2.11%**	
Wodesa river				
Change	Distance (m)	Elevation (m)	Elevation difference	Slope (m/m)
0	0	1787.28	-	-
5	5	1786.96	0.32	6.40%
5	10	1786.62	0.34	3.40%
5	15	1785.9	0.72	4.80%
5	20	1785.63	0.27	1.35%
5	25	1785.05	0.58	2.32%
5	30	1784.42	0.63	2.10%
5	35	1783.59	0.83	2.37%
5	40	1783.21	0.38	0.95%
Average slope of Wodesa river			**2.96%**	

(Source: Author, 2024)

Table 11: - Rivers Cross sectional profile

Rasa river			Wodesa river		
Distance (m)	Elevation (m)	Roughness coefficient (n)	Distance (m)	Elevation (m)	Roughness coefficient (n)
0	1822.68	0.045	0	1797	0.034
5	1819.14	0.045	5	1796	0.034
8	1818.41	0.045	10	1795.5	0.034
12	1816.66	0.045	15	1794.5	0.034
16	1816.16	0.045	20	1794.2	0.034
17	1815.65	0.045	25	1794	0.034
19	1813.27	0.045	30	1793.5	0.034
20	1813.04	0.04	35	1793.4	0.034
22	1811.04	0.04	40	1793.2	0.034
23	1811.01	0.04	45	1793	0.034
24	1811.05	0.04	50	1792	0.034
25	1813.12	0.04	55	1791	0.04
27	1813.16	0.04	60	1790	0.04
30	1813.77	0.04	65	1789	0.04
31	1814.54	0.04	70	1787.28	0.04
34	1816.31	0.04	75	1787.8	0.04
35	1816.88	0.04	80	1789.6	0.04
40	1819.56	0.04	85	1790.6	0.04
45	1820.67	0.04	90	1791.2	0.045
50	1821.32	0.04	95	1791.8	0.045
			100	1792	0.045
			105	1792.8	0.045

(Source: Author, 2024)

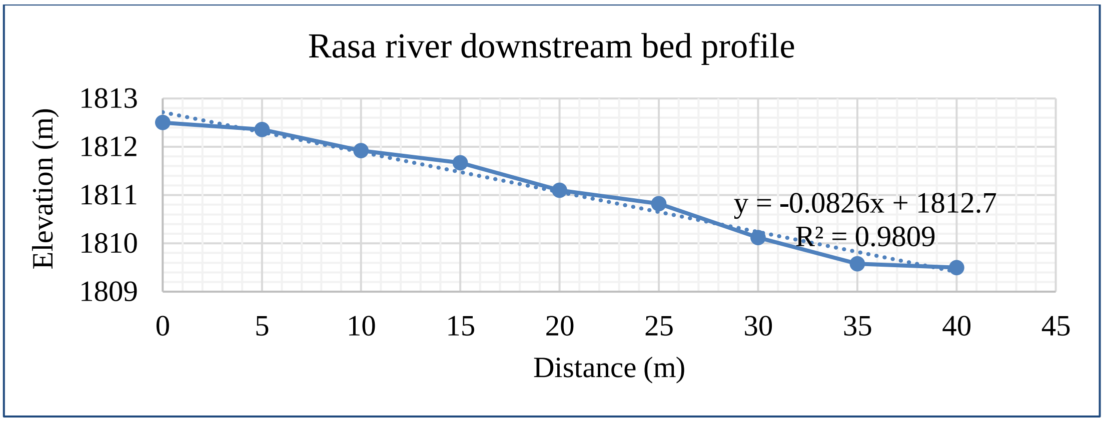


Figure 3:- Longitudinal profiles for downstream of rasa river at diversion site

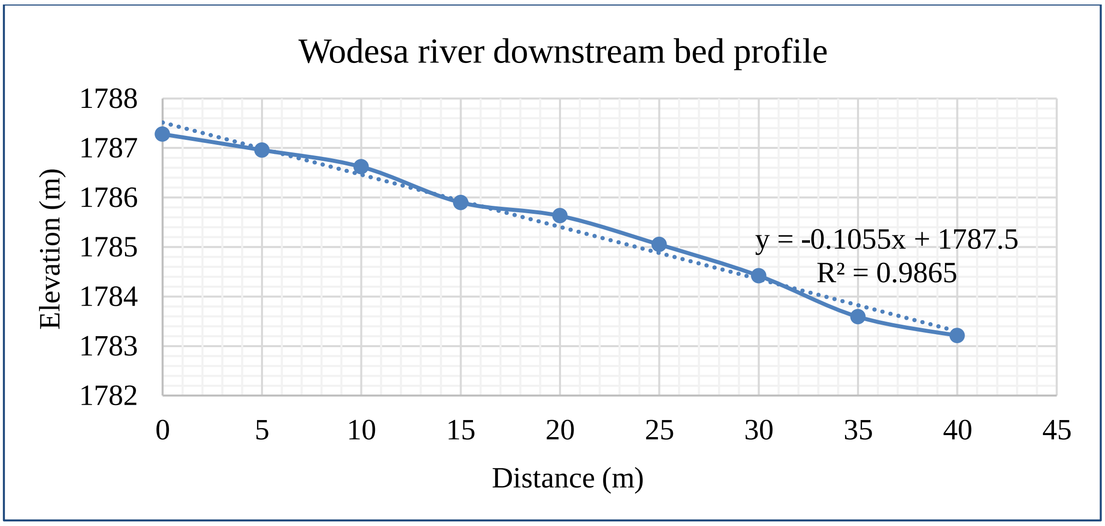


Figure 4:- Longitudinal profiles for downstream of wodesa river at diversion site

