DAM BREACH ANALYSIS AND DOWNSTREAM FLOOD MAPPING USING HEC-RAS AND HEC-FIA (A CASE STUDY OF GERHU-SIRNAY DAM)

MSc THESIS

KEHASE NEWAY GEBRETSADKAN

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

JANUARY, 2020

DAM BREACH ANALYSIS AND DOWNSTREAM FLOOD MAPPING USING HEC-RAS AND HEC-FIA (A CASE STUDY OF GERHU-SIRNAY DAM)

KEHASE NEWAY GEBRETSADKAN

A THESIS SUBMITTED TO THE

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

MASTER OF SCIENCE IN HYDRAULIC ENGINEERING

MAIN ADVISOR: - Dr. MOLTOT ZEWDIE

CO- ADVISOR: - Mr. NEBIYAT FEKEDE

JANUARY, 2020

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

ADVISORS APPROVAL SHEET

(Submission Sheet-1)

This is to certify that the thesis entitled **"Dam Breach Analysis and Downstream Flood Mapping Using HEC-RAS and HEC-FIA (a Case Study of Gerhu-Sirnay Dam)"** submitted in partial fulfillment of the requirements for the degree of **Masters of Science** with specialization in **Hydraulic Engineering**, the graduate program of the **Department of Hydraulic and Water Resources Engineering**, and has been carried out by **KEHASE NEWAY GEBRETSADKAN** Id. No **PGHY/023/09**, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Dr. Moltot Zewdie	______________	____________
Name of major advisor	Signature	Date
Mr. Nebiyat Fekede	______________	____________
Name of minor advisor	Signature	Date

HAWASSA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

EXAMINERS' APPROVAL SHEET

(**Submission Sheet-2**)

We, the undersigned, members of the board of Examiners of the final open defense by **Kehase Neway Gebretsadkan** have read and evaluated his thesis entitled with "**Dam Breach Analysis and Downstream Flood Mapping Using HEC-RAS and HEC-FIA (a Case study of Gerhu-Sirnay Dam)**", and examined the candidate. This is therefore, to certify that the thesis has been accepted in the partial fulfillment of the requirements of the degree.

Dr. Moltot Zewdie	______________	_____________
Name of Major Advisor	Signature	Date
Dr. Alemu Osore	_______________	_____________
Name of Internal Examiner-I	Signature	Date
Mr. Lemma Tufa	______________	_____________
Name of Internal Examiner-II	Signature	Date
Dr. Mekete Dessie	_______________	_____________
Name of External Examiner	Signature	Date
__________________	______________	_____________
SGS Approval	Signature	Date

Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the School of Graduate studies (SGS) through the Department/School Graduate Committee (DGC/SGC) of the candidate's department

Stamp of SGS Date: ___________________

Remark

- Use this form to submit the thesis **with or without minor correction** suggested by the examining board.
- 3 copies

DECLARATION

I the undersigned person declare that, this MSc thesis is my original effort and has not been presented in any other university, and all sources of materials used for this study have been properly acknowledged:

Name: **Kehase Neway Gebretsadkan**

Email Address: **kehaseneway12@gmail.com**

Signature: ________________

Place: **Hawassa University**

Date of submission: ________________

ACKNOWLEDGEMENT

First and foremost, I would like to thank the Almighty **God** and his mother **Saint Marry** for giving me a chance to reach this point of success. God! What you have done for me is really beyond what I was imagined and dreamt.

My truthful thankfulness goes to my major advisor **Dr. Moltot Zewdie** and Co-advisor **Mr.Nebiyat Fekede** for their critical advice, innovative ideas, encouragements and guidance throughout this research work. I would like to express my sincere gratitude to all Hawassa University, Faculty of Bio-System and Water Resource Engineering staffs who gave me the postgraduate courses.

I would like to forward my depth full admiration and thanks for my reliable friend **Mr. Haile Belay** who is a lecturer in Dilla University department of Hydraulic and Water Resource Engineering. For his guidance and help starting from the model selection up to the completion of this study, he was my back bone.

I forward special thanks to all my families for their prayer and moral support and my wife **Mrs. Teamet welday** for her guidance and support on the document's idea consistency, language edition, feedback, financial fund and over all motivations to complete this work on time.

I also acknowledge, Hydrology Department, GIS Department of the Ministry of Water, Irrigation and Electricity that provided me data and information needed for this work. I would like to thank Tigray Water Resource Bureau staff members **Mrs. Tirusew Dawed** and **Mr. Assefa Beyene** for their motivation, appreciation and support while collecting secondary data and **Mr. Gebretsadkan Girmay** who is staff member of Ahferom Wereda Water Resource Bureau for his help while collecting field and secondary data.

LIST OF ABBREVIATIONS AND ACRONYMS

1D/2D	One/ Two dimensional
a.m.s.l	Above Mean Sea Level
AMC	Antecedent Moisture Content
ANCOLD	Australian National committee On Large Dams
Arc SWAT	Aeronautical Reconnaissance Coverage Soil Water Assessment Tool
Arc-GIS	Aeronautical Reconnaissance Coverage Geographic Information System
ASCE	American Society of Civil Engineers
BEED	Breach Erosion of Earth Dam
DAMBRK	Dam Break-Forecasting Model
DDC	Depth Damage Curve
DEFRA	Department for the Environment, Food and Rural Affairs
DEM	Digital Elevation Model
DTM	Digital Terrain Model
EAP	Emergency Action Plan
EASSSA	Ethiopian Agricultural Sample Survey Statistical Agency
EMA	Ethiopian meteorological agency
ERA	Ethiopian Roads Authority
FAO	Food and Agricultural Organization
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FLDWAV	Flood Wave Dynamic Model
FLOW SIM	Flow Simulation Model
GPS	Global Positioning System
HEC-FDA	Hydrologic Engineering Center Flood Damage Analysis
HEC-FIA	Hydrologic Engineering Center Flood Impact Analysis
HEC-Geo RAS	Hydrologic Engineering Center Geographical River Analysis System
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HIS-SSM	Flood Information System Damages and Causalities Module
HSG	Hydrologic Soil Group

IFDRAMM	Integrated Flood Damage Risk Analysis Management and
LIFE-Sim	Life Simulation
MDSF	Modelling and Decision Support Framework Methodologies
MoWIE	Ministry of Water, Irrigation and Energy
NWS	National Weather Service
OGL	Original Ground Level
PAR	Population At Risk
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SA/2D	Storage Area/Two Dimensional Area Connector Tool
SCS	Soil Conservation Service
SMPDBK	Simplified Dam Break Model
SRH	Sedimentation and River Hydraulics
TWRB	Tigray Water Resource Bureau
TWWCE	Tigray Water Works Construction Enterprise
USACE	US Army Corps of Engineers
USDA	U.S. Department of Agriculture
WinDAM	Windows Dam Analysis Modules
WMO	World Meteorological Organization

TABLE OF CONTENTS

LIST OF APPENDIXES

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Dam breach analysis is a science that quantifies the hazard sourced from dam failures. On downstream of Gerhu-sirnay dam there are developments including water treatment plant and multi owner seasonal agricultural areas. The objective of this study was to estimate breach outflow hydrograph to the downstream reach for overtopping and piping scenarios henceforward to assess flood damage for the worst scenario. The quality of rainfall data was tested for outlier, adequacy, homogeneity using Pettitt (1979) test and consistency tests. The probable maximum precipitation calculated by Hershfield (1965) method was 129.7 mm. The inflow hydrograph was obtained by using soil conservation service (SCS) method and its probable maximum flood (PMF) reaches 329.1 m^3/s. The breach parameters for both scenarios were early calculated by the four regression equations integrated with Hydrologic Engineering Center River Analysis System (HEC-RAS) breach calculator tab. Due to comparison in both scenarios, the final breach parameters were taken from Von Thun and Gillette (1990). For overtopping the resulting parameters were 54 m breach bottom width, 85.75 m breach top width, 0.5 side slope (H: V) and 0.73 hrs breach formation time whereas in case of piping these parameters were 32 m, 57.75 m, 0.5 and 0.53 hrs respectively. Then HEC-RAS version 5.0.3 has used to model the dam breach analysis with two dimensional unsteady flow conditions of inflow hydrograph as upstream boundary condition and normal depth as downstream boundary condition. The peak breach outflow for both scenarios were 2,239.7 and 1,282.68 m^3/s respectively as overtopping was the worst scenario. From the prepared inundation maps by GIS for about 521.1 ha area was inundated under the worst scenario with a maximum depth 24.07 m and duration of 1.0 hrs. By using Hydrologic Engineering Center Flood Impact Analysis (HEC-FIA) version 2.2, the flood damage was estimated in terms of direct economic damage and life loss. The total ex-ant direct economic damage result shows 63,650,493.3 Birr. From the report of LIFE-Sim dynamic model integrated with HEC-FIA, about four persons could be lost their life. To minimize the quantified damage, well operation of emergency action plan would be an important tool. The water treatment plant of Gerhu-sirnay town must be displaced from its current place to at least 0.073 km apart from both sides of the river banks until to do this the stakeholders of the plant must be under the age of 65 years to have an efficient warning mobilization.

Key words: *Gerhu-sirnay Dam, Failure Scenarios, HEC-RAS, HEC-FIA and LIFE-Sim*

1. INTRODUCTION

1.1. Background

Dam breach analysis is a science which notifies and quantifies the hazard sourced form dam failures. This type of analysis has been first started after the failure of Puentes dam in 1802, Spain. This dam failure has damaged over 1800 houses and 40,000 trees, up to now many other dams have failed and caused different damages including life loss (US Bureau of Reclamation, 2007). Then currently it becomes the hot area of research all over the world.

In Ethiopia, Dam breach analysis study has very short history comparatively with the other countries of the world but it is very important to minimize property damage and life loss. Therefore starting from 2015 the analysis regarding to this science was running vastly (Jiregna, 2016).

Because of their potential failure and cause of catastrophic flooding, dams may become source of risks to downstream property and life (Zagonjolli, 2007). The HEC-RAS training document for dam breaks lists that the probable dam failure scenarios were overtopping, piping, foundation defects, land slide, overturning, cracking and equipment malfunctions. According to the reviewed literatures, dam breach analysis was conducted to determine the ultimate discharge from a hypothetical breach of a dam. The outcome was a breach hydrograph from dam failure with a flood wave immediately downstream of the dam and mapping of inundation areas used for estimating the potential consequences of a dam breach. In most of the literatures, Dam breach analysis was modeled by HEC-RAS 1D and 2D steady or unsteady flow analysis systems.

In this study, overtopping and piping failure scenarios were analyzed due to their repetitive occurrence (FEMA, 2013) and (Arakelyan, 2018). To succeed this, the general techniques were hydrologic analysis to prepare stream flow, hydraulic modeling to estimate breach out flow and damage modeling to quantify the flood damage throughout the flood area. Therefore the breach analysis modeling was accomplished by HEC-RAS version 5.0.3 2D unsteady flow analysis and its downstream damage assessment in terms of ex-ant direct economic damage and life loss were performed by HEC-FIA version 2.2.

The reviewed dam breach analysis studies in Ethiopia were performed with HEC-RAS 1D steady or unsteady, and 2D steady or unsteady river flow analysis systems to estimate breach outflow and the flood impact estimation also with excel sheet. The additional feature in this study was in case of flood damage assessment, instead of using excel sheet HEC-FIA model was appropriate due to its capability of performing both static and dynamic model simulations. In case of crop damage assessment HEC-FIA considers crop sowing cost, harvest cost and other crop treatment costs starting from date of sowing up to harvesting to be subtracted from the crop production value.

1.2. Statement of the Problem

Dam breach was the most destructive hazard sourced from failure of dams constructed for the purpose of facilitating human and wild lives. For example; the failure of South fork dam caused 2,209 fatalities, Machchu-2 Dam 5,000 fatalities and Banqiao Dam 171,000 fatalities (US Bureau of Reclamation, 2007). Due to lack of enough damage recovery budget and facilities, occurrence of such hazard in developing countries like Ethiopia has many other damages in addition to their direct fatalities. Therefore in order to prevent and minimize this worst hazard an ex-ant dam breach analysis and downstream flood mapping is an option less way in delineating the area of exposure.

In Ethiopia the science of dam breach analysis and flood inundation mapping has not well practiced. Therefore, More than 85% of the constructed dams haven't flood inundation map due to dam breach (Jiregna, 2016). As a result of this Gerhu-sirnay dam have not dam breach analysis and flood inundation mapping study. So this problem provides an initiation to carry out this study on Gerhu-sirnay dam to minimize the problems faced from being not having flood inundation maps.

According to the site supervision report by the client organization which was TWRB (2019) and Actual site visit, for about 2 l/s internal seepage was appeared in the downstream face of Gerhu-sirnay dam. As detailed in the report this failure scenario was occurred due to the presence of permeable material in the core zone and improper work of riprap material, therefore the dam has the probability to fail.

Downstream of the dam, there are developments including water treatment plant of Gerhu-sirnay town (critical infrastructure) with its stakeholders, multi owner field crop agricultural areas, vegetation cover and natural ecosystems ((TWRB, 2019) and Actual site visit).

If flood inundation mapping and hazard assessment was not carried out, the dam failure may cause destructive hazard over the available exposures. Therefore in order to minimize the flood damage on the above listed elements: this study is the most important endeavor.

1.3. Objective of the Study

1.3.1. General Objective

The general objective of this research is to estimate breach outflow hydrograph to the downstream reach.

1.3.2. Specific Objectives

1. To estimate dam breach parameters.
2. Preparation of downstream flood inundation mapping.
3. To assess downstream ex-ant direct flood damage.
4. To develop emergency action plan.

1.4. Research Questions

This study has been initiated to answer the following questions:

- How could be dam breach parameters estimate?
- How much area would be inundated due to Gerhu sirnay dam breach?
- What type of downstream flood damage would appear due to breach out flow?
- What is the advantage of Emergency Action plan for Gerhu sirnay dam breach?

1.5. Scope of the Study

The physical size of this study was limited in between upstream and downstream boundary conditions longitudinally. From the inundation polygon result, it covers 11.04 Km longitudinally from the dam axis and 0.472 km laterally to the maximum inundation width. The breach failure scenarios included to investigate were both overtopping and piping. With this range, the analysis focuses to predict the outflow hydrograph by HEC-RAS model and to estimate or asses flood damage in terms of ex-ant direct economic damage and potential for life loss by HEC-FIA model.

1.6. Significance of the Study

The results of Gerhu-sirnay dam breach analysis and downstream flood mapping study will be used for the following points:

- The beneficiaries of this study's result were: the client of the dam from being responsible taker of flood damage and other responsible organizations for emergency recovery.
- To develop strategies and guidelines for public flood defense as a function of Gerhu-sirnay dam failure. This will be achieved after quantifying the damage of the failure on human life, different infrastructures and agricultural practices.
- Its findings would be an input for safe installation and suitable site selection of future infrastructures regarding to this dam. Because after the flood prone areas will be delineated, any additional development around this dam will be installed out of the flood exposure areas to minimize the flood damage.
- The Modelling approach will be used to initiate and support future researchers on similar scenarios.

1.7. Research Structure

It shows the total structure of the study: the first chapter discusses about the background of the title, statement of the problem, overall objectives in case of Gerhu-sirnay dam breach analysis, significance and its scope. The second chapter discusses and assesses the previously worked studies, which will help in understanding the current technology and modelling of dam breach analysis and downstream flood mapping. According to the conclusion and recommendations of different researchers the appropriate tools will be identified. Then, the third chapter contents were all about the materials used and methodology of this study work. The fourth chapter discuss about the overall results and discussions with some reviewed literatures of this study. Finally, the fifth chapter summarises the overall research work in terms of conclusions and recommendations.

2. LITERATURE REVIEW

2.1. General

Flood sourced from the failures of constructed dams have produced the most devastating disasters of the last two centuries (US Bureau of Reclamation, 2007). In order to reduce the potential damages from dam breach due to flooding, several hydraulic modeling programs have been developed through different researchers. In the Early times, the actual failure mechanics of dam failure have not been well understood for either earthen or concrete dams (Yonatan, 2016). According to that time concepts to predict downstream flooding due to dam failures, it was assumed that the dam was failed completely and instantaneously. But according to the present sciences, a dam breach was the partial or complete collapse of the dam which leads to rapid wave agent flow of water (Fread, 1988).

Dam breach can be occurred from many causes and scenarios but the most common ones were: hydrologic, geologic, structural, seismic, and human influenced (FEMA, 2013). The potential catastrophic failure and the resultant downstream flood damage was a scenario that needs great attention. Therefore the impact mitigation measures need flood modeling so as to develop pre-hazard flood event detail information (FEMA, 2013). So the selection of an appropriate and latest model to correctly simulate dam breach flood routing was an essential step, therefore it would be done properly in the following literature sections.

2.2. Dam Breach History

Dams were structures installed across the waterway that obstructs, directs or slows down, often creating a reservoir, lake or impoundment (FEMA, 2013). These structures contain a lot of dangerous forces and if they were not installed properly the dam may fail and causes massive destruction all over the environment which will be inundated. From previous history hundreds of dams have been failed and every year many dikes have breached due to high flow in the river, sea storm surges, etc. often causes catastrophic consequence (Zagonjolli, 2007).

As evidence, some of the dams failed across the world with their remarkable fatalities and impacts were listed in table 2-1 below:

Table 2-1: List of major failed dams

Name	**Year**	**Location**	**Fatality**	**Detail**
Puentes dam	1802	Lorca, Spain	608	1,800 houses and 40,000 tries destroyed
South fork dam	1889	Johns town united states	2,209	Blamed locally on poor maintenance by owners; courts deemed it an ‘act of God’ following heavy rainfall
Tigra Dam	1917	Gwalior, India	1,000	Failed due to water infiltrating through foundation and caused more fatalities
St. Francis Dam	1935	Santa Clarita, United States	600	Geological instability of canyon wall that could not have been detected with available technology of the time
Mohne Dam	1943	Ruhr, Germany	1,579	Destroyed by bombing during operation Chastise in world war 2
Kurenivka Mudslide	1961	Kiev, Ukraine	1,500	Caused by heavy rains or overtopping scenario
Panshet Dam	1961	Pune, India	1,000	Dam burst due to pressure of accumulated rain water
Teton	1975	Cities of Sugar and Rexburg	14	Piping; Over $ 1 billion estimated property damage has been occurred
Sempor Dam	1967	Java, republic of Indonesia	2,000	Failed due to flash flood during construction
Banqiao Dam	1975	Zhumadian, China	171,000	Extreme rainfall, 11 million people lost their homes. This records the Worst dam failure in the world’s history
Machchu-2	1979	Morbi, India	5,000	Heavy rain and flooding

Source: US Bureau of Reclamation (2007)

2.3. Dam Breach Causes and Scenarios

Based on FEMA (2013) a lot of dams were subjected to different dam failure causes but the most common ones were summarized using the five types of failure causes. These were: hydrologic, geologic, structural, seismic, and human influenced. The details of these basic terms have been described in table 2-2 below:

Table 2-2: Causes of dam failure

Failure causes	Failure scenario details
Hydrologic	Overtopping due to: • Inadequate spillway capacity design and blocked spillway • Loss of freeboard due to embankment settlement or erosion • Structural overstressing of dam components
	Surface erosion due to: • High velocity water and wave action
Geologic	Piping and internal erosion caused by: • Internal cracking, hydraulic fracture, or differential settlement • Inadequate filters and outlet pipe line failure • Pipes through the embankment formed by roots or animal/insect burrows
	Slope instability and hydraulic fracturing • Load exceeds sliding resistance at base or at joint of structure
Structural	Concrete dam: failure of critical structural components Embankment dam: failure of upstream or downstream face
Seismic	Earthquake/ ground movement for liquefiable foundation of embankment materials
Human Influenced	Miss-operation • Sudden rise in reservoir level causes flow through transverse cracks in embankment. • Incidents of gate failure, terrorist activities/political issue etc…

Source: FEMA (2013)

Due to the type of dam's construction material and its site specific conditions, dams may be subjected to one of the above failure scenarios. Therefore, the breach parameters, breach outflow discharge and timing of a dam failure varies according to the type of scenarios analyzed. Embankment dams have not subjected to sudden failure rather tends to breach to where the embankment material can't resist erosion.