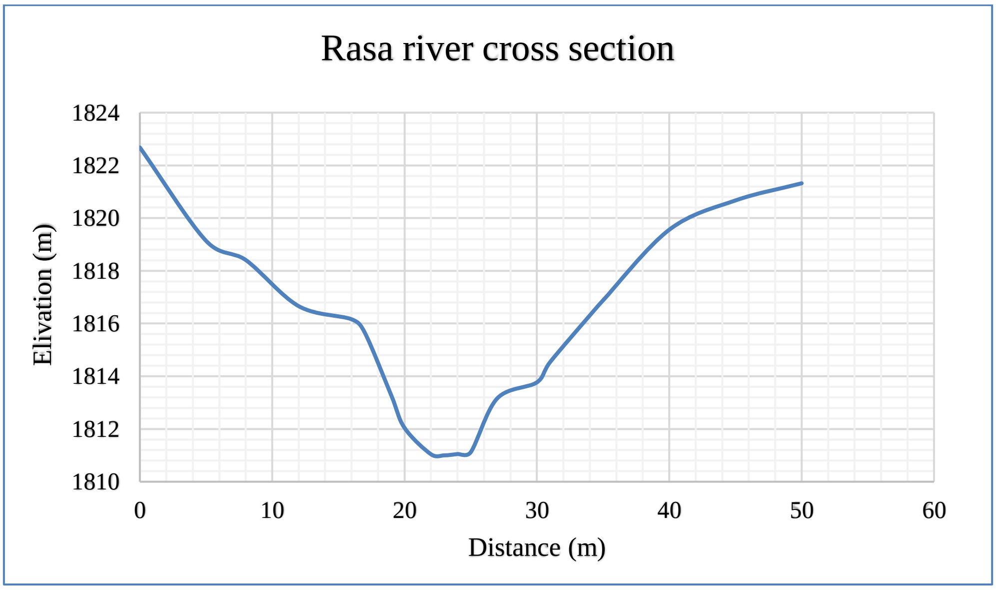


Figure 5:- Cross sectional profiles for downstream of rasa river at diversion site

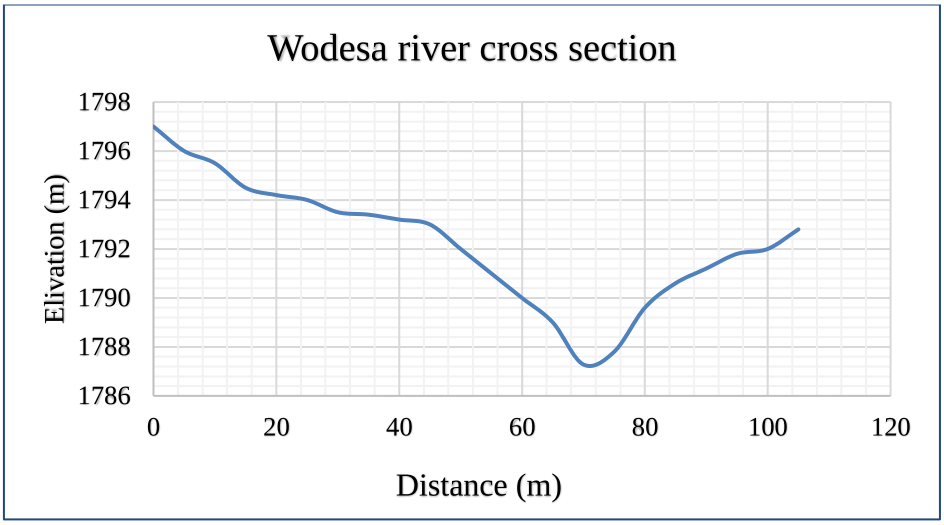


Figure 6:- Cross sectional profiles for downstream of wodesa river at diversion site

Table 12: - Rating curve parameter determination for Rasa rivers at d/s of diversion site

Elevation (m)	Depth (m)	Discharge (m^3/s)	Velocity (m/s)	Flow Area (m^2)	Perimeter (m)	Top Width (m)
1,813.61	2.4	87.43	5.28	16.6	12.6	10.98
1,813.41	2.2	72.12	5	14.4	11.9	10.43
1,813.21	2	59.06	4.76	12.4	11	9.67
1,813.01	1.8	49.23	4.65	10.6	9.7	8.52
1,812.81	1.6	39.09	4.37	8.9	9	7.94
1,812.61	1.4	30.21	4.08	7.4	8.3	7.35
1,812.41	1.2	22.55	3.76	6	7.6	6.77
1,812.21	1	16.05	3.41	4.7	6.9	6.19
1,812.01	0.8	10.96	3.09	3.5	6	5.43
1,811.81	0.6	6.97	2.75	2.5	5.1	4.66
1,811.61	0.4	3.99	2.38	1.7	4.2	3.88
1,811.41	0.2	1.87	1.92	1	3.4	3.17
1,811.21	0	0.52	1.27	0.4	2.6	2.53

(Source: Author, 2024)

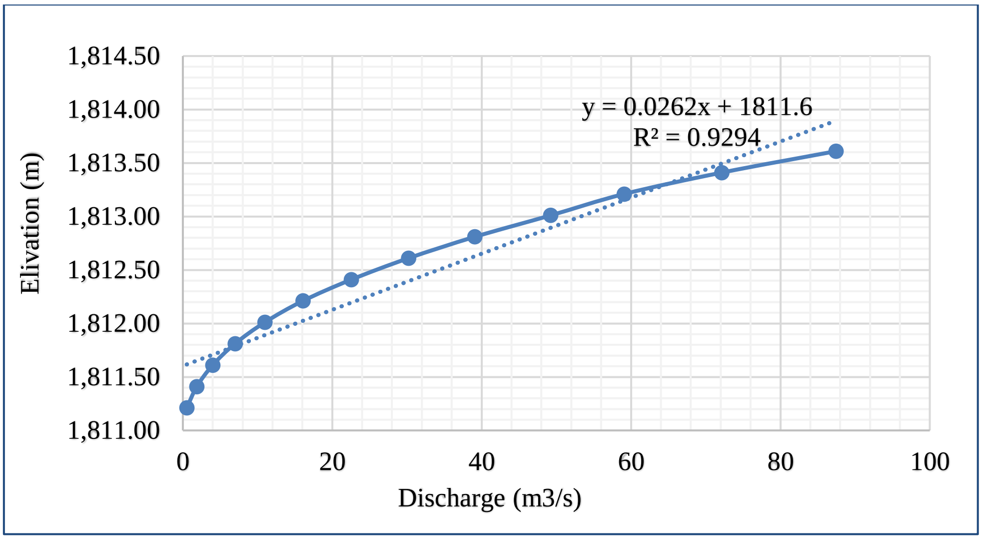


Figure 6:- Rating curve developed for Rasa river at downstream of diversion site

Table 13: - Rating curve parameter determination for wodesa rivers at d/s of diversion site