The most common scenarios for dam failure between January 1975 and January 2011 G.C. were summarized in table 2-3 below:

Table 2-3: Dam failure Scenarios

Failure scenarios	Number of dams failure	Percentage
Flooding or overtopping	465	70.9%
Piping or seepage	94	14.3%
Structural	12	1.8%
Human related	4	0.6%
Animal activities	7	1.1%
Spillway	11	1.7%
Erosion/slide/instability	13	2.0%
Unknown	32	4.9%
Other	18	2.7%
Total number of dam failures		656

Source: FEMA (2013)

According to Arakelyan (2018) study for embankment dam's failure scenario, about 35% of them were due to overtopping, 38% due to piping, 21% also from foundation defects and 6% were from other unknown failures.

According to the above studies, overtopping and piping were the most common scenarios of dam failure. Therefore these common failure scenarios were discussed in below sections to investigate in this study:

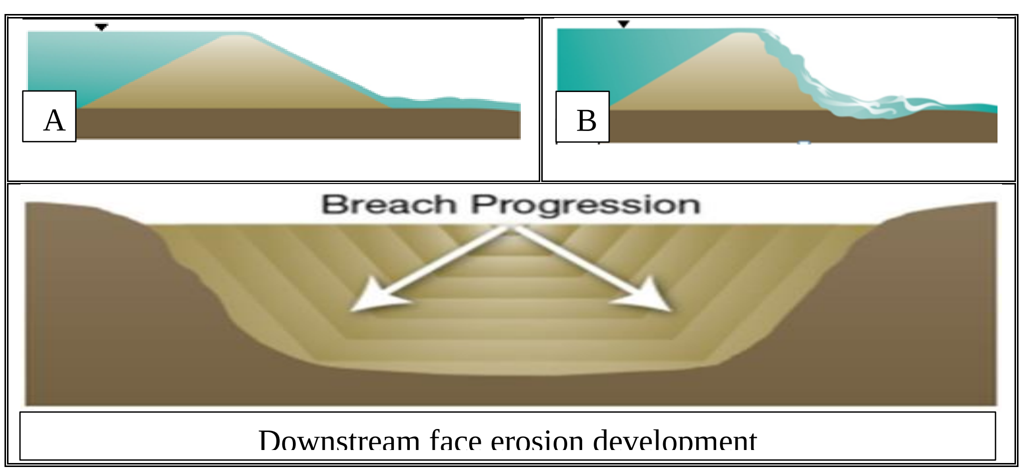
2.3.1. Overtopping Scenario

According to engineering assessment of Ka Loko dam failure by Wehrheim (2006), the Ka Loko Dam in the town of Kilauea was failed due to overtopping scenario under the extreme 40 days rain fall event results in 66.3 inches rainfall in March 14, 2006. This earthen dam was failed 116 years after the completion of the dam's construction with a reservoir capacity of 409 million gallons. The dam had a maximum height of 40 feet and its crest length and width was 770 and 15 feet respectively. Finally the assessment reported that seven life losses including a pregnant woman were recorded.

As Liggett (2009/2010), the Walnut Grove Dam located in Hassayampa River was failed due to overtopping on February 21, 1890 after two years of service time. As the report shows the failure was happened because of the original design capacity of the spillway was reduced during construction to save money. So at the instant of maximum breach outflow the breach scenario possesses 1.09 m overtopping depth.

The Delhi dam in Lake Delhi was failed due to overtopping scenario on July 24, 2010 after 90 years operation time (Daniel, 2010). The spillway capacity of the dam was designed for 708.5 m^3/s peak flow but the failure was happened after the lake receives 10 inches of PMP for about 48 hours raining duration. So this rain situation has caused 1,955.4 m^3/s peak breach outflow that was more than the designed capacity which was caused 4.6 m overtopping depth.

The general visualization of overtopping failure progress was placed below: A represents failure initiation time and B for the development of downstream head cut).



Source: FEMA (2013)

Figure 2-1: Downstream face breach head cutting progress

2.3.2. Piping Scenario

According to the report from FEMA (2013), the catastrophic piping failure of Teton dam in southern Idaho, US led to flooding in the cities of both Sugar and Rexburg. This hazard has caused 14 life losses and over 1 billion USD estimated property damages.

The failure scenario was happened due to the presence of permeable soil material with in core and cracks in the foundation bedrock that leads to seep water through the dam's downstream face. These problems lead to develop internal erosion known as piping scenario and finally it caused the total collapse of Teton dam. The red circles on the figure below shows that, the piping hole was sized laterally over time due to internal hydraulic pressure on Teton dam from figure A to C.

Source: US Bureau of Reclamation (2007)

Figure 2-2: Sequential image of Teton dam piping scenario

2.4. Dam Breach Analysis Techniques

The Colorado State Guidelines for Dam Breach (2010) proposed four well-known and documented dam breach analysis techniques. These are: breach parameter estimation, breach outflow discharge estimation, breach outflow flood routing and inundation mapping. In this study assessment of downstream flood damage using HEC-FIA would be the additional procedure. All those procedures for complete dam breach analysis were briefly discussed as below:

2.4.1. Breach Parameter Estimation

According to FEMA (2013) the shape of breached embankment dams was assumed to be trapezoidal. The most common breach parameters to describe the above assumption were: breach initiation time, breach formation time, breach height (hb), average breach width (B_{avg})

and side slope (H: V) (Wahl, 1998). For further discussion first it needs to define and visualize each parameter. According to FEMA (2013) their definition was as follows:

A. **Breach initiation time (Critical breach development time):** It is the point of time starts with the first sign of dam failure with respect to failure scenario. The duration up to the instant of the first breaching of upstream face of the embankment material is considered as breach initiation phase.

   During overtopping scenario it starts from the instant of overflowing water over the embankment and in piping case from the instant of seepage of muddy water through the embankment. When the flow may controlled it is possible to prevent the dam breach unless the end of this phase determines warning formation time mainly used to release warning information to minimize the life loss of the population at risk.

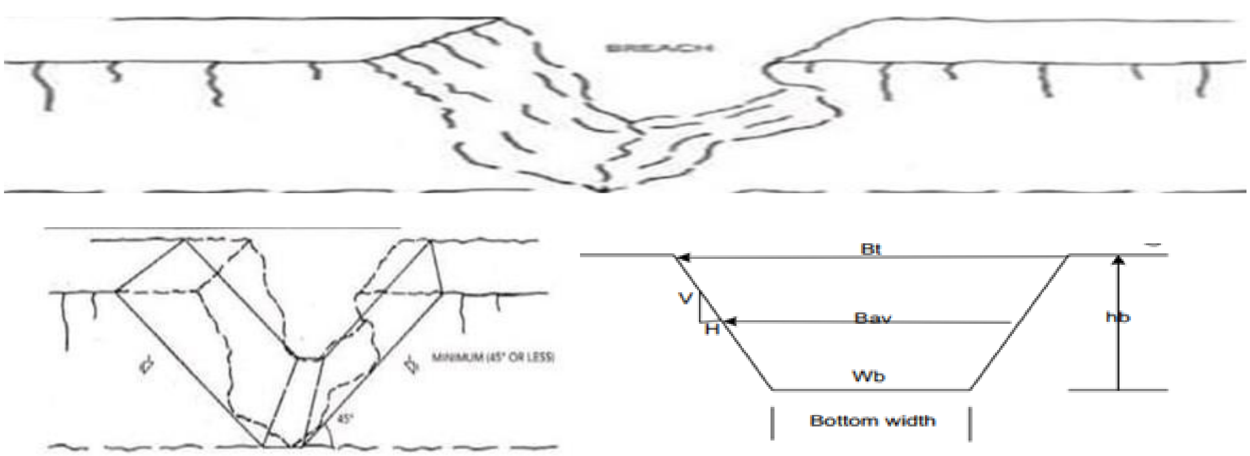
B. **Breach formation time (t_f):** The phase starting from the end of breach initiation phase up to full development of breach dimensions is called breach formation time (t_f).

C. **Breach height (h_b):** This is the vertical extent of the breach, measured from the dam crest down to the invert of the breach.

D. **Average breach width (B_{avg}):** The final breach width, typically measured at the vertical center of the breach height.

E. **Side slope (H: V):** The breach side slope factor along with the breach width and depth fully specifies the shape of the breach opening.

The sequential ideal profile of breached dam and development of its breach parameters was provided in figure 2-3 below:



Source: Wahl (1998)

Figure 2-3: Ideal development of dam breach parameters

Where:

H_b=height of dam

W_{eb}=bottom breach width

B_{ut}=top breach width

B_{av}=average breach width

According to Colorado State Guidelines for Dam Breach (2010), the analysis methods to determine dam breach parameters were: comparative, regression and physical methods as discussed below:

2.4.1.1. Comparative Methods

Under this analysis method, the given failed dam compares with the dams failed and well documented before. Then when the dam under consideration was close similar in size, construction materials and other concerns with the well documented ones, the breach parameters and peak outflow discharge may directly applied to the dam being analyzed (Colorado State Guidelines for Dam Breach, 2010).

2.4.1.2. Regression Methods

This method trusts on statistical analysis of data obtained from the documented dam failures. These regression equations recorded below were used to predict time of failure and the required breach parameters. These were: MacDonald & Langridge -Monopolis (1984), Von Thun & Gillette (1990), Froehlich (1995a, 2008) (Colorado State Guidelines for Dam Breach, 2010). Since those equations were integrated with HEC-RAS breach calculator tool then this method become accurate and widely used by many researchers (Adnan, 2017).

2.4.1.3. Physical Based Model

Many researchers have developed physically based and numerically measured dam breach models. This model predicts breach parameters by using an erosion model of hydraulic principles, soil mechanics and sediment transport (Wahl, 1998).

According to Wahl (1998), the major models were described in a detail way in table 2-4:

Table 2-4: Physical based embankment dam breach models

Model	Sediment transport	Breach morphology	Parameters	Other features
Cristophano (1965)	Empirical formula	Constant breach width	Angle of response, others	-
Harris and Wagner (1967);BRDAM (Brown and Rogers, 1977)	Schoklitsch formula	Parabolic breach shape	Breach dimensions, sediments	-
DAMBRK (Fread, 1977)	Linear pre-determined erosion	Rectangular ,triangular or trapezoidal	Breach dimensions, others	Tail water effects
Lou (1981); Ponce and Tsivoglou (1981)	Meyer-Peter and Muller formula	Regime type relation	Circular shear stress, sediment	Tail water effects
BREACH (Fread, 1988)	Meyer-Peter And Muller Modified By Smart	Rectangular ,triangular or trapezoidal	Critical shear, sediment	Tail water effects, dry slope stability
BEED (Singh and Scarlatos, 1985)	Einstein- Brown formula	Rectangular or trapezoidal	Sediments, others	Tail water effects, saturated slope stability
FLOW SIM 1 and FLOW SIM 2 (Bodine, undated)	Linear pre-determined erosion; Schoklitsch formula option	Rectangular ,triangular or trapezoidal	Breach dimensions, sediments	-

Source: Wahl (1998)

As a conclusion from table 2-4 above, most of these models depends on bed load type erosion formula that suggest the assumptions of gradually varied flow and relatively large flow depth in comparison to the size of roughness elements. These formulations may be appropriate for some stages of breach process, but were not consistent with the mechanism of much of the breaching process. Therefore regression method was more consistent, supported by HEC-RAS 2D, and many researchers (Adnan, 2017). Therefore the breach parameters of this study will be estimated by regression method.

- **Comparison of Regression Equations**

Adnan (2017) recommended that, after the breach parameters were calculated with those equations included in regression method under HEC-RAS breach calculator tool, there was a need to standardize or validate each formula's result by comparing with the range based reference guidelines developed by different federal agencies. According the author's recommendations if more than one equation's breach dimension results were within the range, the equation comparison option would tend to use envelope curve which develops only from fourteen dam failure events.

For small dams classification it was better to use the federal guidelines possible breach parameter ranges to select the best regression equation of breach parameter estimator (USACE, 2016). Envelope curve was developed only for about fourteen failed dam data sets, due to limited data sets it may not estimate accurate upper bound of peak flow hence resulted with error flood damage estimation. The guide lines were dependent on dam height and side slope to calculate average breach width (B_{avg}) and time to failure (T_f). Therefore it provides better result.

The guidelines developed from the governmental organizations were recorded table 2-5 below.

Table 2-5: Possible Ranges of Breach parameters

Dam type	Average Breach Width, B_{avg}	Side Slope (H) H:1V	Failure Time T_f (hrs.)	Owner
Earth fill/ Rock fill	(0.5 to 3.0) x h_d	0 to 1.0	0.5 to 4.0	USACE (1980)
	(0.5 to 5.0) x h_d	0 to 1.0	0.1 to 4.0	USACE (2007)
	(1.0 to 5.0) x h_d	0 to 1.0	0.1 to 1.0	FERC (1988)
	(2.0 to 5.0) x h_d	0 to 1.0 (slightly larger)	0.1 to 1.0	NWS (2006)

Source: USACE (2016)

Where: hd =height of dam

2.4.2. Breach Outflow Estimation

2.4.2.1. Hydrologic Method

As the report from Colorado State Guidelines for Dam Breach (2010), the results of several case studies indicates that this method (HEC-HMS, HEC-1…) was suitable only for most overtopping failure simulations, but scenarios like piping simulation were limited. Therefore, the results may not be valid for a piping failure of smaller reservoirs when piping would occur.

Hydrologic models were the most common tools to generate breach hydrograph but the main problem with these methods were their inability of routing out flow flood hydrograph to downstream of the dam (FEMA, 2013). But the main issue of dam breach analysis was downstream flood routing.

2.4.2.2. Hydraulic Method

As the US Bureau of Reclamation (1988) study on simplified inundation maps for emergency action plan, the hydraulic models have good performance with the capability of simulating breach growth as time dependent linear process and compute breach outflows using principles of hydraulics.

According to Colorado State Guidelines for Dam Breach (2010), the latest versions of HEC-RAS 2D model includes algorithms to model both overtopping and piping breach scenarios. HEC-RAS uses hydraulic principles through reservoir polygon upstream and downstream of the dam to define how the reservoir drains during formation of dam breach. The dam crest was modeled as an inline weir and either piping failure or overtopping failure was simulated with enlargement of breach occurring over time as defined by a specified breach progression. Flow through the piping hole was calculated as orifice flow and flow through the overtopping was calculated as weir flow. HEC-RAS can also model a piping scenario as that does not progress to its maximum point of collapsing up to the dam crest. In this scenario, the piping hole was simulated as a sluice gate.

In case of Piping and weir flow scenarios, the hydraulic (HEC-RAS 2D) model needs different flow coefficients (USACE, 2016) according to the type of the dam:

Table 2-6: Weir and piping flow coefficients

Type of dam	Piping flow coefficients	Weir coefficients
Earthen clay or Clay core	0.5-0.6	2.6-3.3
Earthen sand and Gravel	0.5-0.6	2.6-3.0
Concrete Arch	0.5-0.6	3.1-3.3
Concrete Gravity	0.5-0.6	2.6-3.0

Source: USACE (2016)

- **Comparison of Outflow Estimation Methods**

According to the ideas found from the above papers, comparison of both hydrologic and hydraulic methods was provided as below:

- Breach outflow estimation using hydraulic principles was usually more accurate than hydrologic model because the modeler can more accurately simulate the shape of the reservoir, tail water effects, and drawdown effects.
- Unlike hydrologic model, hydraulic model performs piping head used in orifice flow was measured from the center of the piping hole at all times regardless of its location and size.
- The hydraulic or HEC-RAS model has better accuracy but needs detailed input data over the hydrologic model. But professionally accuracy should come first and needs much more effort.

- Hydrologic models can’t perform downstream flood routing but hydraulic models have the capability of performing flood routing with different channel routing options.

From the above comparison points 2D hydraulic model shows a better performance. Therefore in this study, HEC-RAS 2D version 5.0.3 hydraulic model was selected to estimate the peak breach outflow discharge with all its ordinates throughout the breach simulation time and route it downstream of the reach.

2.4.3. Outflow Flood Routing

Breach outflow flood routing refers to the determination of outflow hydrograph magnitude with respect to time and river stage by considering the flood area as 1D, 1D-2D and 2D only (USACE, 2016). HEC-RAS 2D version 5.0.3 has an option to use either full saint Venant equation or Diffusion wave equation as full unsteady flow category and level pool routing in two dimensions.

In this study HEC-RAS version 5.0.3 2D was used to model dam breach analysis so for its latest and accurate analysis it supports 2D flood plain routing with full unsteady flow and level pool routing (USACE, 2016). As per the manual, for flood plains with no additional hydraulic structures like bridge, diffusion wave equation type of reservoir routing provides faster and accurate result. This method of routing recommends for a steeper streams and flood plains with no additional hydraulic structure (USACE, 2016). So the case study of this reaserch has satisfied both steeper streams and flood plain without additional structures.

Therefore the routing considers 2D flood area with diffusion wave equation to estimate every hydraulic parameter at a critical location throughout the grid will be applied.

In addition to all points discussed above, the model needs boundary conditions to have complete hydraulic flow analysis (USACE, 2016). The possible boundary conditions of the model were illustrated and discussed below:

2.4.3.1. Boundary Conditions

Boundary conditions were necessary to establish the analysis areal limit of the river system. In subcritical and supercritical flow regimes separately, boundary conditions were only necessary at the downstream ends of the river stream.

Whereas if a mixed flow regime calculation was going to be made, then the boundary conditions must be entered at all ends of the river system (USACE, 2016). This case was applied in this study to generate mixed flow analysis from Gerhu-sirnay dam breach under the specified failure scenario.

A. Upstream Boundary Condition

Upstream boundary conditions were required at the upstream end of all reaches that were directly joins the reservoir. The version of HEC-RAS 2D model used in this study has the capability of performing unsteady flow analysis with many upstream boundary condition types including inflow hydrograph, lateral inflow hydrograph, ground water interflow and others mention under the unsteady flow data tab of HEC-RAS 2D version 5.0.3 (USACE, 2016). In this study the upstream boundary condition type used was flow hydrograph of discharge versus time that would be obtained from the hydrologic processes.

B. Downstream Boundary Condition

Downstream boundary conditions are required at the downstream end of all reaches that are not connected to other reaches (USACE, 2016). From several downstream boundary condition types, normal depth from manning's equation or which was the average slope of the larger stream along the 2D flow area was used. As per the manual, this type of downstream boundary condition was easy to feed the model and used in this research.

C. 2D Flow Area Initial Conditions

Before proceeding to the unsteady flow analysis simulation, it needs to set the initial conditions to the whole 2D area system. The most well-known 2D Flow Area initial conditions were (USACE, 2016):

- Completely dry
- Single water surface elevation
- Restart file from the previous run and
- Using ramp up time

Out of these, the completely dry scenario was the most common and was selected here in this study. This condition serves the model as intermediate boundary conditions.

Therefore from the review above, the possible boundary conditions of this study were upstream boundary condition as inflow hydrograph, intermediate boundary condition as dry condition of the 2D flow area and downstream boundary condition also normal depth obtained from the average slope of 2D flow area's main river.

2.4.4. Preparation of Flood Inundation Mapping

Inundation mapping refers the way of creating flood maps which shows the spatial extent of probable flooding. According to this study inundation mapping was prepared by importing the RAS Mapper results to Arch Map and processing here for mapping standards. It supports policy and decision makers to decide about how to allocate resources, warning evacuation models, flood forecasting and significant land use planning in flood prone areas (Haile, 2018).

According to FEMA (2013), Inundation maps can have different uses:

i. **Emergency response:** this is the action taken after an incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the effect on critical infrastructure and environment. This would be the response by the dam owner, local community emergency management to minimize the consequence of actual dam failure or incident.

ii. **Estimation of hydraulic parameters at critical location:** this helps to specify the hydraulic inundation map data at a given locations. Estimation of those parameters was the pre-request for flood impact analysis.

These includes: preparation of flood depth, velocity, duration, water surface elevation, inundation extent, depth of a given failure scenarios, and other data needs to quantify.

iii. **Hazard mitigation planning:** a pre-event action achieved through identifying flood exposed area and its hazard potentials it may happen. In the case of dam failure, hazard mitigation planning involves identifying the population at risk and identifying the action to reduce their exposure. Information required by hazard mitigation planners includes the knowledge of breach inundation boundary with its specific depth, velocity and arrival time of flooding.

Nebiyat (2016) conducted Flood Risk Analysis with regards to Crop Yield in Awash River Basin, Ethiopia. This author has prepared flood inundation map with the help of HEC-GeoRAS and HEC-RAS. Next the map was used to quantify flood magnitude mainly in terms of depth. Finally based on flood depth magnitude, the author has performed flood risk assessment.

From the reviewed studies above, preparation of flood inundation mapping in this research was aimed to quantify the flood characteristics including maximum inundation area, depth, velocity, water surface elevation, flood arrival time and duration at the whole 2D flow area grids. Preparation of these flood elements helps to identify the probable area exposed by flood and assessing flood damage.

2.4.5. Assessment of Downstream Flood Damage

Messner et al. (2007) said that flood economic damage assessment has two steps as ex-post and ex-ant. Ex-post assessments were carried out after the consequence of the disaster was happened. This will help to inform the overall damage amount and budget for recovery and compensation of the concerned body. Ex-ant assessments were also prior event, aimed to evaluate potential economic lose. For this damage assessment purpose there was a standard approach damage function referred as stage-damage curve or fragility curves based on the causal relationship between the intensity of hazard parameter and level of damage for each asset.

Dam breach damage estimation refers to quantifying of the potential consequence of a dam failure. This helps to build pre-event knowledge about the degree of flood damage and warns to organizations that can responsible for damage recovery (FEMA, 2013).

Dam breach flood impact analysis includes the determination of both direct and indirect economic damages and life loss. Direct economic damage stands for all direct economic disorders that comes from weather structural, agricultural or environmental damages whereas the indirect one lefts with like loss of work opportunity, income as local individuals and as a country regarding to the direct infrastructural destruction (Nafari, 2012).

From the above authors' scenario of estimating flood damage, flood damage assessment to be carried out in this study would be ex-ant or prier event type with its direct economic damage and life loss consequences. This helps to prepare well specified, respective and efficient warning evacuation, emergency action plan, societal awareness, recovery and displacement cost.

2.5. Works on Dam Breach Analysis

2.5.1. Worldwide Experience

Historically, dam breach is the most dangerous and destructive hazard that may occur due to improper design, maintenance and or back ward operation system of the structure (FEMA, 2013). Literature review of the dam breach studies have carried out to understand and analyze the latest methodologies adopted in the studies conducted by various authors on various dams all over the world. This helps in choosing the suitable methodology and numerical model to carry out dam breach analysis. Because of its impact, imagine in how much it would currently a serious issue: due to this it becomes a hot area for research and some researchers talk about its analysis as follows:

The majority of dam failure fatalities have been caused from dams having a size category of intermediate or large. The failure of four dams in the late 1800s and early 1900s, US: Mill River Dam, South Fork Dam, Walnut Grove Dam, and St. Francis Dam: all having a size category of intermediate or large caused more than 2,800 fatalities (Liggett, 2009/2010).

In case of Africa, according to the Nile Basin Capacity Building Network for River Engineering team assessment on an inventory and performances of micro dams in Sudan, Uganda and Ethiopia, a number of micro dams constructed in those countries were analyzed. As the report shows, most of them were faced severe siltation problems and some were failed due to spillway failure (over topping) (Bashar, 2005).

Based on the Australian Guideline for Flood Disaster Resilience (2017) a flood from dam breach can cause a potential damage and hence it needs flood hazard quantification to minimize the damage. The hazard quantification was dependent on the flood behaviour like depth, velocity and the combination of both. But as a general rule, flood depth and velocity magnitude of 4 m and 4 m/s was characterised by destructive nature for all types of buildings, vehicles and people.

2.5.2. Experience in Ethiopia

As Jiregna (2016) has said the Ethiopian experience of dam breach analysis study was much more insufficient only accounts 15%. But sometimes later it was investigating and studying yearly as it has been shown in the following researchers' order of study writing:

Tariku (2015) has studied dam break analysis and risk assessment in the case of Tendaho dam, Ethiopia. The analysis was based on both piping and overtopping failure scenarios. From these scenarios overtopping was the worst scenario when the available three spillways have been closed. According to the risk assessment for the worst scenario the precise and vague fatalities under the given warning were 1,012 and 2,972 respectively.

Motuma (2015) conducted One Dimensional Dam Break Analysis (The Case Study of Nashe Dam). According to the methodology of this study, model setup by using cross sections was the first step in dam breach analysis. After that the flow data, breach parameters and other such as boundary values are inserted as input data in the HEC-RAS model and the model result was exported to Arc-GIS for inundation mapping.

According to the study of Hayimanot (2015) on one dimensional dam breach modelling and downstream risk analysis for Arjo-Dedessa dam has said, analysis and simulation of embankment dam breach events and the resulting floods are critical to identify and reducing threats due to potential dam failure. The conclusion said that effective emergency action plan requires accurate prediction of inundation levels and the time of flood wave arrival at downstream critical locations.

Dam breach analysis and inundation map for melka wakena dam was analyzed for the most happening breach scenarios which were piping and overtopping. This analysis was on the basis of only one dimensional unsteady flow routing technique used to carry out dam breach analysis. According to the magnitude of peak outflow discharge Overtopping was dangerous than piping. Finally the researcher recommended that the dam owner should give special attention to Dam breach analysis and make detail investigation by using the latest dam breach software's (Yonatan, 2016).

Jiregna (2016) conducted dam break analysis study on the case of fincha'a rock fill dam, Ethiopia. The main objective of this study was to analyse dam break by using hydraulic model for both piping and overtopping failure scenarios. The breach parameters were calculated by (Von Thun and Gillette, 1990) regression equation and used as input for unsteady flow analysis. Finally after the dam break simulation, the study concluded that the peak discharge formed by overtopping scenario was more devastating than that of piping.

Tesfa (2016) said that, dam breach analysis was essential for setting out risk management, development of emergency action plan, to protect both life and property damage during sudden dam failure. To carry out this the pre-event dam breach analysis study in the case of Gidabo dam for both overtopping and piping scenarios has performed. Finally the conclusion was, the analysis should have checked for all probable scenarios to identify the worst scenario for the development of effective and economical emergency action plan.

As Abimeal (2015) dam breach analysis study in the case of Kesem kebena, the results from such analysis can be used to protect downstream population from risk and it can also be used while designing and implementing future infrastructure. The dam has been checked for both overtopping and piping using one dimensional river analysis model under the empirical equations used to predict dam breach parameters. The modeling process was unsteady flow calculations in the intent of routing breach outflow up to downstream of 60 Km from the dam axis.

Adnan (2017) said that, Dam breach analysis was essential to predict dam breach parameters, outflow hydrograph and its downstream nature of propagation. As per this study analysis Koga Dam found in the Tana sub basin of Abay Basin in Ethiopia has been selected as a case study. The dam has been checked for both overtopping and piping using one dimensional river analysis HEC-RAS model and empirical equations to predict dam breach parameters for the input in the model.

Haile (2018) has studied Dam Breach Analysis in the case of Gidabo dam under overtopping scenario. This analysis was two dimensional unsteady flow simulations by using inflow hydrograph as upstream boundary condition and normal depth as downstream boundary condition. Finally for about 2050 ha area with a maximum depth of 12.14 m was delineated.

Hadush (2019) proposed dam breach analysis and emergency action plan in the case study of Mihtsab azmati dam. The author said that, due to the repetitive occurrence probability of overtopping and piping scenario by many researchers Mihtsab azmati dam was analysed for these scenarios. To perform the analysis the breach parameters were estimated by the regression equations But the Macdonald & Langridge-Monopolis (1984) equation was the most accurate to take the final parameters for next model analysis. Finally the author determined that, overtopping was the worst scenario to estimate downstream risk on population and property, and to develop an integrated emergency action plan.

Addisalem (2019/20) examined dam breach analysis in the case study of lower awash dam. According to this thesis observation, dam breach was essential to investigate the future effects posed to human life and property by a sudden release of water to the inundated area downstream of the dam. According to this analysis, the probable maximum flood inflow hydrograph was taken as upstream boundary condition to model the dam breach.

Tesfay (2019/20) carried out one dimensional dam breach analysis and downstream inundation using HEC-RAS and HEC-Geo RAS at midimar dam reservoir. The geometric files were extracted by HEC-Geo RAS and, the breach parameters also by empirical formulas. Finally the dam breach modelling was performed by HEC-RAS and the breach outflow has been quantified.

General: from the above Ethiopian authors result, both overtopping and piping were the only scenarios analysed and studied as the main cause of dam failure in Ethiopia. Due to its rainy season event all of them have said that overtopping was the worst scenario. So they recommended that, any downstream further analysis activity like risk assessment, development of warning evacuation, preparing of emergency action plan and emergency recovery, and budget recovery should be in concern of this scenario. Therefore, Gerhu-sirnay dam breach analysis would be tested for those hot dam failure scenarios in Ethiopia and its flood damage assessment would also for the worst scenario.

2.6. Models Used

2.6.1. Dam Breach Analysis Modelling

Dam breach analysis mainly involves the generation of outflow hydrograph, channel routing, delineation of the area exposed by flood and quantification of flood damage.

To undertake these procedures many models were developed in different time's categorically as one and two dimensional aspects. Therefore in order to select the latest and accurate model to analyze Gerhu-sirnay dam breach analysis and downstream flood mapping, the following previously worked studies were reviewed according to their time order of study as in the following way:

According to FEMA (2013) the models used for the analysis of dam breach were categorized based on their performance. The classifications were based on their Breach outflow hydrograph generation only, one dimensional and two dimensional performance aspects. In a summarized form they were given in the following table.

Table 2-7: The most used dam breach analysis models

Methods	Peak discharge generation	Breach parameters	Breach hydrograph	Downstream routing capability			
				Steady state	Unsteady state	1D	2D
Breach hydrograph generation only							
Empirical Equations	X	X					
NWS BREACH	X	X	X				
USACE HEC-1 and HEC-HMS	X		X	Without downstream hydrologic routing	X		
One-dimensional models							
WinDAM		X	X				
NWS SMPDBK	X			X		X	
NWS FLDWAV	X	X	X		X	X	
USACE HEC-1 and HEC-HMS	X		X	Downstream hydrologic routing	X	X	
USACE HEC-RAS	X		X	X	X	X	

Two-Dimensional Models							
MIKE© FLOOD	X		X		X	X	X
HEC-RAS Version 5.0.3	X	X	X	X	X	X	X

Source: FEMA (2013)

According to the review of FEMA (2013) on the above table, HEC-RAS Version 5.0.3 model was the best and have well performance. This is the decision from one governmental organization but to take better decision and latest dam breach modeling of this study, the review of other private researches was mandatory and carried out in below paragraphs.

Cook et al. (2015) conducted a study on selection of dam breach inundation software. The research has assessed HEC-RAS 1D, 2D and much other hydraulic software. In their experience they used HEC-RAS 5.0 coupling of one-dimensional and two-dimensional modeling for dam breach analysis but still no significant result difference were observed with the pure two-dimensional analysis rather they mentioned that, the coupling model was tedious in case of 1D data editing. According to the conclusion of the analysis HEC-RAS 2D was more stable than HEC-RAS 1D, in addition to the tabular and graphical result details the model has animation capabilities to show change of all concerned parameters with respect to time, in case of result validation the model also reports errors and warnings during computation to avoid errors with in the input data to provide a valid result. Therefore instead of data validation and input data feeding, the model was time saver and experience valid result.

HEC-RAS and HEC-GeoRAS were used to model dam failure. HEC-GeoRAS was used to extract geometric data from digital terrain model and imported to HEC-RAS. Next unsteady flow analysis has performed by HEC-RAS and results were mapping using GIS. Finally the results prepared accordingly were an input for floodplain managers and emergency management personnel to protect against the loss of life and property damage (Ackerman and Brunner, 2006).

Quiroga et al. (2016) investigated 2D flood simulation on the plains of Llanos de Moxos in Bolivian Amazonia which was continuously being flooded by Mamore River. They used HEC-RAS with two dimensional (2D) capabilities for simulating February 2014 flood in Bolivian Amazonia. After they also compared the results obtained from HEC-RAS 2D hydraulic model simulation and numerical model with satellite images of the flood event, they observed that HEC-RAS 2D hydraulic model simulation shows good performance. Finally they concluded that, the new HEC-RAS 2D was an important tool for studying and understanding the flood event.

Thornton (2016) presented 2D HEC-RAS model development in data poor areas of India. The author recommended that, flood inundation modelers in an area with limited data were suitable to use 2D HEC-RAS model with a typical reason that due its capability of reporting errors and warnings it can't need calibration with observed data.

Tesfa (2016) used HEC-RAS version 5.0 to model the dam failure and HEC-Geo RAS in connection with Arc Map for extracting basic geometric data serves input data for HEC-RAS. Next to this development of terrain modelling was based on DEM, finally the breach parameters were estimated by the regression equations used to compute unsteady flow analysis.

Jiregna (2016) quoting Gray (2014) that HEC-RAS was a tool used to compute breach outflow hydrograph and route the hydrograph using a steady or unsteady state solution. The researcher said that, HEC-RAS can also be used to route an inflowing hydrograph through reservoir with any of the three methods: one-dimensional unsteady flow routing (full Saint Venant equations), two-dimensional unsteady flow routing (full Saint Venant equations) and with level pool routing. Finally the author proposed, the breach outflow discharge estimated from HEC-RAS model must be compared with an envelope curve, developed from set of historically breached dams.

According to the analysis result of Ekaningtyas (2017) 2D Flood inundation prediction of overtopping scenario due to Logung dam breach in Kudus Regency, Indonesia was carried out using HEC-RAS 5.0 modelling software. The methodology showed that the dam breach parameters used as inputs of the model were first calculated by empirical formulas integrated with in the HEC-RAS model. Due to comparison of those formulas the author used only

Froehlich's equation. Finally by computing the simulation of unsteady flow analysis from 56 m high and 20.15 million m^3 reservoir the generated outflow hydrograph was with the 15,022 m^3/s peak discharge though the dam body. The 2D-simulation result showed that at the downstream of Logung Dam, the maximum depth was 55 m and the maximum velocity was 39 m/s.

Amini et al. (2017) have studied estimation of peak flood for overtopping and piping scenario at Vahdat Dam, Kurdistan Iran. In this research dam break was simulated by integrating HEC-RAS and Arch Map. The downstream flood area bed roughness was collected by field measurements and observations. Dam breach parameters were calculated by empirical equations and finally the study recommended that, dam break result was dependent on the proper selection of break parameter estimator methods.

Joshi and Shahapure (2017) studied 2D dam break flow for Ujjani Dam in Pandharpur Region, India. They used HEC-RAS 5.0.1 (2D) model to simulate the unsteady flow analysis by wave diffusion equations to analyse the flood susceptible area of Pandharpur Region, Solapur District and Maharashtra. The 30 m resolution DEM was used to create terrain model in the RAS mapper next the reservoir, inline structure and downstream flood inundation area setup has been accomplished. Finally the inundation area was created as mesh rather than feeding cross section of the main river due to the fact that the water flows by searching the lowest river bed elevation up on the mesh of the terrain model.

Deal et al. (2017) gave a comparable study on One- Dimensional and Two-Dimensional Hydraulic Models for River Environments to minimize the limitations with hydraulic engineers from considering only unidirectional river flow analysis on the basis of floodplain and flow velocity calculation accuracies. Their comparison covers three models with the first one was one-dimensional HEC-RAS version 4.1 and two of them were two-dimensional HEC-RAS versions 5.0, 5.0.1, and 5.0.3 and SRH-2D version 3.0. Then the result shown that, 2D models have higher quality results, one-dimensional models are due to the assumption that all the incoming velocities at every cross section points must be normal but regularly this is not true in real rivers whereas two-dimensional models are based on the velocity vectors along a given horizontal mesh in two directions with officially better hydraulic occurrence in a given river.

In case of 2D models regarding to simulation time and flexibility they said that, HEC-RAS 2D simulation was faster than SRH-2D with the same computational equation and model extent by 32% to 149% times. HEC-RAS 2D also provide better flexibility in controlling both external boundary conditions.

HEC-RAS hydraulic model was used to analyse floodplain mapping for part of Kabul River in Pakistan. Then they computed the model to find out the corresponding flood situation expected throughout the river reach from Warsak dam to Attock. Then after, the result obtained from this model were exported to ArcGIS to carry out the flood mapping requirement processes like flood plain mapping to identify the flood extent and areas that would inundate. Their conclusion shows that for about 400% of the normal river flow was magnified and over lied over agricultural land. Finally the developed conclusion said that, HEC-RAS in combination with Arc GIS provides more realistic flood plain results (Kumar et al., 2017) quoting Khattack et al. (2016).

Kumar et al. (2017) quoting Hicks and Peacock (2005) has suggested that HEC-RAS steady flow analysis model was used for flood forecasting and routing. The application of this model was very appropriate to accurately delineate floodplain, accurate for flood forecasting, no needs a second model for related results and no need for calibration of Manning's n but still advised to investigate the sensitivity of the parameters.

Alzahrani (2017) conducted the application of comparing 1D and 2D hydraulic modeling system using HEC-RAS in Dayton. According to the review, the result recommends modeling of flood plain was better to be based on 2D-Dimensional analysis. The author said that the center of 2D flow analysis was creating mesh of the flood affected area and connects it with the dam axis and reservoir to make a complete model of 2D-dimensional flood analysis.

Abimeal (2015) has used models HEC-RAS and HEC-GeoRAS alternatively. HEC-GeoRAS extracts topographic data from Digital Elevation Model (DEM) and prepares geometric file in ArcGIS. By using the geometric files imported into HEC-RAS and unsteady flow data (PMF inflow, initial flow and normal depth); the unsteady flow analysis was performed in HEC-RAS. Then finally the inundation data were exports to Arc GIS to prepare a flood map.

According to the study by Haile (2018) on dam breach analysis, HEC-RAS 2D was used to analyse the breach modelling and Arc GIS to prepare inundation maps. According to the methodology of this thesis dam breach parameters were calculated from the regression equations integrated with HEC-RAS 2D calculator tab.The findings of this study have shown that the right side irrigation command area with in the inundation area delineated was the most vulnerable area for flood.

Yakti et al. (2018) have studied 2D modeling of flood propagation due to overtopping failure of Way Ela Dam on 25 July, 2013 in Ambon, Island. They used HEC-RAS version 5.0 2D modeling to simulate the overflow flooding which severely damaged houses and many public infrastructures. According to their result, they determined peak overflow 1,268.4 m^3/s, maximum time arrival as 2 hrs and the maximum depths in the low zone, middle zone and upper zone near the dam were 6 m, 7 m and 8 m respectively. Finally they concluded that, maximum flood depth was happened at the topographically narrow flood plain area.

Faudzi et al. (2019) conducted detailed of 2D HEC-RAS Simulation on Sultan Abu Bakar Dam failure on October 23, 2013 due to overtopping scenario to identify the area affected by flood and to prepare risk predictable hazard maps. The unsteady flow analysis simulation was carried out using high resolution DEM to prepare terrain model, upstream and downstream boundary conditions as flow hydrograph and normal depth respectively and reservoir-inline structure-2D flow area mesh setup as river analysis system modelling. As per the result, the outflow hydrograph was generated with 300 m^3/s peak outflow and 50 m^3/s maximum safest outflow to the downstream reach. The downstream maximum inundation extent as per the peak outflow was 4 km long by 0.2 km wide which was the cause for three live losses and damages over 100 residential houses.