Elevation (m)	Depth (m)	Discharge (m^3/s)	Velocity (m/s)	Flow Area (m^2)	Perimeter (m)	Top Width (m)
1,789.78	2.5	133.53	5.06	26.4	20.5	19.8
1,789.28	2	80.58	4.57	17.6	16.1	15.51
1,788.78	1.5	42.1	3.9	10.8	12.5	12.08
1,788.28	1	16.28	2.97	5.5	9.5	9.24
1,787.78	0.5	2.64	1.69	1.6	6.4	6.26
1,787.28	0	0	0	0	0	0

(Source: Author, 2024)

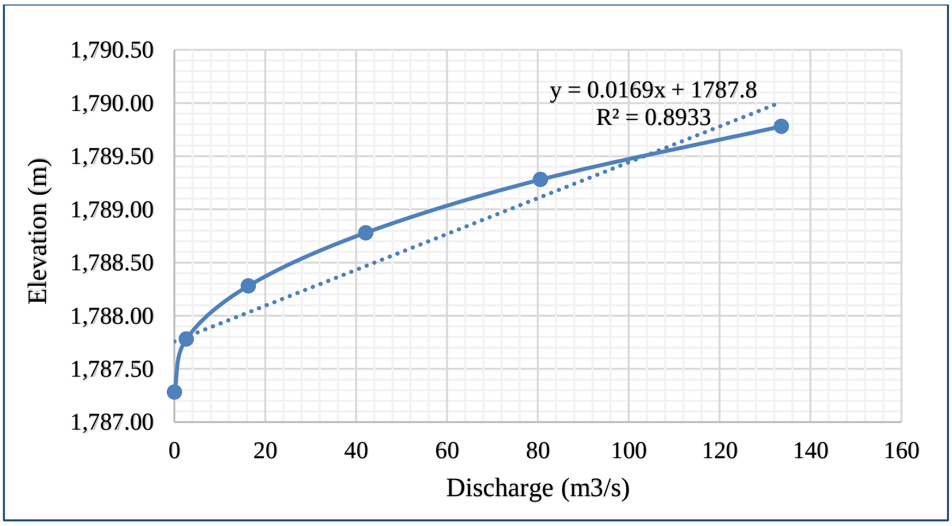


Figure 7:- Rating curve developed for wodesa river at downstream of diversion site

Table 14: - Actual parameters of headwork structures

S/no	Parameters	Units	Rasa diversion headwork	Wodesa diversion headwork
1	**Weir dimensions**			
1.1	Height (P)	m	0.4	1
1.2	Length (L)	m	7	10
1.3	Top width(b)	m	1	1.5
1.4	Bottom width (B)	m	1.8	3
2	**Under sluice section**			
2.1	Width(w)	m	0.5	1
3	**Impervious length**			
3.1	Total impervious length	m	8.8	14.8
3.2	D/s concrete floor length	m	4	8
4	**Cutoff piles**			
4.1	U/s cutoff pile	m	1	1
4.2	D/s cutoff pile	m	1.75	2
5	**Wall Dimension**			
5.1	Top Width	m	0.5	0.5
5.2	Bottom width	m	2	2.45
5.3	U/s Height of wing wall	m	3	4
5.4	D/s Height of wing wall	m	2.5	2.9
5.5	Foundation depth	m	1	1