General: all the studies discussed in the above paragraphs decided that, HEC-RAS 2D was the best and latest model to analyse dam breach studies. According to the review, the model has better performance in addressing and solving all problems raised from dam breach analysis and has a good connectivity with Arc GIS. Therefore in this study, HEC-RAS 2D would use to model Gerhu-sirnay dam breach analysis.

2.6.2. Downstream Flood Damage Assessment Modelling

LIFE-Sim can be used for dam safety assessment, risk assessment and to explore options for improving the effectiveness of emergency planning and response by the dam owner and local authority. It has integrated with HEC-FIA model to estimate dynamic life loss scenarios. During dynamic analysis with this model sensitivity studies were presented for varying the warning initiation time and emergency shelter location cases to obtain accurate life loss reports (Aboelata et al., 2004).

According to Mohammod et al. (2014) study on flood damage quantification in the case study of Neka River, HEC-FDA was used to quantify the expected annual damage from the flood risk analysis. This model was developed by US Army Corps of Engineers and examined and accepted for international tool to calculate annual flood damage statically.

Nafari (2012) has carried out a study on flood damage assessment with the help of HEC-FIA model. They said that HEC-FIA can calculate both direct and indirect damage assessment. According to their work GIS pre-processing steps should accomplished in order to extract the main input data of HEC-FIA. The model focuses on flood impact assessment of Structural damages: Direct and tangible, Content Damages: Direct and tangible, Agriculture losses: Direct and tangible, Life Losses: Direct and Intangible.

According to Lehman and Light (2016) HEC-FIA depends on the geospatial datasets to prepare structure inventories/details and populations per structure. Based on this information, HEC-FIA estimates direct and indirect economic and life loss consequences for flood hazards.

Buchanan (2016) has suggested a detailed investigation on Consequence assessment for dam failure simulations using HEC-FIA. As checked in this study, flood risk was the function of both probability and consequence. The model was flexible for importing input hydraulic data form HEC-RAS 2D like depth grids, arrival time, inundation extent, flood duration, water surface elevation and the like according to the project extent. After that the model prepared consequence estimation variables like: warning system curves, mobilization curves, evacuation velocity and warning time relative to breach imitation time. According to this methodology the model works impact analysis in a physical damage for economic analysis and life loss estimates during day and night working scenarios. Online with email address

“Kurt.L.Buchanan@usace.army.mil” I was contact him and he advised me regarding to HEC-FIA input data with their format from HEC-RAS 2D and its flexibility with Arch GIS.

ESTDM model were developed in England by FHRC. This model applies the property by property approach and would be matched with standard depth-damage data provided by FHRC (Penning et al., 2003)

In Germany the engineering consultancy ProAqua developed an access based tool called HWSCalc. In Netherlands GIS based software called HIS-SSM has been developed on the behalf of highway and hydraulic engineering department (Messner et al., 2007).

United Kingdome has developed decision support software tool which was MDSF. This model has the capability of calculating economic damage based on the damage standard prepared in FHRC (DEFRA et al., 2004).

General: from the discussions in the above paragraphs, HEC-FIA has good flexibility and connection with HEC-RAS 2D and Arc GIS selected for modelling dam breach. Due to this reason, flood damage assessment of Gerhu-sirnay dam would be performed with HEC-FIA Version 2.2. Therefore, HEC-RAS 2D, Arc GIS and HEC-FIA were the models selected to perform the study on dam breach analysis and downstream flood mapping a case study of Gerhu-sirnay dam.

After the review of many researches in section 2.6.1 and 2.6.2 for dam breach modelling and flood damage assessment model selection respectively, HEC-RAS 2D and HEC-FIA has selected. However, the detail of these models selection criteria’s has needed to be clear for the reader. Therefore it was provided briefly in section 2.7 below.

2.7. Models Selection Criteria

Selecting an appropriate model is the turning point for developing accurate modelling application. From the model’s used review section 2.6 above, much specific information have gained to select an appropriate model to compute Gerhu-sirnay dam breach analysis and flood damage assessment.

Therefore according to the review, the common points of selection were listed below:

- Data availability
- Model performance for addressing all or most objectives

- GIS connection nature of the model
- Model short step process, time and effort saver and accuracy of result
- Type of results needed
- Model flexibility regarding to sequential task input output data exchange.

Therefore, the selected models for the analysis of this case study were: HEC-RAS 2D version 5.0.3 to model dam breach and to estimate breach parameters by the integrated regression equations with it, breach out flow and to route it through the 2D flood area with the help of simulation and gridded map to prepare inundation mapping details in a good connection with Arc GIS. HEC-FIA version 2.2 also selected for flood damage assessment in terms of both ex-ant direct economic and life loss damages. These models used in this case study were flexible one with another. This means, the outputs of HEC-RAS 2D were inputs for Arch GIS and the outputs of Arch GIS and HEC-RAS were also inputs for HEC-FIA.

2.8. Flood Damage Category

2.8.1. According to Flood Magnitude

According to the Australian Guideline for Flood Disaster Resilience (2017), flood damage was classified due to the magnitude of flood depth, velocity and combination of both velocity and depth. The peak flood damage may not occur at the instant of peak flood rather it depends on the size of the watershed. In small and medium catchments, the guide reported that, the exact occurrence of peak flood damage is determined through the numerical values of the combined depth and velocity (D*V). At the flood duration when the value of D*V is maximum refers, the flood damage will be high.

Therefore to decide the flood damage category and its time of occurrence, the parameters mentioned above were initially estimated using the available tools. In this study the selected available tool was HEC-RAS 2D version 5.0.3.

The classification was given in below table:

Table 2-8: Damage category

Damage category	**Up to D (m)**	**Up to V(m/s)**	**D* V (m^2/s)**
M_1	0.3	2	≤0.3
M_2	0.5	2	≤0.6
M_3	1.2	2	≤0.6
M_4	2	2	≤1
M_5	4	4	≤4
M_6	>4	>4	>4

Source: Australian guide for flood risk management (2017)

Where:

M_1 = Safe for people, vehicles and buildings

M_2= Unsafe for small vehicles

M_3 =Unsafe for vehicles, children and elders

M_4 =Unsafe for vehicles and people

M_5 =Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure.

M_6 =Unsafe for vehicles and people. All building types considered vulnerable to failure.

D= Depth and V=Velocity

According to Kreibich et al. (2009) conclusion on investigation of flow velocity as a significant parameter in flood damage modelling ‘‘the 2 m flow depth was critical impact level in terms of depth’’. Above this depth it was destructive for public infrastructures especially for residential houses.

2.8.2. According to Exposure Type

As per the categorization of Colorado State Guidelines for Dam Breach (2010), Hazard potential classification was the placement of dam failure hazards in to one of the four categories as summarized below:

A. **High hazard potential:** this type of hazard has been recorded if loss of human life, damage of public infrastructures and residential houses would be expected.

B. **Significant hazard potential**: a hazard category for which significant damage was expected to occur, which causes economic loss, environmental damage, disruption of lifeline facilities, or impact of other concerns, but no loss of human life would be expected. It also defined if damages occurred to: mass agricultural areas, residential areas, ecology and structures where people generally live; work, recreate and public facilities.

C. **Low hazard potential:** the hazard assigned as low-hazard potential classification were if the failure was not likely to result in loss of human life and public damages rather it causes only low economic and /or environmental damages related with private owners.

D. **No public hazard potential:** A category for which no loss of human life, damage for public and private property were expected rather damages expected only to the dam's constituent property was categorized as no public hazard potential.

2.9. Emergency Action Plan (EAP)

Developing emergency action plan was used to identify potential emergency conditions at a dam and specifies pre planned actions to minimize property damage and loss of life. The downstream inundation map and Hazard potential classification was the basis for developing Emergency Action Plan (FEMA, 2013).

According to US Bureau of Reclamation (1988) study for high and significant flood hazard potentials, EAP was a key tool to minimize life loss and property damage. The study mentioned that the preliminary objects to build EAP was preparation of detailed inundation maps. The purpose of EAP was to minimize the risk of loss of life and property in the downstream of the dam failed.

Juliastuti and Setyandito (2017) have said that, every dam needs EAP guideline to create a great awareness of the local communities. The first step of develop effective EAP was considering the capacity and awareness of local communities to interpret and practice the prepared emergency action plan.

3. MATERIALS AND METHODS

3.1. Description of Study Area

3.1.1. Location

Gerhu-sirnay dam site is located at Mereb Basin: Sub- Basin V Tigray, Ethiopia with a total distance of 1085 km from Addis Ababa. The geographic coordinates of the study area lies between 14º27′0″N -14º29′0″N north and 39º8′0″E -39º10′0″E. The site was accessible with 27 km gravel road from Mekelle to Adwa Asphalt road (TWRB, 2016).

3.1.2. Watershed

Watershed is the total area from which surface runoff flows in to common point of confluence or watershed outlet. According to the classification depending on the arrangement of drainages with in it, watershed type of this study is Dendric (Strahler, 1957). This watershed is mainly characterised by acute junction of drainages with the main river as displayed in below figure. The main parameters including: length of main river, total area, time of concentration and average slope of the watershed were 4400 m, 14.27 Km^2, 0.84 hrs and 0.05 respectively (appendix E, i).

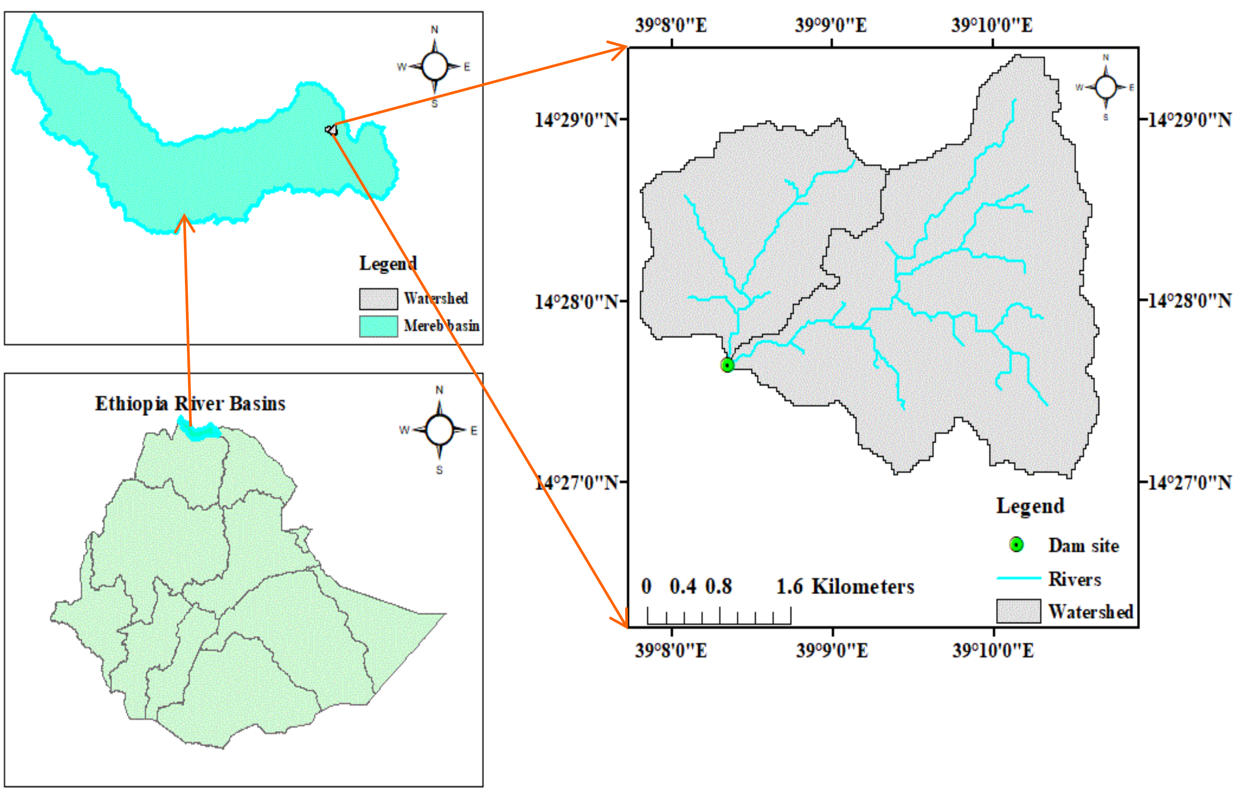


Figure 3-1: Location of Gerhu-sirnay dam site in Mereb, Ethiopian River Basins

3.1.3. Topography

According to the 30 by 30 DEM resolution detail of MoWIE (2019), the general topography of the watershed ranges between 1818 m to 2053 m a.m.s.l. The lowest point which is 1818 m a.m.s.l was the dam site up on the main river flow mass center as prepared in the map below:

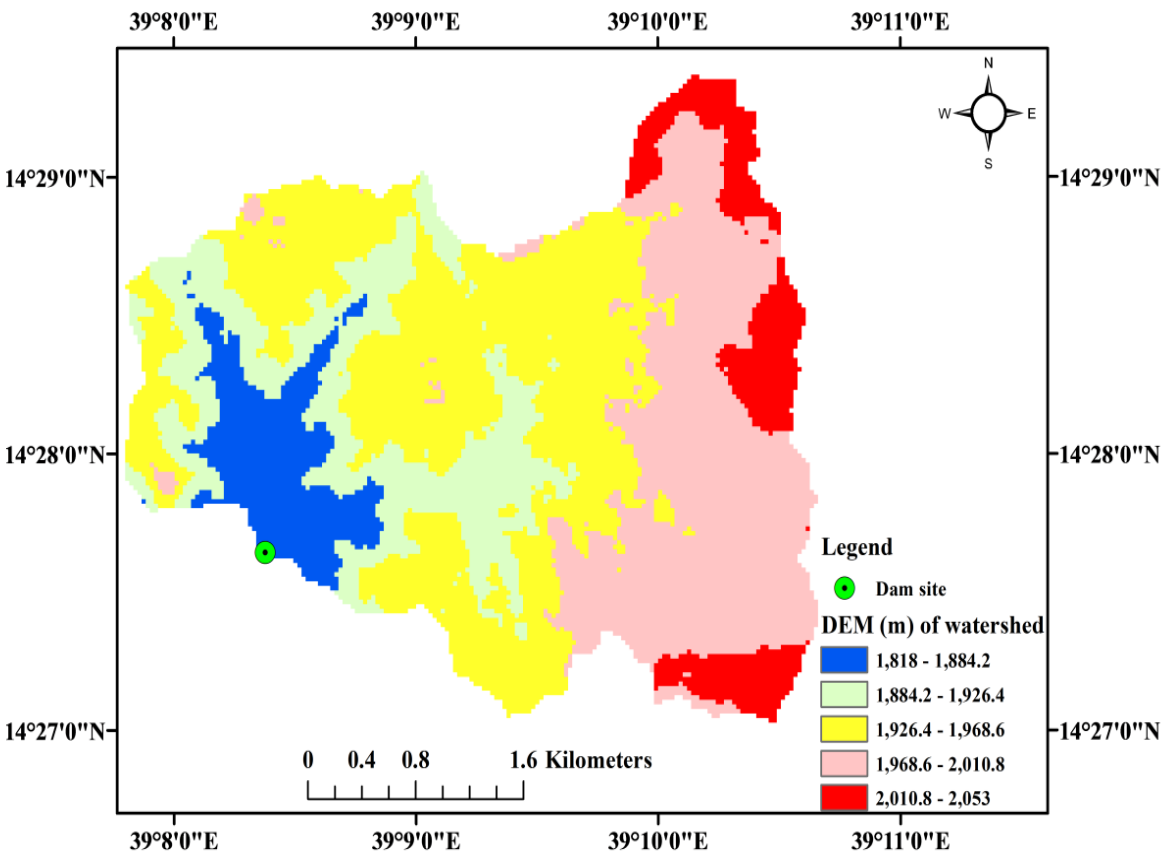


Figure 3-2: Topography of the watershed

3.1.4. Meteorological Data

3.1.4.1. Availability of Rainfall

According to EMA (2019), the study area has only one meteorological station which is Gerhu-sirnay station with 21 years (1998-2018) daily rainfall data series with 7.28% missing precipitation data. The rainfall distribution of the area was not uniform throughout the year. Moreover it looks that, 60% of the rain was received only in the two months of the year (July and August), June and September contribute 26% and the remaining eight months receive 14% of the annual rainfall.

The geographical location of Gerhu-sirnay station is visualized on the below map:

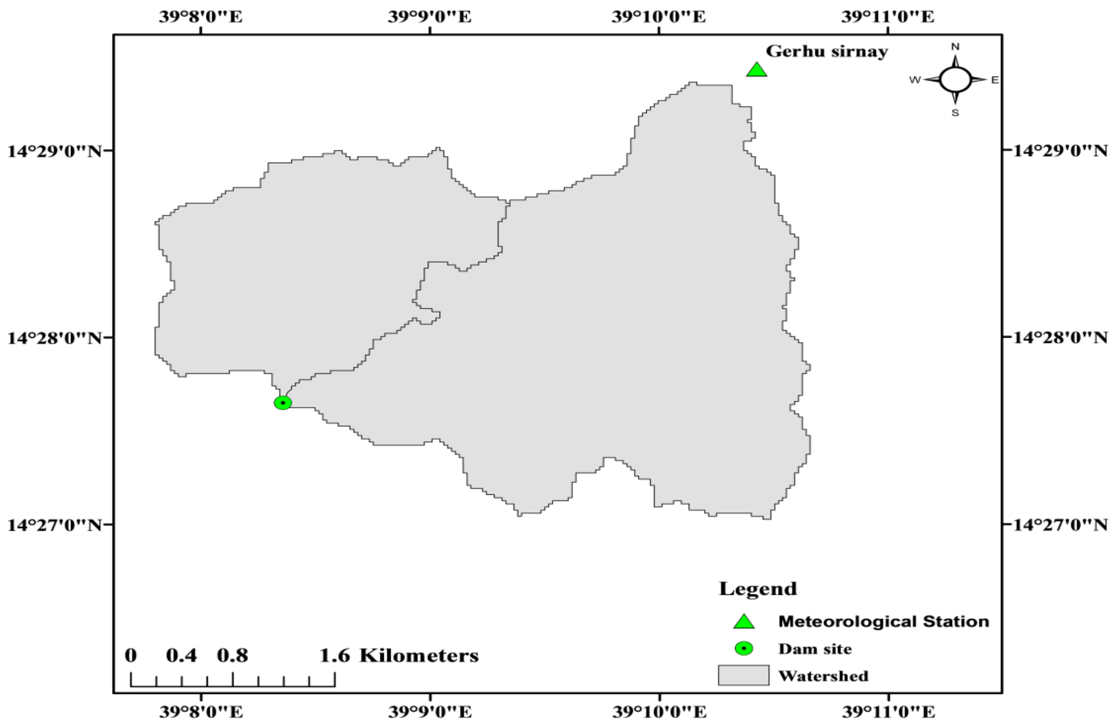
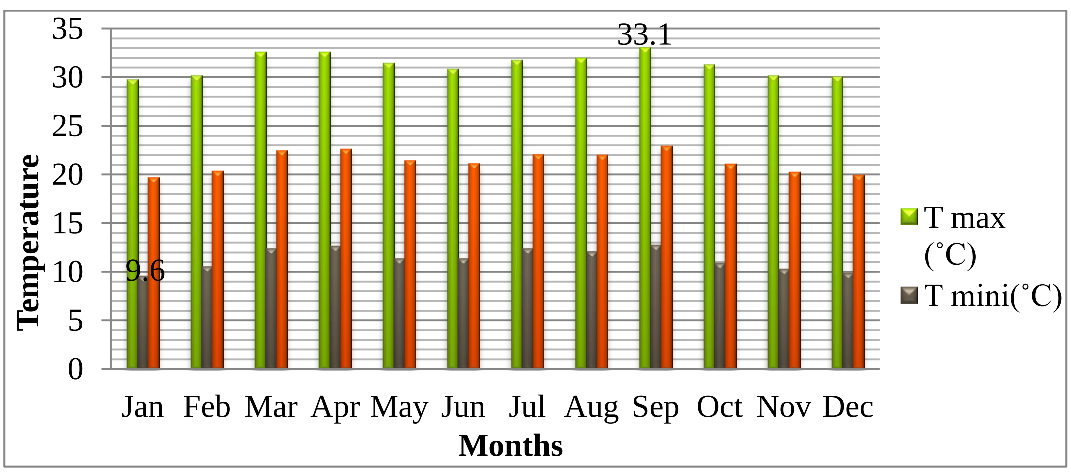


Figure 3-3: Location of meteorological station

3.1.4.2. Temperature

The Average daily temperature of the study area due to Gerhu-sirnay station varies from 33.1˚C to 9.6˚C. The maximum temperature of the study area was 33.1˚C recorded in September while the minimum temperature is also 9.6˚C recorded in January (EMA, 2019). The temperature vararion of the study area was displayed in below chart:



Source: EMA (2019)

Figure 3-4: Temperature variation of Gerhu-sirnay station (˚C)

3.1.5. Agricultural Data

3.1.5.1. Soil Type

According to the soil classification data obtained from MoWIE (2019) GIS department, the study area has only two types of soils due its minimal size as displayed on the following map. The classification was according to FAO and the larger area is covered with Haplic Xerosols (76.7%) and the rest one is Orthic Solonchaks.