Appendix (C) Structural analysis

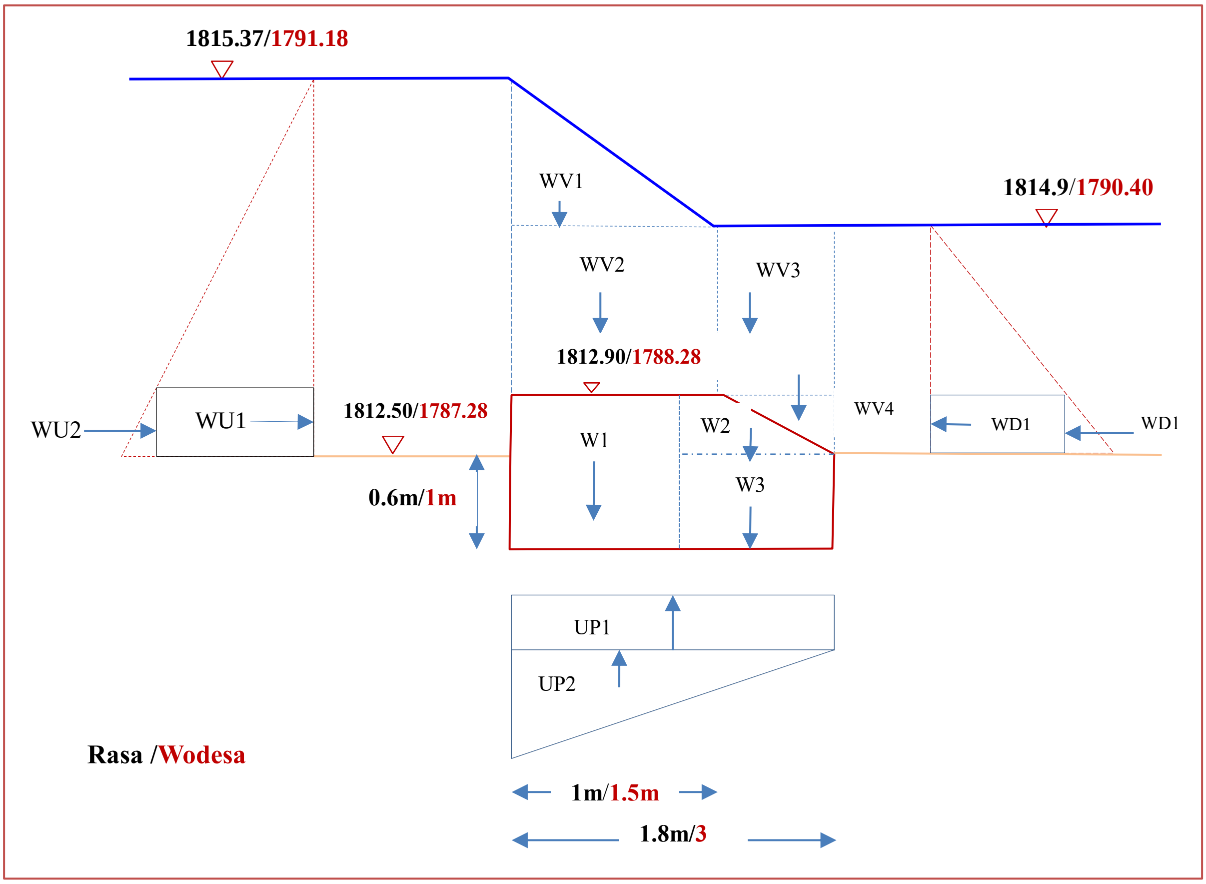


Figure 8:- Forces acting on weir body for Rasa and Wodesa diversion headwork

For the analysis of stability, the following parameters value and symbols are adopted:

- Unit weight of Concrete = 24KN/m^3
- Unit weight of Water = 9.81KN/m^3
- Allowable soil bearing pressure for clay loam soil = 88KN/m^2
- Allowable soil bearing pressure for sandy loam soil = 100KN/m^2
- W1, W2, W3, - Weight of the weir body and foundation
- Ws1, Ws2, Ws3, - Seismic load on the weir body and foundation
- WV1, WV2, WV3 and WV4 – weight of water wedge
- WU1 & WU2 upstream static forces, and WD1& WD2 downstream water force
- Up1 and Up2 Uplift pressure

Table 15: - Stability analysis of weir body for Rasa diversion headwork in both conditions

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+Ve	-Ve	+Ve	-Ve		+Ve	-Ve
1	**Self-weights**							
	W1↓	24	-	-	-	1.3	31.20	-
	W2↓	3.84	-	-	-	0.53	2.04	-
	W3↓	11.52	-	-	-	0.4	4.61	-
2	**Water load**							
	High flood level/overflow condition							
	WV1↓	4.19	-	-	-	1.47	6.15	-
	WV2↓	15.89	-	-	-	1.3	20.66	-
	WV3↓	12.71	-	-	-	0.4	5.09	-
	WV4↓	1.57	-	-	-	0.27	0.42	-
	WU1→	-	-	-	8.91	0.2	-	1.78
	WU2→	-	-	-	0.78	0.13	-	0.10
	WD1←	-	-	6.36	-	0.2	1.27	-
	WD2←	-	-	0.78	-	0.13	0.10	-
	Pond level /No flow condition							
	WU2→	-	-	-	0.78	0.13	-	0.10
3	**Uplift forces**							
	High flood level/overflow condition							
	UP1↑	-	35.67	-	-	0.9	-	32.10
	UP2↑	-	7.50	-	-	1.2	-	9.01
	Pond level /No flow condition							
	UP2↑	-	3.53	-	-	1.2	-	4.24

Table 16: - Stability analysis of weir body for Wodesa diversion headwork in both conditions

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+ve	-ve	+ve	-ve		+ve	-ve
1	**Self-weights**							
	W1↓	72	-	-	-	2.25	162	-
	W2↓	18	-	-	-	1	18	-
	W3↓	36	-	-	-	0.75	27	-
2	**Seismic load**							
	Ws1↑	-	7.2	-	-	2.25	-	16.2
	Ws2↑	-	1.8	-	-	1	-	1.8
	Ws3↑	-	3.6	-	-	0.75	-	2.7
	Ws1→	-	-	-	7.2	2.25	-	16.2
	Ws2→	-	-	-	1.8	1	-	1.8
	Ws3→	-	-	-	3.6	0.75	-	2.7
3	**Water load**							
	High flood level/overflow condition							
	WV1↓	5.76	-	-	-	2.5	14.41	-
	WV2↓	31.20	-	-	-	2.25	70.19	-
	WV3↓	31.20	-	-	-	0.75	23.40	-
	WV4↓	7.36	-	-	-	0.5	3.68	-
	WU1→	-	-	-	28.45	0.5	-	14.22
	WU2→	-	-	-	4.91	0.33	-	1.62
	WD1←	-	-	20.80	-	0.5	10.40	-
	WD2←	-	-	4.91	-	0.33	1.62	-
	Pond level /No flow condition							
	WU2→	-	-	-	4.91	0.33	-	1.62
4	**Uplift forces**							
	High flood level/overflow condition							
	UP1↑	-	91.82	-	-	1.5	-	137.73
	UP2↑	-	11.48	-	-	2	-	22.96
	Pond level /No flow condition							
	UP2↑	-	14.72	-	-	2	-	29.43

Table 17: - Sum of forces & moments for Rasa weir body at high and low flow conditions

High flood level		**No flow condition**	
ΣM+	71.54	ΣM+	37.84
ΣM-	42.99	ΣM-	4.34
ΣMnet	28.55	ΣMnet	33.50
ΣFV+	73.72	ΣFV+	39.36
ΣFV-	43.17	ΣFV-	3.53
ΣFVnet	30.55	ΣFVnet	35.83
ΣFH+	7.14	ΣFH+	0
ΣFH-	9.69	ΣFH-	0.78
ΣFHnet	-2.55	ΣFHnet	-0.78

Table 18: - Sum of forces & moments for Wodesa weir body at high and low flow conditions.