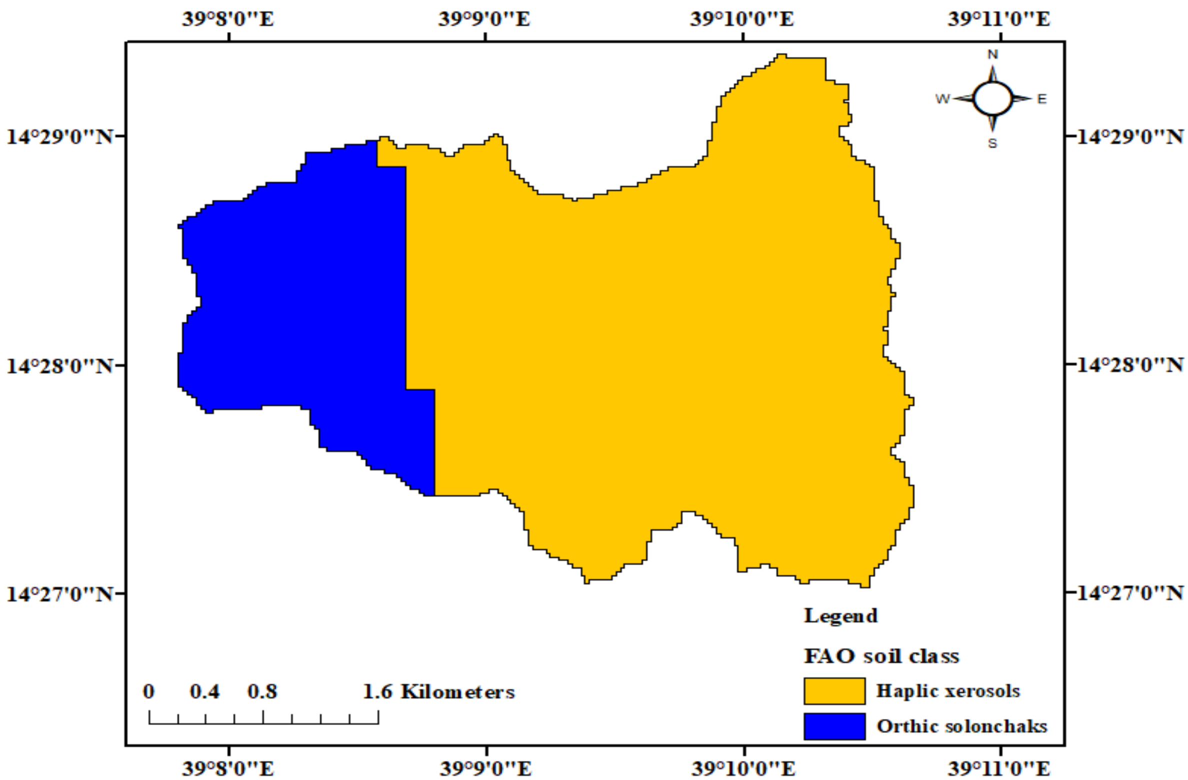


Figure 3-5: Soil classification of the watershed

3.1.5.2. Land Use Land Cover

From the GIS result, the total area of Gerhu-sirnay dam's watershed is 1,427.2 ha. Since the dam is small scale its watershed is also proportionally small. According to the land use land cover data of MoWIE (2019), the largest area(>50%) of the watershed was covered by field crop land with good hydrologic condition and the remain also open grass land and sparse shrubs in a fair hydrologic condition.

For better visualization of these parameters, the following map prepared:

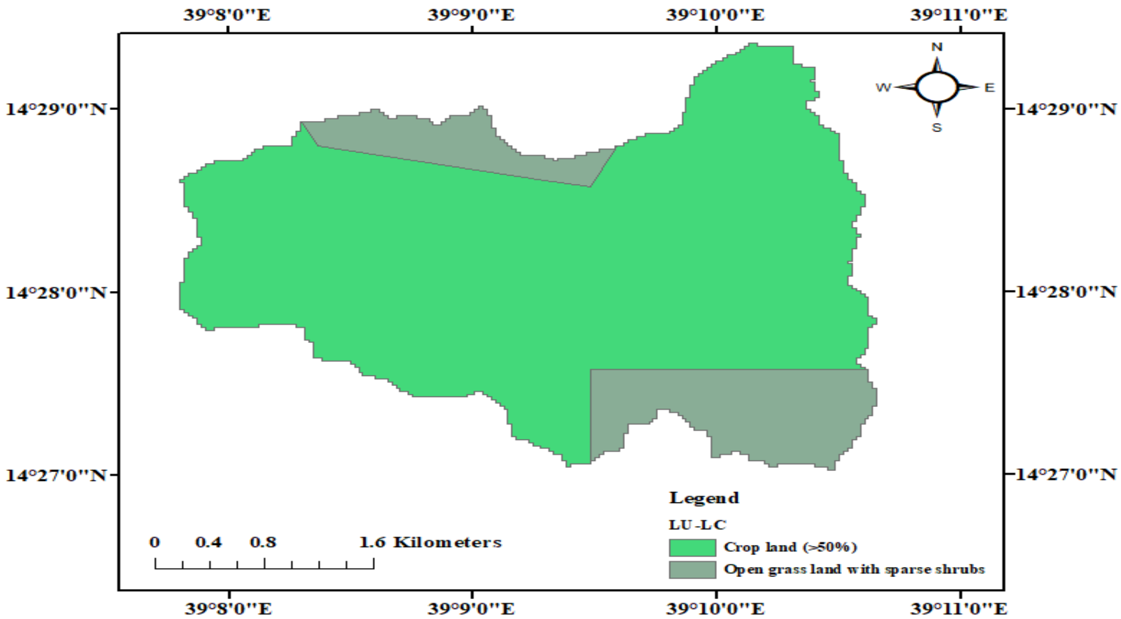
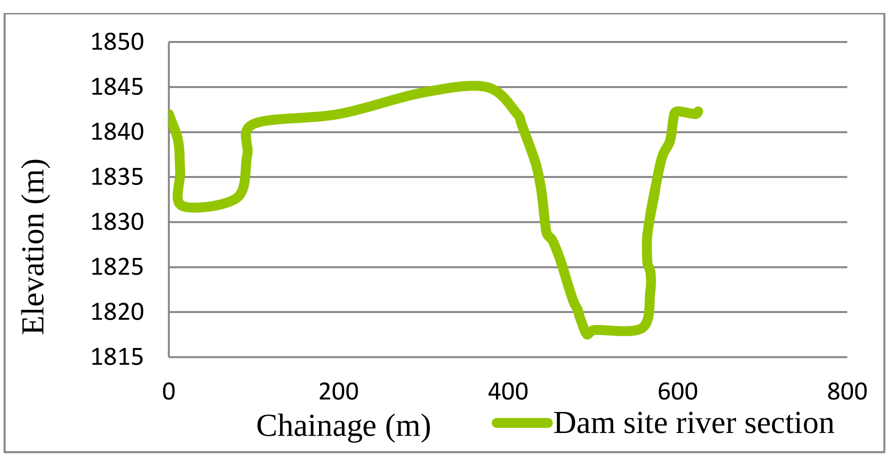


Figure 3-6: Land use land cover of the watershed

3.1.6. Dam Characteristics

3.1.6.1. Dam Site River Cross Section

The original ground level points of Gerhu-sirnay dam site has obtained from the design document of the dam (TWRB, 2016). By using these points and scatter plot as below, the lowest bed elevation of the river at the dam site was 1818 m a.m.s.l and the dam has constructed over such type of river valley (river section).

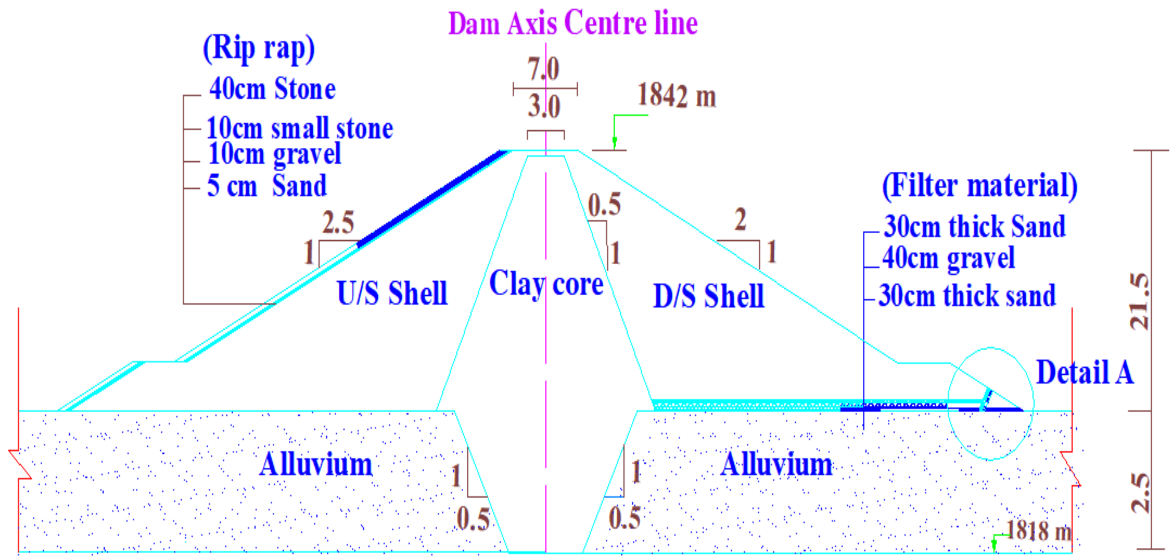


Source: TWRB (2016)

Figure 3-7: Dam and spillway site river section

3.1.6.2. Dam Type

Gerhu-sirnay dam is zoned earth fill with central clay core; alluvium basement and sand-gravel filter materials as shown in below figure. Its main purpose was to supply drinking water for Gerhu-sirnay town.The crest length, top width and height of the dam are 210 m, 7 m and 24 m respectively. During the data collection time (2011E.C.), the dam was completed but its water treatment plant was under construction at the downstream of the dam by TWWCE. The sectional view of the dam was shown below:



Source: TWRB (2016)

Figure 3-8: Sectional view of Gerhu-sirnay dam

3.1.6.3. Reservoir and Spillway Capacity

The maximum storage capacity of Gerhu-sirnay dam was $1.08*10^{6}$ cubic metres with a reservoir area of 15.66 ha for 20 years useful life (TWRB, 2016) (the detail of reservoir capacity curve has given in appendix C). The spillway type with this reservoir was ogee side spillway with 1842 m a.m.s.l crest level, 20 m crest length, 10 m high and located 320 m apart from the right side abutment of the dam with a maximum routing capacity 105.5 m^3/s.

3.2. Conceptual Framework

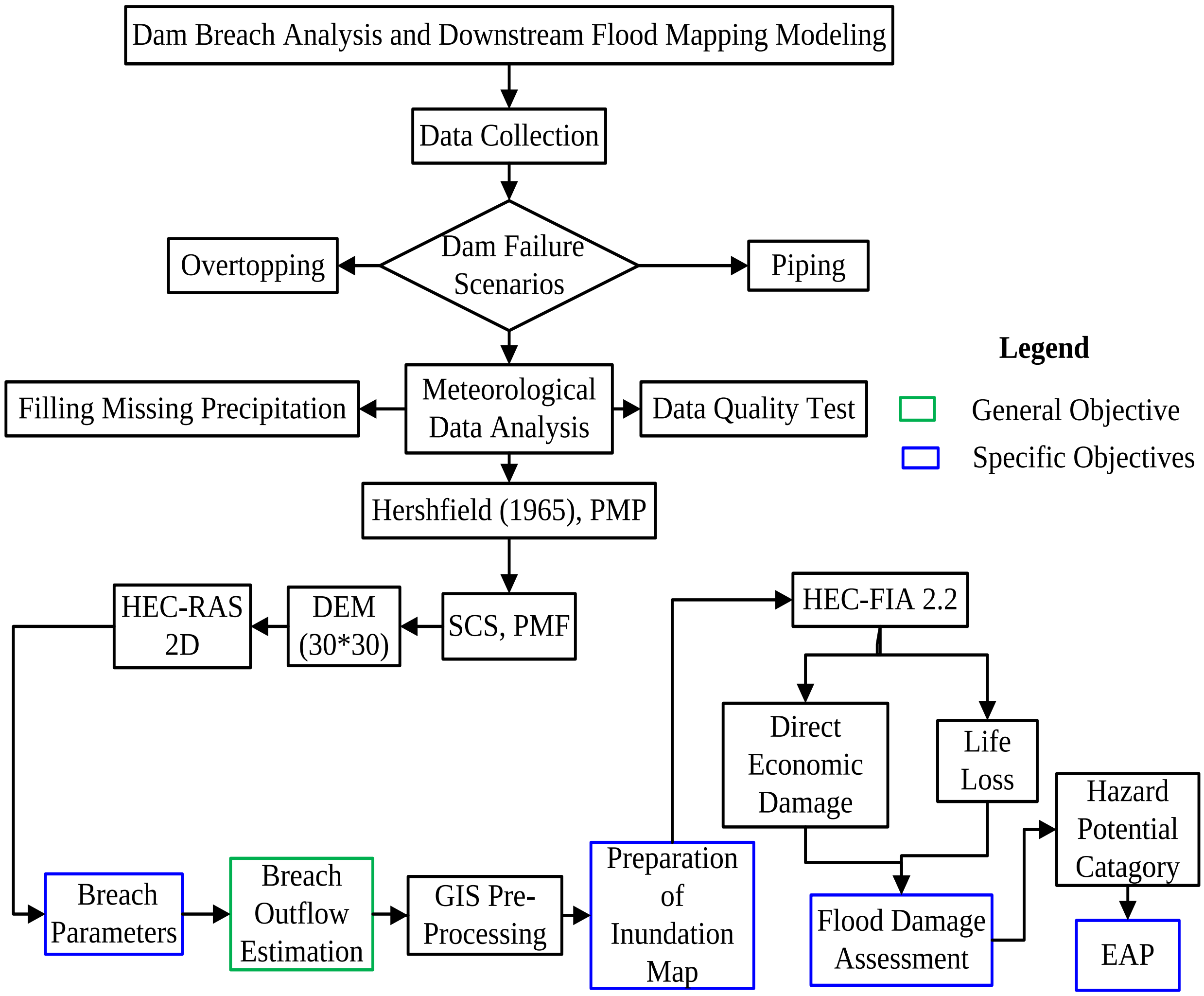


Figure 3-9: Overall research work flow chart

3.3. Data Collection

This study needs a lot of data categorically as primary and secondary data which collect according to the following ways:

3.3.1. Data Type and Collection Methods

A. The primary data location of watershed outlet, dam axis elevation and coordinate, location of upstream boundary condition, treatment plant inventory at that time and existing feature of agricultural area downstream of dam would collected through the method of field survey using Theodolite, frequent and careful specific site observation regarding to the required data type.

B. **The secondary data:** The 30*30 DEM, water treatment plant details (treatment plant geo spatial location, structural design specification, content equipment types and cost, the number of stakeholders, the number of vehicles like transportation service), dam and its appurtenance structure design detail, crop types and its coverage of 2D flow area, meteorological data with station location would collected from (TWRB, 2016).

Table 3-1: Summary of overall data collection

Data category	**Data type detail**	**Data source**
Primary Data	Survey data: reservoir polygon, location of spillway and treatment plan, existing feature of agriculture	Field collection and TWRB (2016)
Secondary Data	DEM with 30*30 m resolution and GIS data	MoWIE (2019)
	Meteorological data (precipitation & temperature)	EMA (2019)
	Dam, spillway and site characteristics	TWRB (2016)
	Crop type & coverage of flood area and area-production relation of crops with Ethiopia regions	Ahferom Wereda Agricultural sample survey (2019) & EASSSA (2017/18)
	Water treatment plant details: design, cost, number of stakeholders with respect to their age, content cost and structure depth-damage cost	(TWWCE, 2019)

3.4. Identification of Dam Failure Scenarios

As the report of FEMA (2013) from 656 failed dams for dominance of failure scenarios, overtopping shows about 70.9% and piping 14.3%. According to TWRB (2019) report on Gerhu-sirnay dam for about 2 l/s seepage was happened due inappropriate core material selection and error arrangement of riprap. The selected HEC-RAS 2D models used to model dam breach of this study was also supports only piping and overtopping scenarios. Theses all points show that both were the most common dam failure scenarios and the analysis of this study focuses on those repeatedly occurring problems. Therefore, the dam breach analysis and downstream flood mapping of this study was based on both overtopping and piping scenarios.

3.5. Data Analysis

The availability of reliable data determines the accuracy of study's result (Wijesekera and Perera, 2012).The data analysis carried out in this study was involved in below sections:

3.5.1. Filling Missing Rainfall Data

The missing of precipitation data along a given time series was appear due to inappropriate location of stations for continuous observation, lack of professionalism and equipment quality, environmental change due to natural hazards and political disorders resulting for data record gab and displacement of gauging stations for specific reasons (Mahmut et al., 2010). To fill these missing data, a number of methods are available including station average method, normal ratio method, regression methods and inverse-distance weighting method (Nebiyat, 2016).

The data series of this study's meteorological station (Gerhu-sirnay) has observed with significant missing as quantified in the rainfall availability section (3.1.4.1) above hence to fill this, the most nearby another one meteorological station (Enticho) was accounted. Due to the investigation among the methods of filling missing rainfall data and support from many researches, normal ratio method was accurate for this case study. Its main reason is as per definition of the test: if the normal annual precipitation at the surrounding station differs by more than 10% of the normal annual precipitation at station with missing, then the normal ratio method could be adopted to estimate the missing observation of the station (Chow et al., 1988).

The normal annual precipitation of Enticho and Gerhu-sirnay were 654.7 and 365.73 mm respectively hence the missed observation of this case study satisfies the criteria to be filled by normal ratio method. The formula for this method is:

$$\mathbf{P}_{\boldsymbol{x}} = \frac{\mathbf{1}}{\boldsymbol{m}} \sum_{\boldsymbol{i}=\mathbf{1}}^{\boldsymbol{m}} \left[\frac{\mathbf{N}_{x}}{\mathbf{N}_{i}}\right] \mathbf{P}_{\boldsymbol{i}} \text{ --- } (3\text{-}1)$$

Where:

P_X = Estimates for the ungagged stations

Pi = Rainfall values of the gauged stations

N_x = Normal annual precipitation of X stations

M = No. of surrounding stations and N_i = Normal annual precipitations of surrounding

3.5.2. Data Quality Test

3.5.2.1. Outlier

Outliers are points up on the precipitation series used to limit the real lower and upper precipitation values (Nebiyat, 2016). The values from the series above the highest datum and below the lower datum were removed and again filled according to the missing concept (Chow et al., 1988). However, if the outlier values were with in the series regarding to each bound it easily replaced by the respective datum values (Wijesekera and Perera, 2012). According to this, the American Water Resource Association Statistical method was the best way of detecting the real outlier. According to the concept of this test, the value of coefficient of skewness (C_s) detailed in table 3-2 below has great role with three cases.