High flood level		**No flow condition**	
ΣM+	330.69	ΣM+	207.00
ΣM-	217.93	ΣM-	72.45
ΣMnet	112.76	ΣMnet	134.55
ΣFV+	201.51	ΣFV+	126
ΣFV-	115.90	ΣFV-	27.32
ΣFVnet	85.61	ΣFVnet	98.69
ΣFH+	25.70	ΣFH+	0
ΣFH-	45.95	ΣFH-	17.51
ΣFHnet	-20.25	ΣFHnet	-17.51

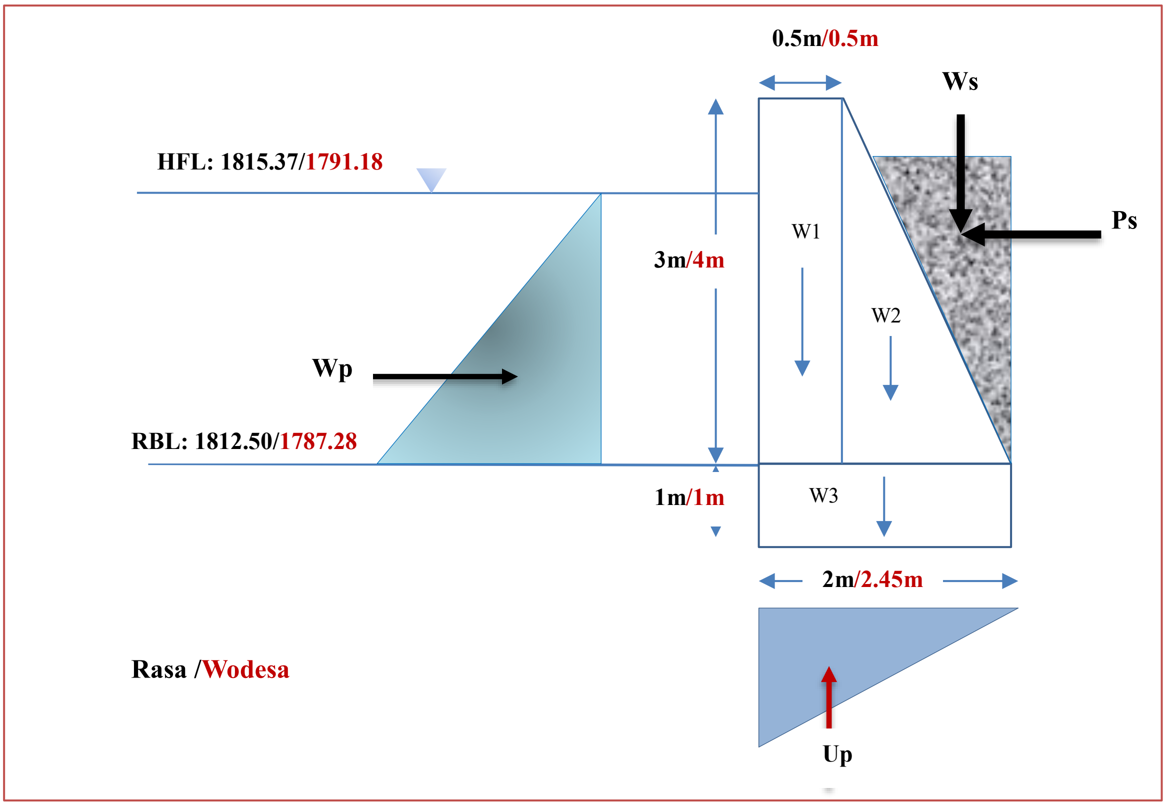


Figure 9:- Forces acting on wing wall for Rasa and Wodesa diversion headwork

Table 19: - Stability analysis of U/s retaining wall for Rasa diversion head work

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+ve	-ve	+ve	-ve		+ve	-ve
1	**Self-weights**							
	W1	36	-	-	-	1.5	54	-
	W2	45	-	-	-	0.83	37.35	-
	W3	42	-	-	-	0.88	36.96	-
2	**Water load**							
	High flood level/overflow condition							
	Wp→	-	-	-	40.40	0.96	-	38.79
	Pond level /No flow condition							
	Wp→	-	-	-	0.78	0.13	-	0.10
3	**Uplift forces**							
	High flood level/overflow condition							
	UP1	-	24.64	-	-	1.17	-	28.82
	Pond level /No flow condition							
	UP1	-	3.49		-	1.17	-	4.09
4	**Soil pressure**							
	Ws↓	42.75	-	-	-	0.42	17.96	-
	Ps←	-	-	25.65	-	2	51.3	-

Table 20: - Stability analysis of D/s retaining wall for Rasa diversion head work

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+ve	-ve	+ve	-ve		+ve	-ve
1	**Self-weights**							
	W1	36	-	-	-	1.5	54	-
	W2	45	-	-	-	0.83	37.35	-
	W3	42	-	-	-	0.88	36.96	-
2	**Water load**							
	High flood level/overflow condition							
	Wp→	-	-	-	20.01	0.96		19.21
	Pond level /No flow condition							
	Wp→	-	-	-	0.78	0.13	-	0.10
3	**Uplift forces**							
	High flood level/overflow condition							
	UP1	-	17.34			1.17		20.29
	Pond level /No flow condition							
	UP1	-	3.49		-	1.17	-	4.09
4	**Soil pressure**							
	Ws↓	42.75	-	-	-	0.42	17.96	-
	Ps←	-	-	25.65	-	2	51.3	-

Table 21: - Sum of forces & moment for U/s retaining wall of Rasa diversion head work

High flood level		**No flow condition**	
ΣM+	197.57	ΣM+	36.96
ΣM-	67.61	ΣM-	4.19
ΣMnet	**129.96**	ΣMnet	**32.77**
ΣFV+	165.75	ΣFV+	123
ΣFV-	24.64	ΣFV-	3.49
ΣFVnet	**141.11**	ΣFVnet	**119.51**
ΣFH+	25.65	ΣFH+	0
ΣFH-	40.40	ΣFH-	0.78
ΣFHnet	**-14.75**	ΣFHnet	**-0.78**

Table 22: - Sum of forces & moment for D/s retaining wall of Rasa diversion head work