Table 3-2: C_s values

Case 1	If Skewness (C_s) < -0.4 check only for lower outlier
Case 2	If Skewness (C_s) > +0.4 check only for higher outlier
Case 3	If Skewness (C_s, -0.4<C_s< +0.4 check for both outlier

Source: Chow et al (1988)

The formula for Coefficient of skewness provides below:

$$C_s = \frac{N\sum_{n=1}^{n}(\mathrm{Y_i} - \mathrm{Y_m})^3}{(N-1)(N-2)(\delta_\mathrm{n})^3} \text{-----------------------------} (3.2)$$

Where:

C_s=Coefficient of skewness

N=Number of sample size

δ_n=Standard deviation

$\mathrm{Y_i}$ =Value of individual element

$\mathrm{Y_m}$= Mean value

The formulas for lowest and highest datum are given below:

Y=log X

$$\mathrm{R_L} = 10^{Y_L} \text{-----------------------------} (3.3)$$

$$Y_L = Y_{av} - K_\mathrm{n} * \delta_\mathrm{n}$$

$$R_H = 10^{Y_H} \text{-----------------------------} (3.4)$$

$Y_H = Y_{av} + K_n * \delta_n$

Where:

X= Daily maximum precipitation of all stations

C_s=Coefficient of Skewness

R_L=Lowest datum

R_H=Highest datum, adoption

K_n=Constant number which was a function of precipitation series size, n varies from 0 to 140, can read from appendix B (Chow, 1959). For this research, n=29 & K_n =2.549.

From the method above, the calculated value of coefficient of skewness in this study was 0.69 which was greater than +0.4. Therefore as per the definition of this test, the test for outlier remains only with the highest datum. The calculated value of higher datum R_H from the test was 103.8 mm but the maximum precipitation value of the 21 years daily precipitation data series of Gerhu-sirnay station was 93.9 mm at 1998. Now this value is within the recommended upper value of outlier, R_H (for detail steps of the test see appendix D, i).

According to outlier test's concept; now the data series to model dam breach analysis and downstream flood damage assessment of this study were qualified for further processing.

3.5.2.2. Adequacy and Reliability

Adequacy and reliability of rainfall data series should be checked and realized by calculating the Relative Standard Error, δ_e and checks it with the rule of reliability below calculation methods (Wijesekera and Perera, 2012). This method determines whether the amount of error in percentage with in the data series were acceptable or not. The data series could be considered as an adequate and reliable if relative standard error (RSE), δ_e< 10%.

Therefore, the general equation for RSE was:

$$\delta_n = \sqrt{\frac{\sum(X - X_m)^2}{(N - 1)}}$$

$$S_e = \delta_n/\sqrt{N}$$

$$\delta_e = \frac{S_e}{Xm}\% \text{-- (3.5)}$$

Where:

N=Precipitation series size

X=Heaviest precipitation, mm/day of the given N years

X_m=Precipitation mean

δ_e= The relative standard error (RSE).

δ_n=Standard deviation of the precipitation series

S_e=Standard error

From the test's result the value for Relative standard error δ_e was 9.84% which was within 10% (for detail steps of the test see appendix D, i). Therefore, the sampling error within the 21 years precipitation data series of this study was acceptable and the data was ready or adequate for further analysis.

3.5.2.3. Homogeneity

Inhomogeneity of precipitation data series may arise from various reasons to hinder data series from being homogenous. Therefore, before using of such data for any hydrological modelling their quality must tested in terms of homogeneity to develop accurate and real model. The homogeneity nature of precipitation data series can be tested in absolute or relative methods (Mahmut et al., 2010) quoting Karab et al (2007). Absolute method has carried out for individual stations whereas the second method applies for more than one neighbouring reference stations. Due the lack of other neighbouring stations, this study considers only Gerhu-sirnay station except for filling of missing purpose, Due to this reason absolute type of homogeneity test has selected for this study.

Absolute homogeneity test performs by Standard Normal Homogeneity Test (SNHT), (Swed-Eisenhart) RUN test and Pettit (1979) test (Mahmut et al., 2010). However, due to the type of inhomogeneity location detections required, Pettit (1979) test was selected in this research. This type of test determines the data break or point of inhomogeneity within the time series record (Mahmut et al., 2010) quoting Costa & Soares (2009).

The mathematical detail of Pettit (1979) test has given below from Agha et al. (2017).

Provide rank for the annual observations (X), 1 to N. where N is number of observed series.

- V_i is estimated from the following relations:
  V_i= N+1-2R_i, where i=1, 2…..N and R_i is the rank of X_i over N observations.
- Estimate U_i from,
  $U_i = U_{i-1} + V_i$ for $U_1 = V_1$

- Calculate K_N from:

  $K_N = \max_{1 \le i \le N} |Ui|$

- The value of P_{OA} is estimated from:

  $P_{OA} = e^{[-\frac{6K^2{}_N}{(N^3+N^2)}]}2$-- (3.6)

As per the test, the null hypothesis has rejected when the value of P_{OA} is less than α, where α represents for statistical significance level of the test.

From the above noted procedures the value of P_{OA} was became 0.0011 that is less than the statistical significance level of the test, 0.05. Therefore, the developed null hypothesis was rejected by notifying the annual precipitation series of Gerhu-sirnay station has a break point at the year having a value of K_N which was 110. By chance, the values of K_N were located at 2007 and 2008 years, therefore as per the definition of Pettit (1979) test, the data series forms a break from 2007 year (for detail steps of the test see appendix D, ii).

Agha et al. (2017) concluded that, if a series was identified as inhomogeneous or with break point by Pettit (1979) test, the data series must be tried to make homogeneous by using double mass curve analysis starting from the point that shows break.

3.5.2.4. Consistency

Inconsistency of precipitation data series arises from the systematic errors during recording of the data (Wijesekera and Perera, 2012). From the result of homogeneity test above, the data series was inhomogeneous. Therefore in order to calculate Hershfield (1995)'s frequency factor Km, the data series must be first processed to be homogenous by using double mass curve with Enticho station. According to the concept of this analysis, the value for coefficient of determination R^2 lies between 0.75 and 1 but for best fit it should tends to 1. To carried out this, the applied governing equation was given below:

$P_{cx} = P_x \frac{M_c}{M_a}$--- (3.7)

Where: Pcx= Corrected precipitation at any time period

Px = Original recorded precipitation at time period,

Mc = Corrected slope of the double mass curve and

Ma = Original slope of the double mass curve

The double mass curve analysis was prepared below:

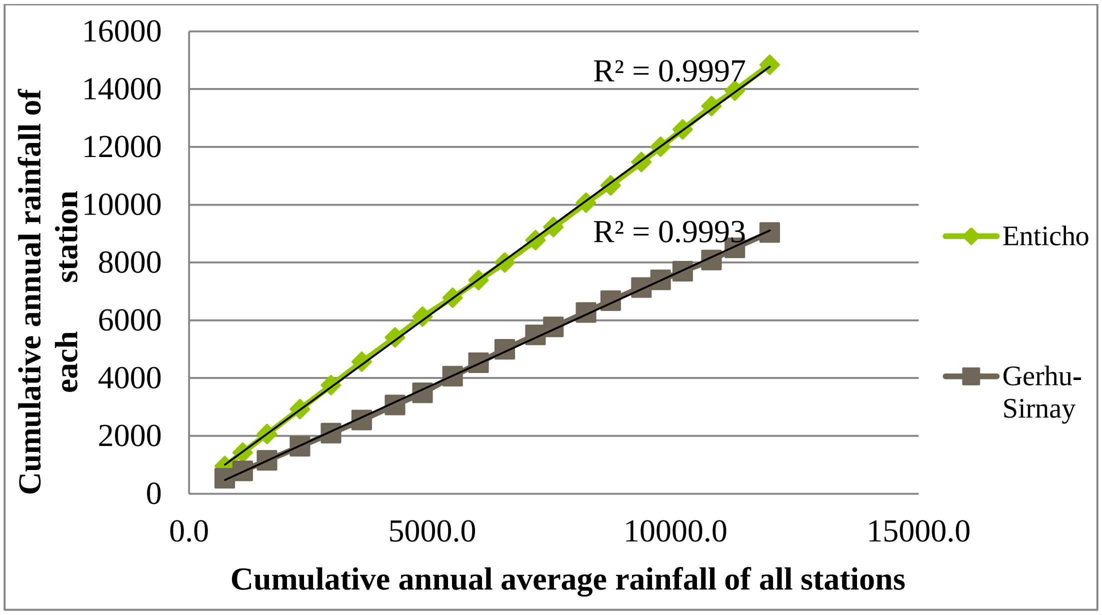


Figure 3-10:Double mass curve of rainfall stations

The coefficient of determination, R^2 result from the curve above was 0.9997 and 0.9993 for Enticho and Gerhu-sirnay stations respectively. Therefore the data series was homogenous and consistent and ready to calculate the Hershfield (1995)'s frequency factor, Km.

3.5.3. Estimation of PMP

3.5.3.1. Meteorological Station Point PMP

The application of probable maximum precipitation (PMP) determination was aimed to know the greatest depth of rainfall for the given duration that would possible meteorologically and may Cause PMF (WMO, 1986).

- **Qualifications of Hershfield (1965) for Gerhu-sirnay Dam breach analysis point PMP estimation:**

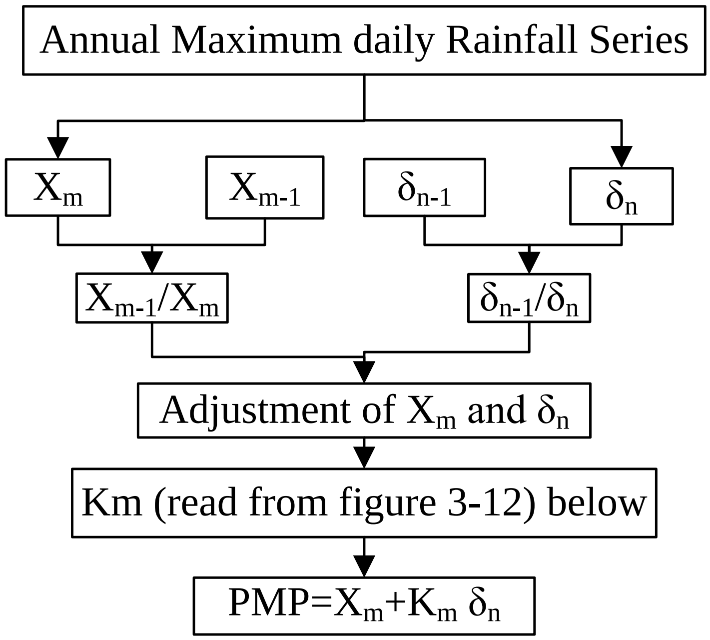
A. The formula was valid for the watershed size less than 25 km^2. As a result of this the watershed size of this case study was 14.27 km^2 (WMO, 1986)

B. This method involves data quality check: the X_m and δ_n was parameters calculated from a limited sample (n). So they need adjustment for sample size and effects of outlier (Salas et al., 2014).

C. Popularized internationally to practice in watersheds having lack of hydro-meteorological data (Salas et al., 2014).

- **Steps for Hershfield's (1965) Statistical Method:**

Hershfield's method was used to estimate meteorological station's point PMP. Therefore steps of Hershfield (1995) method of PMP calculation was listed in the below flowchart:



Source: Birhan (2017)

Figure 3-11: Hershfield's method to estimate PMP

Where:

X_{max}= maximum observation upon the series

X_m = Mean of the given series of n annual maximum rainfall values

X_{m-1}= Mean of the series excluding the maximum observation

δ_n= Standard deviation for the whole series

δ_{n-1}= Standard deviation excluding the maximum observation

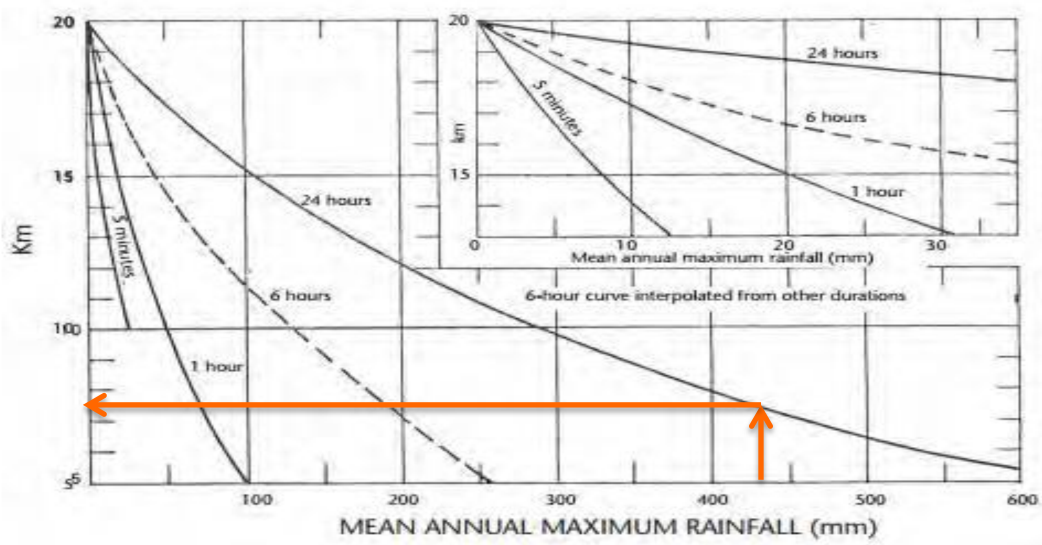
K_m=frequency factor

- **Frequency Factor (Km) from Hershfield's Curve**

Frequency factor has developed by Hershfield under the 2,700 stations with 90% of them were in USA and its maximum value was 15.

However, in 1995 Hershfield examined that that frequency factor value of 15 has not well matched for all stations of USA. Therefore, Hershfield tends to develop a curve providing Km variability from 5 to 20 regarding to rainfall duration and annual mean rainfall values of each station (WMO, 1986).

Then the frequency factor of this study was determined according to the Hershfield's curve as a function of mean annual rainfall and rainfall duration of Gerhu-sirnay station with 430.6 mm and 24 hours respectively. So the curve is provided below:



Source: Birhan (2017)

Figure 3-12: Hershfield's curve to estimate Km

From the above curve the value of frequency factor, Km correspondingly with mean annual rainfall and rainfall duration of Gerhu-sirnay station was around 7.18 as indicated by the red bold lines and which exactly satisfies the range provided by Hershfield (1995) curve.

Finally the PMP value of Gerhu-sirnay station according to the above method was shown in table 3-3 below:

Table 3-2: PMP of Gerhu-sirnay station (daily)

ID	Rainfall station name	Easting (x)	Northing (y)	PMP (mm)
1	Gerhu-sirnay	519243.2	1602021.11	**129.7**

3.6. Estimation of Stream Flow using SCS Method

The appropriate method to predict inflow hydrograph from point PMP for this research was Soil Conservation Service (SCS). Based on ERA (2002), the main reasons to select this method were:

i. Time of Concentration (TC) value should lies within the range of 0.1 to 10 hrs.

ii. Watershed area must greater than 50 ha and it must be non-homogeneous basin.

iii. No needs calibration

iv. The type of PMF results required: the HEC-RAS 2D model to run the dam breach analysis model needs PMF in the form of hydrograph or all coordinate points of the PMF throughout the duration.

So according to ERA (2002), the general formula of SCS method to derive runoff from PMP would be:

$$Q = \frac{(P-0.2\,S)^2}{(P+0.8\,S)} \qquad (3.8)$$

Where:

Q=Runoff in mm

P=Excess PMP value in mm

S= Retention parameter

The Retention parameter (S) can be calculated as:

$$S = \left(\frac{25400}{CN}\right) - 254 \qquad (3.9)$$

Where: CN = runoff curve number

3.6.1. Watershed parameters

The watershed parameters like: watershed area (A), longest flow path (L), time of concentration (Tc) and slope of the main water course were important parameters to determine the watershed PMF. These parameters were calculated under the Arc SWAT watershed delineator tool integrated with GIS (see the detail at appendix E, i).

3.6.2. SCS- Curve Number Determination

According to ERA (2002) for antecedent moisture conditions (AMC) in Ethiopia: it may be dry, wet or average of both. The manual indicates that, for vegetation cover poor to sparse the AMC type is wet. As indicated in figure-13 below, the watershed of this study has a sparse vegetation cover consequently the AMC type used was wet.

The Curve Number (CN) refers dimensionless catchment parameter which determines the nature of catchment runoff and retention. The value of this dimensionless parameter varies from 0 to 100. CN value of zero professionally represents that regardless of the precipitation amount the catchment has zero runoff and high retention, While CN value of 100 also conceptually to mean that the catchment is completely impermeable with zero retention (ERA, 2002).

Soil Conservation Service (SCS) has prepared a standard table format of curve number in relation to the land use land cover and hydrological soil group (SCS-USDA, 1986). HSG shows a classification of soil ranging from A to D. Hydrologic soil group A represents for sand and aggregate with high infiltration rates and D corresponds with soils swell significantly when wet and have low infiltration rate (Schulze et al, 1992). The classification shows in below table 3-4 in a summary form.

Table 3-3: HSG and corresponding surface runoff nature

Hydrologic soil group, HSG	Surface runoff nature
A	Low
B	Moderately low
C	Moderately high
D	High

Source: SCS –USDA (1986)

A catchment with sub- basins may have different land use land cover and HSG used to determine the composite curve number by weighting of the individual sub-basin curve numbers in relation to the area of sub-basins as in below formula:

$$\mathrm{CN_c} = \frac{CN_1 A_1 + CN_2\ A_2 + \cdots + CN_i A_i}{A_1 + A_2 + \cdots + A_n} \text{ --------------------------------- } (3.10)$$

Where:

CNc=composite curve number

CN_i=the curve number of sub-basin i

A_i= the area of sub-basin i

n= total number of sub basins

The first step here is preparation of land use land cover shape file and soil type shape file. Then in GIS environment intersect them and the map was generated with the results of land use and hydrologic soil group (Haile, 2018). Then the result of this process gives the map below:

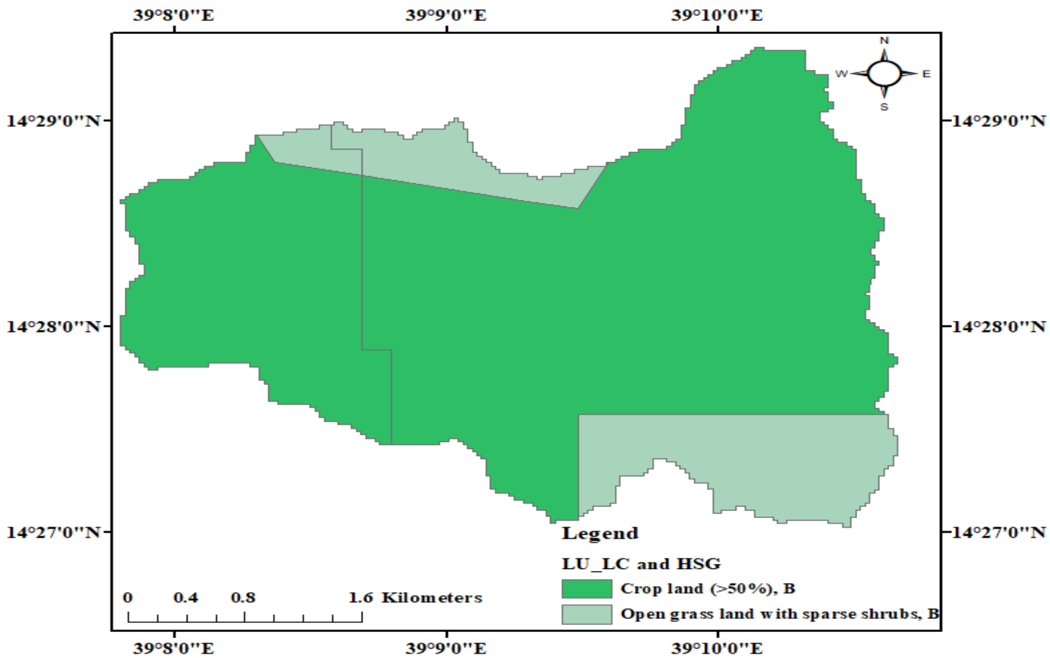


Figure 3-13: Land use type and HSG intersection result of the watershed

Equation (3.9) was accessed with all parameters from map 3-13 then by substitution the composite curve number, CNc were calculated (for detail see appendix E (ii)). Next, using equation (3.8) the retention parameter (S) would be calculated. Finally, equation (3.7) would have accessed with all parameters then its PMF value with all ordinates would be calculated (the details for SCS method of calculating PMF was provided in appendix E, iii).