High flood level		**No flow condition**	
ΣM+	197.57	ΣM+	36.96
ΣM-	39.50	ΣM-	4.19
ΣMnet	**158.06**	ΣMnet	**32.77**
ΣFV+	165.75	ΣFV+	123
ΣFV-	17.34	ΣFV-	3.49
ΣFVnet	**148.41**	ΣFVnet	**119.51**
ΣFH+	25.65	ΣFH+	0
ΣFH-	20.01	ΣFH-	0.78
ΣFHnet	**5.64**	ΣFHnet	**-0.78**

Table 23: - Factor of Safety for Rasa Upstream retaining wall

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	2.92	8.83	Should be > 1.5	ok
2	Stability against sliding	0.10	0.01	Should be < 0.65	ok
3	Stability against tension				
	$X_{average}$	0.72	0.29		
	e	0.05	0.71		
	B/6	0.33	0.33		
	Overstress/Tension	ok	Not safe	e < B/6	ok
4	Stability against contact pressure < 88KN/m^2				
	Contact pressure maximum (KN/m^2)	93.33	191.46		ok
	Contact pressure minimum (KN/m^2)	67.94	-69.69		ok

Table 24: - Factor of Safety for Rasa Downstream retaining wall

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	5.00	8.83	Should be > 1.5	ok
2	Stability against sliding	0.04	0.01	Should be < 0.65	ok
3	Stability against tension				
	$X_{average}$	1.07	0.27		
	e	0.19	0.60		
	B/6	0.29	0.29		
	Overstress/Tension	ok	Not safe	e < B/6	ok
4	Stability against contact pressure				
	Contact pressure maximum (KN/m2)	140.06	208.95		ok
	Contact pressure minimum (KN/m2)	29.55	-72.37		ok

Table 25: - Stability analysis of U/s retaining wall for wodesa diversion head work

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+ve	-ve	+ve	-ve		+ve	-ve
1	**Self-weights**							
	W1	48	-	-	-	1.45	69.6	-
	W2	57.6	-	-	-	0.8	46.08	-
	W3	40.8	-	-	-	0.85	34.68	-
2	**Water load**							
	High flood level/overflow condition							
	Wp→	-	-	-	74.61	1.3	-	96.99
	Pond level /No flow condition							
	Wp→	-	-	-	4.905	0.33	-	1.62
3	**Uplift forces**							
	High flood level/overflow condition							
	UP1	-	32.52	-	-	1.13	-	36.75
	Pond level /No flow condition							
	UP1	-	8.34	-	-	1.13	-	9.42
4	**Soil pressure**							
	Ws↓	45.6	-	-	-	0.4	18.24	-
	Ps←	-	-	45.6	-	2.67	121.75	-

Table 26: - Stability analysis of D/s retaining wall for wodesa diversion head work

S/n	Force descriptions	Magnitude of force (KN)				Moment arm (m)	Moment at toe (KN.m)	
		Vertical		Horizontal				
		+ve	-ve	+ve	-ve		+ve	-ve
1	**Self-weights**							
	W1	48	-	-	-	1.45	69.6	-
	W2	57.6	-	-	-	0.8	46.08	-
	W3	40.8	-	-	-	0.85	34.68	-
2	**Water load**							
	High flood level/overflow condition							
	Wp→	-	-	-	47.75	1.3		62.07
	Pond level /No flow condition							
	Wp→	-	-	-	4.905	0.33	-	1.62
3	**Uplift forces**							
	High flood level/overflow condition							
	UP1	-	26.02			1.13		29.40
	Pond level /No flow condition							
	UP1	-	8.34			1.13	-	9.42
4	**Soil pressure**							
	Ws↓	45.6	-	-	-	0.4	18.24	-
	Ps←		-	45.6	-	2.67	121.75	-

Table 27: - Sum of forces & moment for U/s retaining wall of Wodesa at high and low flow conditions.

High flood level		**No flow condition**	
ΣM+	290.35	ΣM+	150.36
ΣM-	133.73	ΣM-	11.04
ΣMnet	**156.62**	ΣMnet	**139.32**
ΣFV+	192.00	ΣFV+	146.4
ΣFV-	32.52	ΣFV-	8.34
ΣFVnet	**159.48**	ΣFVnet	**138.06**
ΣFH+	45.60	ΣFH+	0
ΣFH-	74.61	ΣFH-	4.91
ΣFHnet	**-29.01**	ΣFHnet	**-4.91**

Table 28: - Sum of forces & moment for D/s retaining wall of Wodesa at high and low flow conditions.

High flood level		**No flow condition**	
ΣM+	290.35	ΣM+	150.36
ΣM-	91.47	ΣM-	11.04
ΣMnet	**198.88**	ΣMnet	**139.32**
ΣFV+	192.00	ΣFV+	146.4
ΣFV-	26.02	ΣFV-	8.34
ΣFVnet	**165.98**	ΣFVnet	**138.06**
ΣFH+	45.60	ΣFH+	0
ΣFH-	47.75	ΣFH-	4.91
ΣFHnet	**-2.15**	ΣFHnet	**-4.91**

Table 29: - Factor of Safety for Wodesa Upstream retaining wall

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	2.17	13.62	Should be > 1.5	ok
2	Stability against sliding	0.18	0.04	Should be < 0.65	ok
3	Stability against tension				
	$X_{average}$	0.98	1.01		
	e	0.13	0.16		
	B/6	0.28	0.28		
	Overstress/Tension	ok	ok	e < B/6	ok
4	Stability against contact pressure				
	Contact pressure maximum (KN/m2)	137.53	126.82		ok
	Contact pressure minimum (KN/ m2)	50.09	35.61		ok

Table 30: - Factor of Safety for Wodesa Downstream retaining wall

S/n	Factor of safety	HFC	NFC	Satisfactory condition	Check
1	Stability against overturning	3.17	13.62	Should be > 1.5	ok
2	Stability against sliding	0.01	0.04	Should be < 0.65	ok
3	Stability against tension				
	$X_{average}$	1.20	1.01		
	e	0.35	0.16		
	B/6	0.28	0.28		
	Overstress/Tension	Not safe	ok	e < B/6	ok
4	Stability against contact pressure				
	Contact pressure maximum (KN/m2)	217.63	126.82		ok
	Contact pressure minimum (KN/ m2)	-22.35	35.61		ok