3.7. The HEC-RAS 2D Modelling

From the reviewed literatures, HEC-RAS 2D Version 5.0.3 was the reasonably selected software to perform two-dimensional unsteady flood river hydraulic analysis from Gerhu-sirnay dam failure scenarios.

According to USACE (2016), HEC-RAS version 5.0.3 allows the user to carry out one-dimensional (1D) steady flow, two-dimensional (2D) unsteady flow river hydraulics by routing hydrographs through the system, combined 1D and 2D unsteady flow modeling, hydraulic calculations for cross-sections, bridge, culverts, and other hydraulic structures for unsteady flow condition, computation of one-dimensional sediment transport/ movable boundary calculations resulting from scour and deposition specially for a single flood event, water quality analysis (algae, dissolved oxygen, dissolved ammonium nitrate, and other water quality constituent elements…).

Two-dimensional unsteady flow analysis was more flexible, accurate for flood plain area delineation and estimation of inundation data, time saver in case of model calibration and no need for second model for related results and supported by many researchers than both one-dimensional and hybrid HEC-RAS models. Therefore HEC-RAS 2D unsteady flow analysis will be used to analyze Gerhu-sirnay Dam Breach Analysis and Downstream Flood Mapping.

Therefore in order to succeed the targeted objectives of this research, the modeling procedures of HEC- RAS 2D was prepared in the following figure:

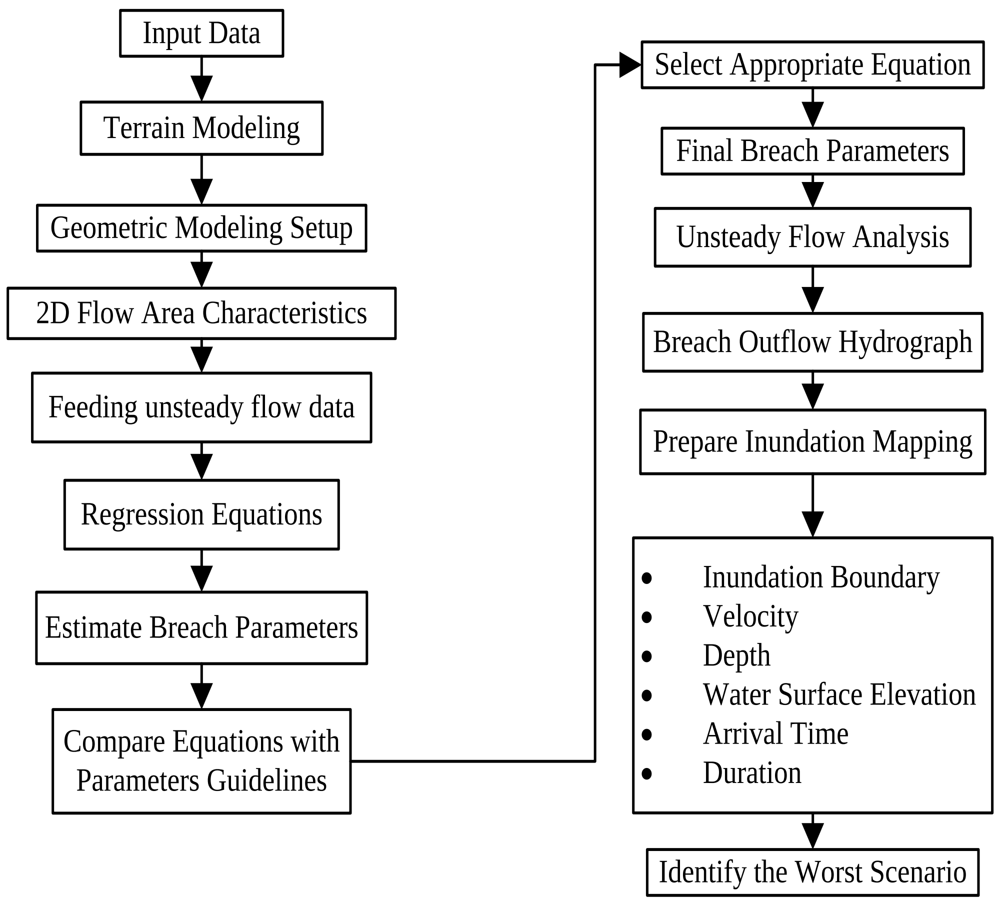


Figure 3-14: HEC-RAS 2D dam breach modelling development

The above chart shows only the basic procedural terms of HEC-RAS dam breach analysis model development of this research. So to make it clear, the details of each modelling terms was tried to discuss in the following sub titles:

3.7.1. Input Data

The main input data of HEC-RAS 2D in this dam breach modelling were:

- 30 by 30 resolution DEM

- 2D flow area land cover type and exposed inventories detail
- Dam profile or weir embankment profile in terms of station and elevation
- Constants (Weir Coefficient (Cd), Manning Roughness' (n)).
- Storage, 2D area detail and storage area/2D flow area connection
- Failure scenarios: overtopping and piping

3.7.2. Terrain Modelling

Digital terrain model (DTM) represents the digital terrain surface details of a given land by using 2D point coordinates latitude & longitude values (Christian, 2014). The DTM of this study was generated from gridded data of DEM with 30 *30 resolutions. But for the case study of this research, the terrain model can't represent the real topography of the area, that means Gerhu sirnay dam's main river is not visible. So to develop accurate flood inundation parameters the DTM should be modified until the river would be visible. Therefore to represent the actual area on the terrain model, the original terrain model was modified by using cross sections in RAS mapper and geometric editor of HEC-RAS version 5.0.4.

A. Terrain Model Modification

HEC-RAS version 5.0.4 was used to modify the un real terrain model. The data used to modify the terrain model were river centerline, bank lines and cross sections. River centerline and bank lines were collected in RAS mapper whereas the cross sections were collected from field survey and cross section interpolations.

When cross sections were collected, their interval has great accuracy on providing accurate river profile. According to Samuel (1989), the spacing of river cross section (ΔX) depends on average bank full depth (D) and average bed slope(S_0) of the river. Mathematically it was related as follows:

$$\Delta \boldsymbol{X} \leq \frac{\mathbf{0.156\ D}}{\boldsymbol{S_0}} \quad \text{---} \quad 3\text{-}11$$

Where:

ΔX= Cross section spacing (ft)

D= Average bank full depth (ft)

S_0= Average bed slope

According to the above equation the maximum possible cross section spacing of this study was 34.85 m for an average bank full depth 4.89 m of the river and 2D flow area average slope of 0.02189. However, for accuracy the cross section spacing used to interpolate the unknown cross sections of this research was 25 m.

From the RAS mapper inundation extent delineation detail with the unmodified terrain, this modeling needs cross section data from dam axis up to 11.04 Km to the downstream reach. Then from the dam axis up to only the first 17 number of cross sections were collected at 25 m interval by using actual field survey to collect more than this, the topography was risky. Therefore the rest cross sections were collected by using Google earth at 5 m by 5 m resolution but this haven't real profile. So the first 17 cross sections collected from Google earth was validated with the respective cross sections collected by field survey and used to pick out the validation factor. In below figure the validation of some cross sections are given:

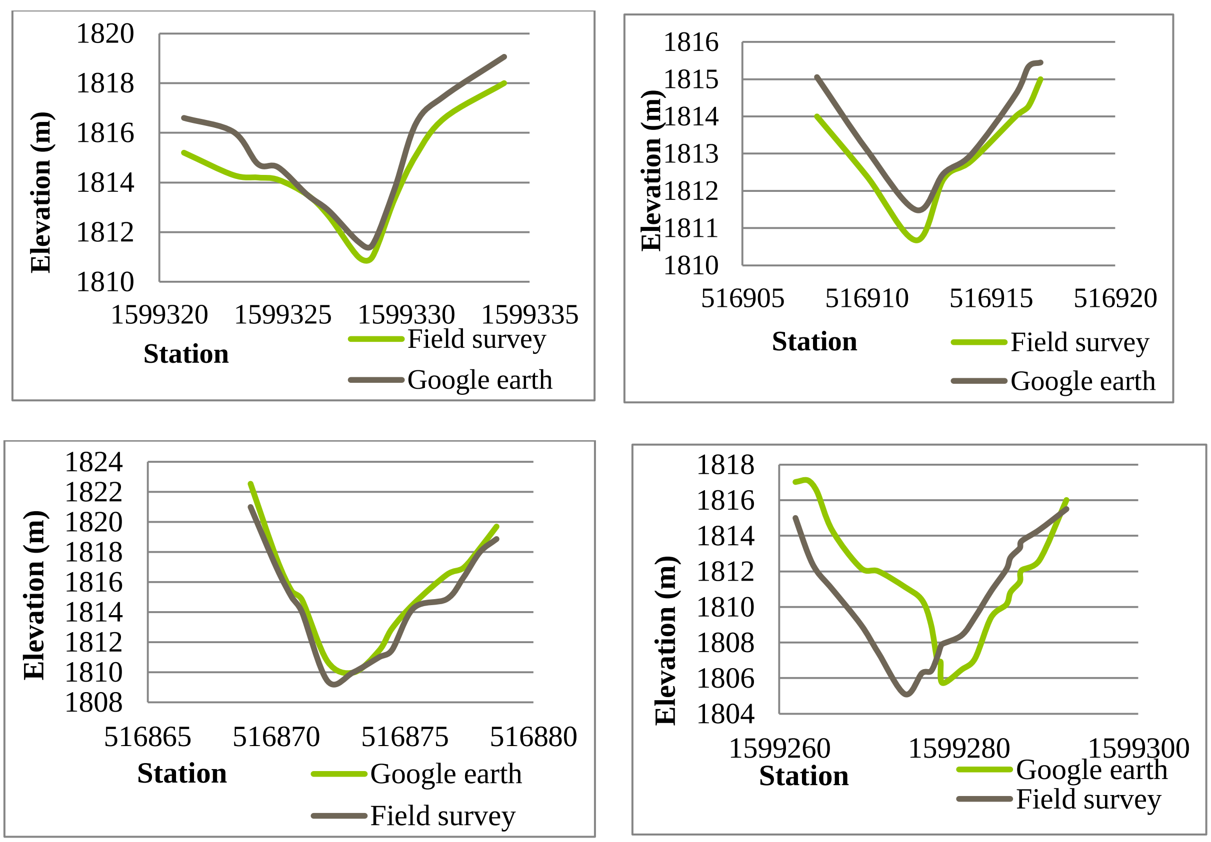


Figure 3-15: Validation of some cross sections

From the sample validated cross sections above the validation factors were fixed as vertically and horizontally. Then accordingly the vertical factor was 0.61 m below the maximum elevation and the horizontal one was also 1.73 m to the left of the cross section alignment collected from the Google earth.

The last cross section was also collected from Google earth and validated with the validation factor obtained from the validated cross sections. Then the cross section laid between 17th and the last one were interpolated at the predetermined cross section spacing. It is possible to collect in Google earth but instead interpolation among the validated cross section is better (USACE, 2016). Finally, due to cross section interpolation the cross sections between the first and last cross sections were obtained.

B. Cross Section Arrangement

The cross sections collected according to the above method was arranged in HEC-RAS geometric editor as in below figure:

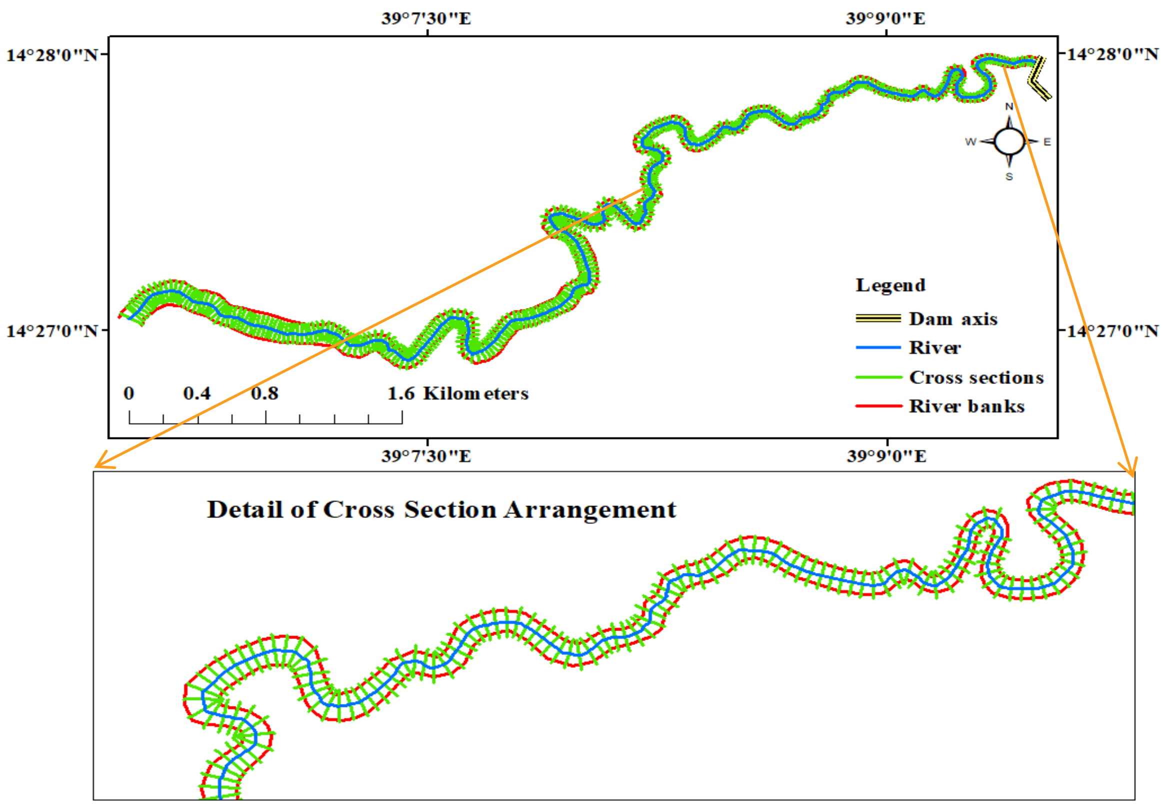


Figure 3-16: Total cross sections arranged

Finally to modify the terrain model, two terrain models would be needed in RAS mapper: One the previous un real terrain and the new one is the terrain model developed with the arranged cross section. Next in RAS mapper ,both terrain models would be merged to develop the modified river channel by subtracting the river profile according to the cross sections from the previous terrain model. Finally, the river tends to be visible and any flood inundation analysis would be carried out on the developed terrain as displayed in below figure (Haile, 2018):

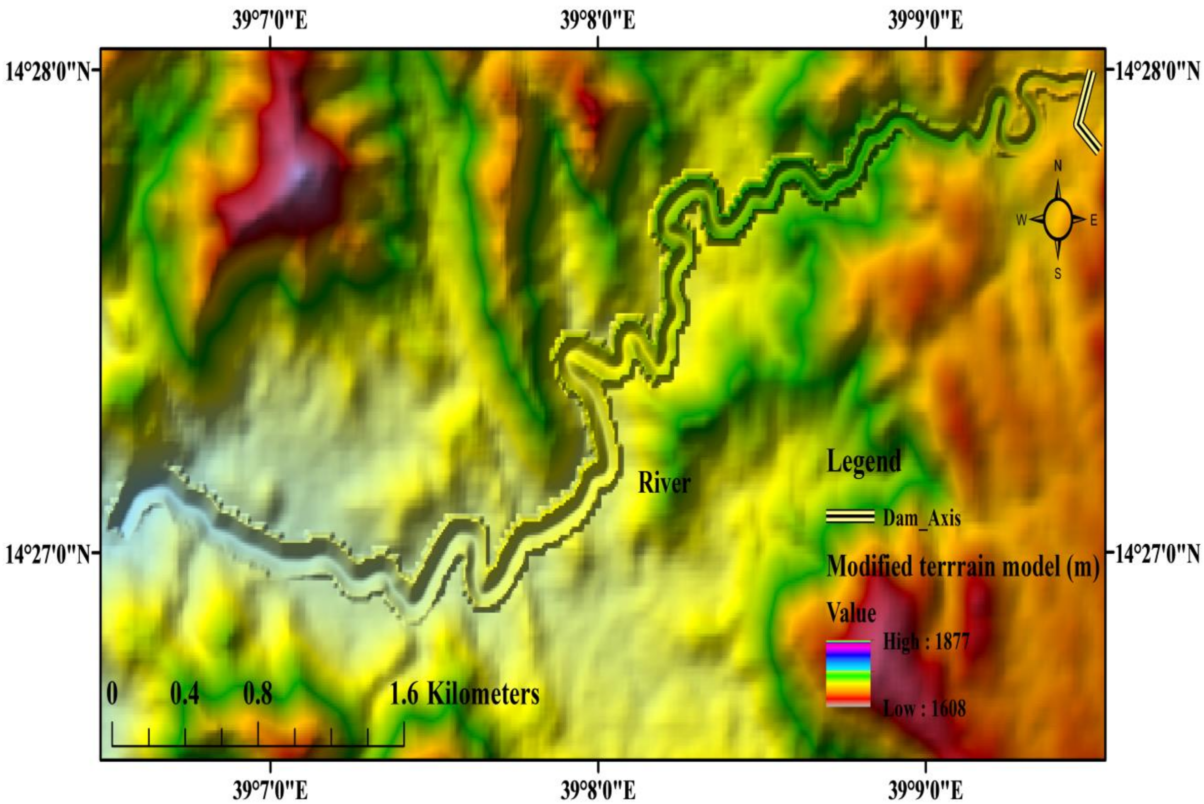


Figure 3-17: Modified terrain

3.7.3. Geometric Modelling Setup

HEC-RAS 5.0.3 has the capability to perform two dimensional unsteady flows routing with different geometric elements (USACE, 2016). The geometric data included to develop geometric modelling of this research were: dam as inline structure, storage area, 2D flow area, main river centre line and, upstream and downstream boundary conditions.

The storage area should represent the accurate area of the reservoir polygon and the dam as a connector of both reservoir and 2D flow area. The 2D flow area also used as the area of performing 2D inundation analysis results. Both upstream and downstream boundary conditions also used as longitudinal input data limits of the 2D flow area and the center line of the main river also to show the river’s real location on the flood plain.

These all geometric elements would have connected and modeled by using SA/2D Area Connector tool located in Edit menu Geometric Data of the HEC-RAS main window (USACE, 2016). Therefore to develop the model, the main geometric elements were created as below techniques:

A. **Storage Area**: First the reservoir polygon was drawn arbitrarily in its real location within the terrain model. Then it adjusted by feeding the real reservoir polygon data in place of the temporary polygonal data in the form of elevation versus volume curve obtained from (TWRB, 2016), for detail see appendix C.

B. **The Dam (SA/2D Area Connection):** First, three coordinate points were collected from the dam axis (points at both abutments of the dam and one point also at the center of the dam axis). Next, by importing those points to GIS software and create the shape file (dam axis line). Then by opening HEC-RAS 2D: RAS mapper, select map layers and right click on map layers and just click on add map data layers to display the created shape file which represent the inline structure (USACE, 2016).