Appendix (D) Seepage pressure analysis

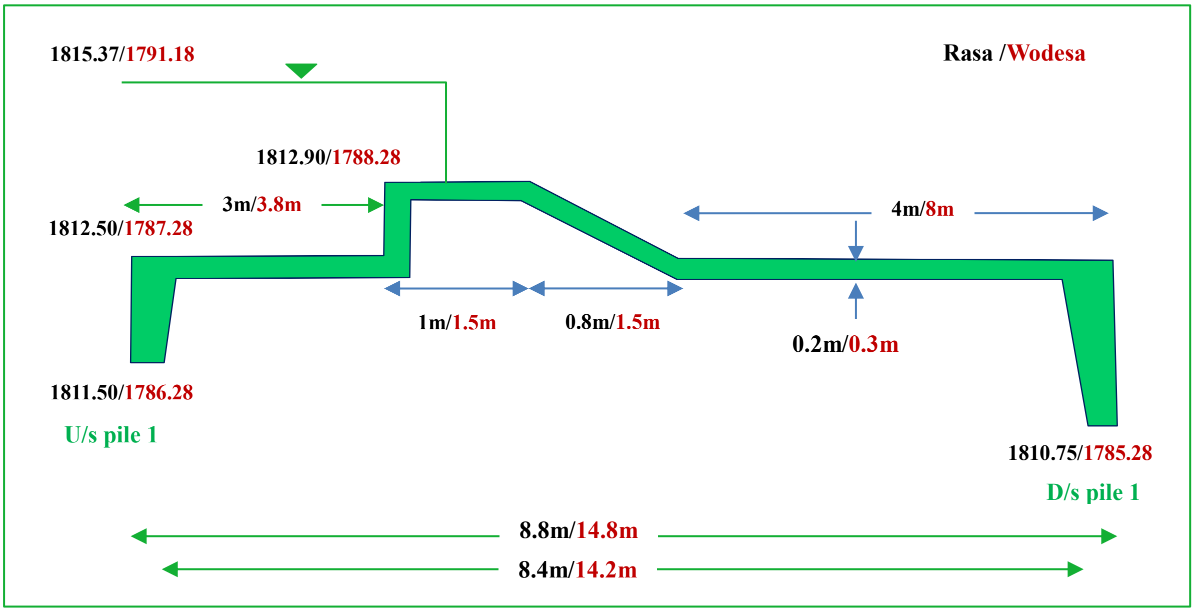
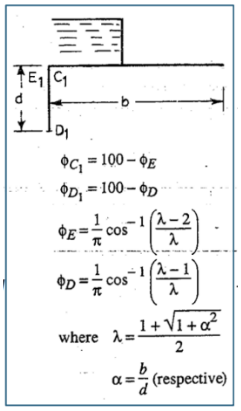


Figure 10:-Weir profile for Khosla's theory

Table 31: - Rasa diversion weir seepage analysis by using Khosla's theory

Rasa diversion weir seepage analysis			
1	**For U/s pile line**		
	Total floor length (b)		8.8
	Depth of U/s pile line (d)		1
		α= b/d	8.8
		1/α	0.11
		λ	4.93
	From curve plate		
		ΦEo	31.14
		ΦDo	21.52
		ΦE1	100
		ΦC1	68.86
		ΦD1	78.48
	Therefore, correction is required at ΦC1		
	Correction required for ΦC1		



1.1	**Due to floor thickness**	
	Thickens of floor	0.2
	Ct=(ΦD1-ΦC1)*t/d1	**1.92**
	Correction is +ve b/c it's in the drxn of flow	

1.2 Due to mutual interference of D/s pile on pile U/s pile

Depth of U/s pile line (d)	1	
Depth of D/s pile line (D)	1.75	
Distance b/n piles (b')	8.4	
Correction	**2.71**	

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

Correction is +ve b/c its in the drxn of flow

1.3 Due to slope

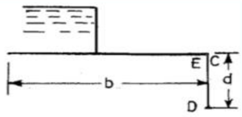
Cs **0**

Correction is 0 b/c no slope at U/s pile

Corrected ΦC1 **73.49**

2 For D/s pile line

Total floor length (b)		8.8
Depth of D/s pile line (d)		1.75
	α= b/d	5.03
	1/α	0.20
	λ	3.06
From curve plate		
	ΦE2	38.74
	ΦD2	26.5
	ΦC2	0



$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

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

$\alpha = \frac{b}{d}$ (respective)

Therefore, correction is required at ΦE2

Correction required for ΦE2

2.1 Due to floor thickness

Ct=(ΦE2-ΦD2) *t/d2 **1.40**

Correction is -ve Since the pressure observed from curve at E2' more than E2 it's in the drxn of flow

2.2 Due to mutual interference of U/s pile on pile D/s pile

Depth of U/s pile line (D)	1	
Depth of D/s pile line (d)	1.75	
Distance b/n piles (b')	8.4	
Corrected ΦC1	**2.05**	

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

Correction is -ve b/c it's in the opposite drxn of flow

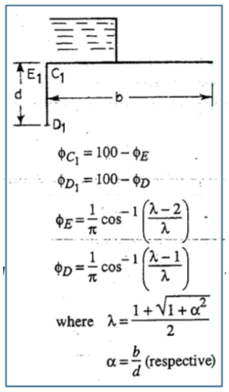
2.3 Due to slope

Cs **0**

	Correction is 0 b/c no slope at D/s pile	
	Corrected ΦE2	**35.29**

Table 32: - Wodesa diversion weir seepage analysis by using Khosla's theory

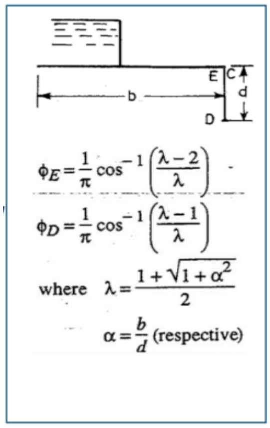
Wodesa diversion weir seepage analysis		
1	**For U/s pile line**	
	Total floor length (b)	14.8
	Depth of U/s pile line (d)	1
	α= b/d	14.8
	1/α	0.07
	λ	7.92
	From curve plate	
	ΦEo	23.13
	ΦDo	16.17
	ΦE1	100
	ΦC1	76.87
	ΦD1	83.83
	Therefore, correction is required at ΦC1	
	Correction required for ΦC1	
1.1	**Due to floor thickness**	
	Thickens of floor	0.3
	Ct=(ΦD1-ΦC1) *t/d1	**2.09**
	Correction is +ve b/c it's in the drxn of flow	
	Due to mutual interference of D/s pile on pile	
1.2	**U/s pile**	
	Depth of U/s pile line (d)	1
	Depth of D/s pile line (D)	2
	Distance b/n piles (b')	14.2
	Correction	**1.45**
	Correction is +ve b/c it's in the drxn of flow	
1.3	**Due to slope**	
	Cs	**0**
	Correction is 0 b/c no slope at U/s pile	
	Corrected ΦC1	**80.40**



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

2 For D/s pile line

Total floor length (b)	14.8
Depth of D/s pile line (d)	2
$\alpha = b/d$	7.40
$1/\alpha$	0.14
λ	4.23
From curve plate	
$\Phi E2$	32.32
$\Phi D2$	22.34
$\Phi C2$	0



Therefore, correction is required at ΦE2

Correction required for ΦE2

2.1 Due to floor thickness

Ct=(ΦE2-ΦD2) *t/d2	**1.50**

Correction is -ve Since the pressure observed from curve at E2' more than E2 it's in the drxn of flow

2.2 Due to mutual interference of U/s pile on pile D/s pile

Depth of U/s pile line (D)	1
Depth of D/s pile line (d)	2
Distance b/n piles (b')	14.4
Correction	**1.01**

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

Correction is -ve b/c it's in the opposite drxn of flow

2.3 Due to slope

Cs	**0**

Correction is 0 b/c no slope at U/s pile

Corrected ΦE2	**29.81**

Table 33: - Soil textural classes laboratory result

s/n	Diversion headwork	% clay	% sand	% silt	Class
1	Rasa	25.11	43.96	30.93	Sandy loam
2	Wodesa	44	30	26	Clay loam

(Source: Feasibility design documents)

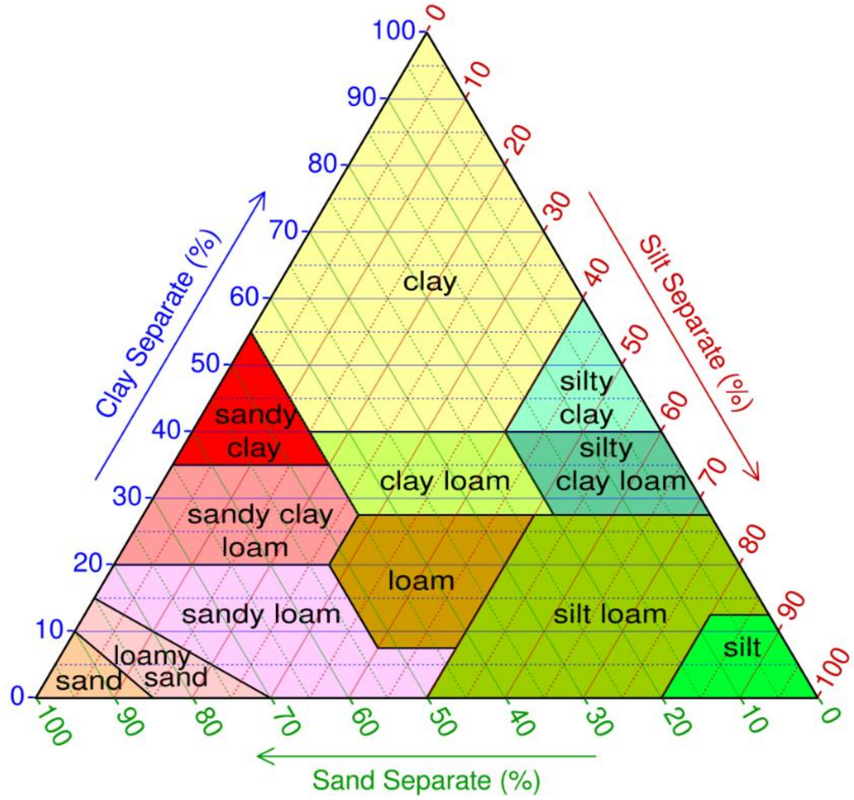


Figure 11: - A soil texture diagram redrawn from the USDA
Sources:webpag http://nrcs.usda.gov/Internet/FSE_MEDIA/nrcs142p2_050242.jpg

Table 34: - Typical Values of Unit Weight for Soils

Soil type	γsat (kN/m3)	γd(kN/m3)
Gravel	20 –22	15 – 17
Sand	18 – 20	13 – 16
Silt	18 – 20	14 – 18
Clay	16 – 22	14 – 21

Sources: (Arora, 1996)

Table 35: - The allowable bearing capacity of different soil types

Soil Type	Allowable Bearing Capacity (kN/m^2)
Soft Clay	50 - 100
Firm Clay	100 - 200
Stiff Clay	200 - 300
Loose Sand	50 - 150
Medium Dense Sand	150 - 250
Dense Sand	250 - 450
Gravel and Sandy Gravel	300 - 600
Hard Clay	300 - 600
Silt (Loose)	50 - 100
Silt (Medium Dense)	100 - 150
Silt (Dense)	150 - 250
Rock (Soft Weathered)	500 - 1000
Rock (Hard Bedrock)	1000 and above

Sources: (Braja, 2007)

Figure 12:-Photos of damaged structures of Rasa diversion headwork (Source: Author,2024)