C. **The Downstream 2D Flow Area Mesh:** It represents the downstream area up on which the breach out flow water inundates. It can be created in the Geometric editor of HEC-RAS by first drawing simple polygon, providing cell sizes and computing the mesh area. The real size of this area was determined by arbitrary drawing trials with the flood inundation. The parameters to be changed the given trial were inundation data. These data like depth, velocity, shear stress,…must be zero at any side of the polygon, at the first trial of drawing it may narrowed or widen to route the outflow flood (USACE, 2016).

   Therefore, the accurate and current extent of 2D flow area of this study was determined at the 17th drawing trial from 30 by 30 mesh sizes with a total of 7,477 cell numbers. This trial's accuracy has been checked by measuring the value of the inundation data (velocity and depth) were zero at any inundation polygon part from the dam axis.

D. **Upstream Boundary Condition (UBC) and Downstream Boundary Condition (DBC):** These geometric elements for modelling 2D dam breach analyses were used as input data limits (USACE, 2016). The detail of these geometric elements was given in section 3.7.5.1 below.

Finally, from the above procedures the developed geometric setup was given in the following figure:

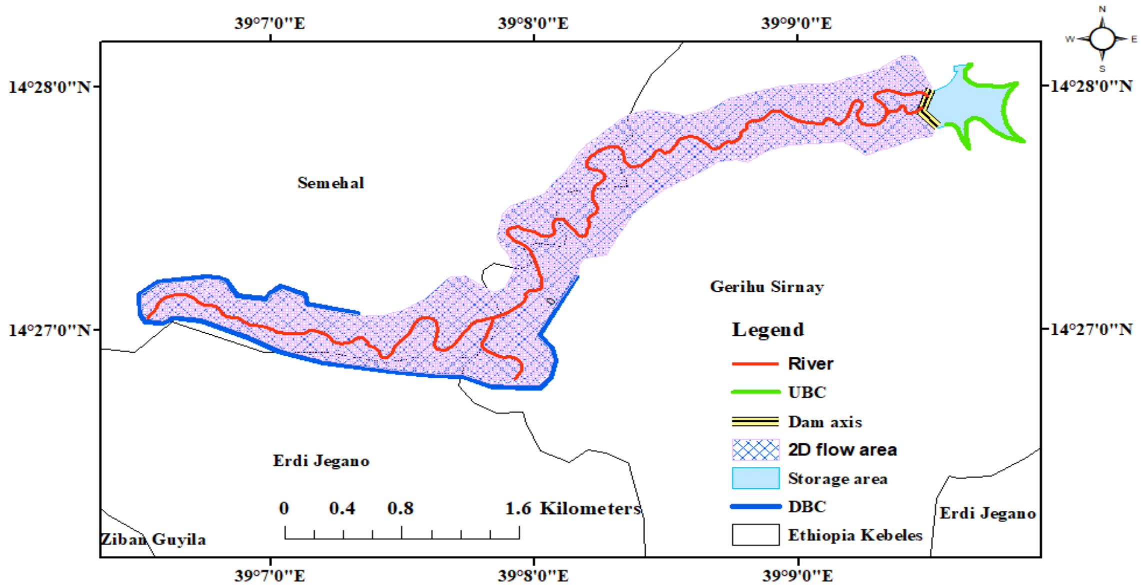


Figure 3-18: Geometric modelling

3.7.4. The 2D Flow Area Characteristics

Dam breach analysis and downstream flood mapping study needs two study areas known as upstream and downstream study areas. The upstream study area known as watershed was served as a source of input data for meteorological analysis, hydrological analysis, and HEC-RAS modelling and was delineated in figure 3-1.

The downstream study area known as 2D flow area was the area up on which the results of dam breach analysis would quantify and measured for further analysis. The size of this study area was dependent on the size and data strengths of upstream study area. It proportions directly with the magnitude these parameters have changed. Therefore measuring of some areal characteristics like topography, crop data, land use- land cover and Manning roughness coefficient has great contribution for accurate modelling and analysis of dam breach.

A. Topography

From the map below the most elevated point was at the abutments of the dam which is 1877 m a.m.s.l. The lowest point is also located at the farthest distance from the dam axis which is 1608 m a.m.s.l (MoWIE, 2019). This point was determined by the end inundation polygon or extent in the RAS mapper.

The overall areal topography of the 2D flow area was prepared in the following map:

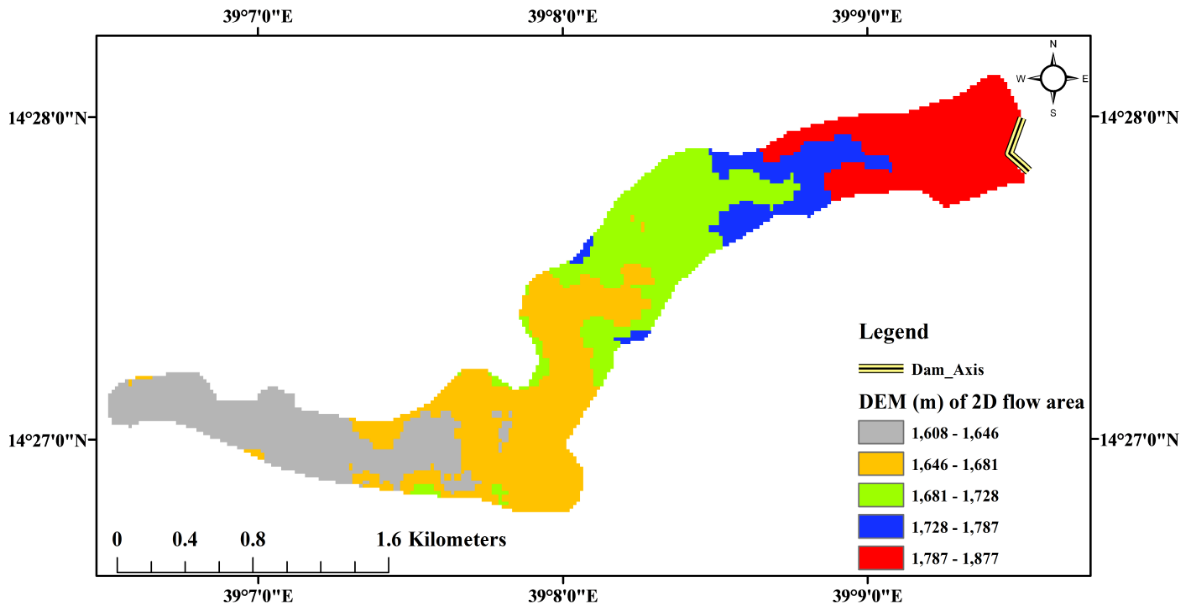


Figure 3-19: Topography of 2D flow area

B. Crop Data

According to the crop data obtained from AWAB (2019), the type of crops available in the 2D flow area of Gerhu-sirnay dam failure were Sorghum, Barley and Millet. As the report shows the percent coverage of these crops were ordered as Sorghum, Barley and Millet from high to low respectively.

C. Land use land cover and Manning roughness coefficient

Land use land cover shows the primary usage of the concerned land and its percentage cover. According to the land use land cover data obtained from MoWIE (2019), the 2D flow area of this study was covered by 30% of bare soil with open shrub land, 40% of sparse vegetation and 30% by open wood land. Its detail was provided in figure 3-18 below.

Roughness coefficient was the value that shows flow resistance nature of the flow concerned area. The main purpose of accurately estimating this value was used to determine the accuracy of flood inundation data like flood depth, flow velocity, time arrival and water surface profile. The factors that can affect the value of manning roughness were: surface roughness, vegetation cover, channel irregularity, channel alignment, obstructions, transported material load, etc....

The HEC-RAS version 5.0.3 which was used as a tool in this study has the capability to create land cover data in RAS mapper and associated it with specific geometry data set by using the following procedures (USACE, 2016):

- Open the RAS mapper
- Export 2D area as a shape file
- Open Arch map and load the shape file. Clip this shape file from the local area input feature land cover
- Finally import the clipped land cover to the RAS mapper and fill the corresponding manning's n values according to (Chow, 1959) and click on create.

Accordingly, the result of the 2D flow area land use land cover and manning's regions with their respective n value were summarised in the following figure 3-20:

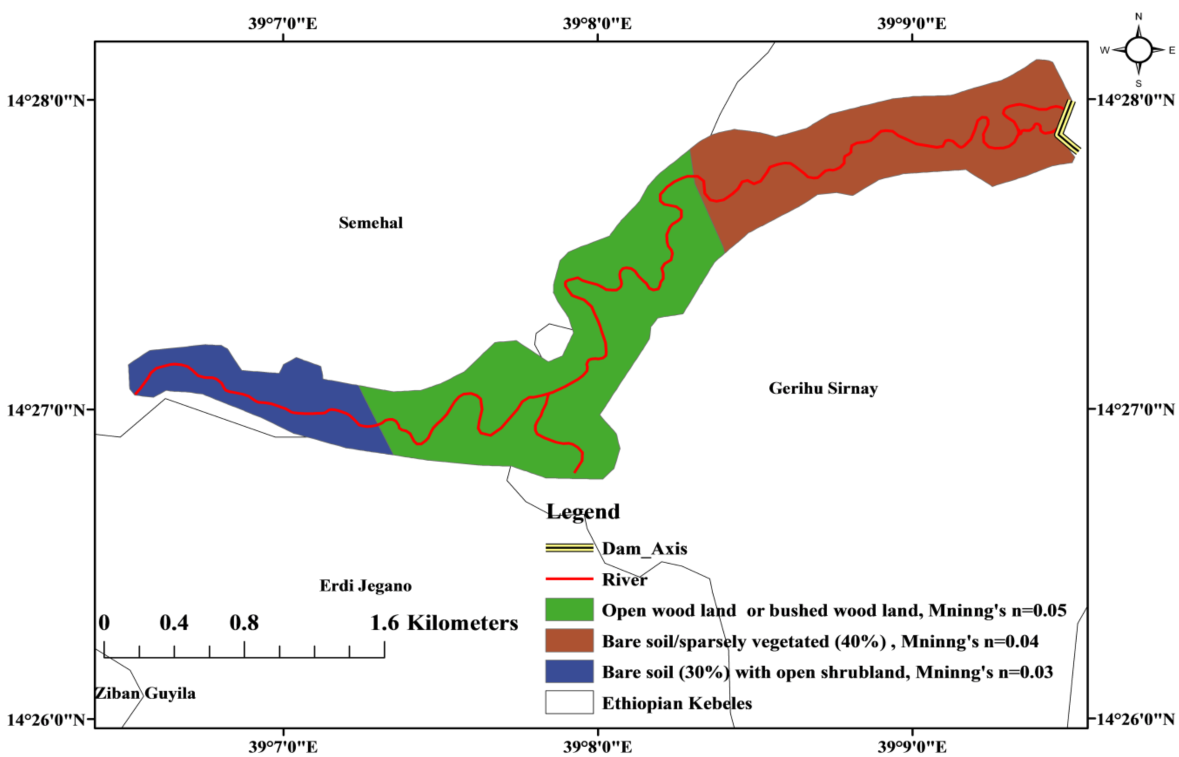


Figure 3-20: Land use land cover and manning's n regions

3.7.5. Feeding Unsteady Flow Data

3.7.5.1. Dam Profile

Dam profile for modelling of 2D unsteady flow dam breach analysis is the dam profile as a function of station and elevation of the dam body and its spillway. As per the definition of the modelling, this geometric element was used to identify the probable dimensions of the breach parameters and 2D flow area from the reservoir by setting the accurate profile of the dam from left to right or vice versa abutments of the river.

3.7.5.2. Storage Data

The storage data included in this model were: accurate storage polygon, storage elevation versus elevation curve and storage initial elevation (USACE, 2016). As per the manual, storage initial elevation refers to the maximum elevation which represents the distant point of upstream boundary condition from dam axis.

3.7.5.3. Boundary Conditions

As it was reviewed, Dam breach analysis and downstream flood mapping needs at least the downstream, upstream and 2D flow area initial boundary conditions as longitudinal analysis limits. Shortly all of them were briefed below:

A. Upstream

For both failure scenarios, the Gerhu-sirnay dam upstream boundary condition was the watershed inflow hydrograph obtained from SCS, soil conservation service. According to USACE (2016) flow hydrograph of discharge versus time was the most used upstream boundary condition type of its simplicity to feed the model hence it would be used in this analysis.

B. Downstream

The simplest downstream boundary condition for open ended reach natural channel was the normal depth of last inundated part that allows sustaining flow equilibrium condition (USACE, 2016). This depth was appeared at the moment when, the flow drag force due to gravity would numerically equal with the flow resistance force due to river bed roughness nature. Both of these forces would be equal notifies that, the water previously in flow was reached its stationary condition. Therefore this would a point upon which the 2D flow area flood inundation has been stopped.

To calculate this depth, HEC-RAS uses manning's equation to compute the depth but professionally it can be entered as friction slope or average slope of the main river produces a stage considered to be normal depth. The friction or average slope of the main river for Gerhu-sirnay dam failure 2D flow area was, 0.02189. The equation used to calculate this was given below:

$Q = \frac{1}{n} V R^{2/3} \sqrt{S}$--- (3.12)

Where:

Q =Flow rate, m/s

R=Hydraulic radius, m

V=flow velocity expressed as normal depth flow rate, y_n

S = Channel slope

n=manning's roughness coefficient

C. 2D flow area initial condition

The 2D flow area initial condition of this study unsteady flow analysis was completely dry condition. This situation has allowed only for 2D connections and accomplishes by just leaving the initial condition elevation column as blank and the software understands to start simulation as the 2D flow area was dry (USACE, 2016).

3.7.6. Estimation of Breach Parameters

USACE (2016) recommended that, many regression equations were developed to estimate dam breach parameters. But equations that have been targeted at earth and rock fill dams were: (Froehlich, 1995a), (Froehlich, 2008), (MacDonald and Langridge-Monopolis, 1984), (Von Thun and Gillette, 1990) and Xu and Zhang (2009).

Gerhu-sirnay dam is zoned earth fill with clay core, therefore breach parameters of Gerhu-sirnay dam breach analysis was estimated by the above regression equations under the HEC-RAS breach calculator tool. But Xu and Zhang (2009) equation's breach development time has over estimation nature in fact it would be not used in this study.

Since these equations were integrated with HEC-RAS model, it makes simple to use them in case of need for other tools, saving time and owning corresponding equations to estimate peak breach outflow as a function of the parameters estimated before.

The detail relations and assumptions of these mentioned above breach parameter equations were provided below (USACE, 2016):

A. Froehlich (1995a): Froehlich (1995a) used 22 zoned earthen, earthen with clay core wall (i.e., clay), and rock fill data sets to develop as set of equation to predict average breach width, side slopes, and failure time. The data used for his regression analysis had the following ranges:

Height of the dams: 3.66-92.96 meters

Volume of water at breach time: 0.0130-660 x 10^6 m^3

Froehlich (1995a)'s regression equations for average breach width and failure time were:

$B_{avg} = 0.1803 K_O {V_w}^{0.32}\ {h_b}^{0.19}$--- (3.13)

$t_f = 0.00254 {V_w}^{0.53} {h_b}^{-0.9}$--- (3.14)

Where:

B_{avg}=Average breach width (meter)

K_o=Constant (1.4 for over topping failure, 1.0 for piping)

V_w=Reservoir volume at the time of failure (cubic meter)

h_b=Height of the final breach (meter)

t_f=Breach formation time (hrs.)

Froehlich, 1995a states the average side slopes must be:

1.4H: 1V for overtopping scenario

0.9H: 1V for piping scenario

B. Froehlich (2008): In this analysis he uses 74 earthen dam case studies to develop the equations which can predict the breach parameters. Froehlich (2008) used the following data range to develop the regression analysis:

Height of the dams: 3.05-92.96 meters

Volume of water at breach time: 0.0139-660 x 10^6 m^3

In this method, breach dimensions were dependent only on the depth and volume of water stored by the dam. The average breach width (B_{avg}) and failure time (t_f) was calculated as:

$$B_{avg} = 0.27\, K_O V_w^{\,0.32}\, h_b^{\,0.04} \text{-----} \quad (3.15)$$

$$t_f = 63.2\sqrt{\frac{V_w}{g\ \ h_b^{\,2}}} \text{-----} \quad (3.16)$$

Where:

B_{avg}=Average breach width (meter)

K_O=Constant (1.3 for over topping failure, 1.0 for piping)

V_w= Reservoir volume at the time of failure (cubic meter)

h_b=Height of the final breach(meter)

t_f=Breach formation time(hrs.)

g=Acceleration due to gravity (g=9.81)

C. **MacDonald and Langridge-Monopolis (1984):** They utilized 42 dam data sets (predominantly earth fill dam, earth fill dam with clay core and rock fill dams) to develop relationship for “Breach Formation Factor”. This factor was the product of the volume of water coming out of the dam and the height of water above the dam. Then MacDonald and Langridge-Monopolis related the breach formation factor to the volume of material eroded from the dam’s embankments. The data that they used for their regression analysis had the following ranges:

Height of the dam: 4.27-92.96 meters
Breach outflow volume: 0.0037-660.0x10^6 m^3

The developed equations for volume of material eroded and breach formation time were:

- **For earth fill dam:**

$$V_{er} = 0.0261(\mathrm{V_{out}} * \mathrm{H_w})^{0.769} \text{-----} \quad (3.17)$$

$$t_f = 0.0179 V_{er}^{\,0.364} \text{-----} \quad (3.18)$$

Where:

V_{er}= Volume of material eroded from the dam (Cubic meter)

$\mathrm{V_{out}}$=Volume of water that passes through the breach (Cubic meter)

H_w=Depth of water above the bottom of the breach (meter)

t_f =Breach formation time (hours)

- **For rock fill dams:**

$V_{er} = 0.00348(V_{out} * H_w)^{0.852}$--- (3.19)

MacDonald and Langridge-Monopolis stated that the breach should be trapezoidal with side Slopes of 0.5H: 1V.

From the above equations and dam geometry the average bottom width of breach calculates as:

$B_{avg} = \frac{V_{er} - h_b^{\,2}\left(CZ_b + \frac{h_b Z_b Z_3}{3}\right)}{h_b(C + h_b Z_3/2)}$-- (3.20)

Where:

B_{avg} = Average bottom width of breach (m)

h_b = Height from top of the dam to bottom breach (m)

C = Crest width of the top of the dam (m)

Z_3=Z_1+Z_2

Z_1= Average slope (Z_1:1) of the upstream face of dam

Z_2= Average slope (Z_2:1) of the upstream face of dam

Z_b= Side slopes of the breach (Z_b:1), 0.5 for the MacDonald method

D. **Von Thun and Gillette (1990)**: Von Thun and Gillette (1990) have been used 57 failed embankment case studies as input data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984**)** to proposed empirical relations of average breach width, breach side slope and breach formation time. These equations were valid for easily erodible and erosion resistant embankment dams.

The equation for average breach width was given as below:

$B_{avg} = 2.5\, h_w + C_b$-- (3.21)

Where:

B_{avg}=Average breach width (m)

h_w=Depth of water above the bottom of the breach (m)

C_b=Coefficient, which was a function of reservoir size (for this study, C_b= 6.1)

Its detail and recommended range was given in table 3-5 below:

Table 3-4: Reservoir size and its corresponding Cb values

Reservoir size (m^3)	C_b(meters)
$<1.23*10^6$	6.1
$1.23*10^6$-$6.17*10^6$	18.3
$6.17*10^6$-$1.23*10^7$	42.7
$>1.23*10^7$	54.9

Source : Wahl (1998)

Von Thun and Gillette also have developed two sets of empirical formulas to forecast breach development time, t_f as follows: The first set of equation for breach development time was a function of water depth above the breach bottom.

$t_f = 0.02h_w + 0.25$ (Erosion resistant)--- (3.22)

$t_f = 0.015\ h_w$ (Easily erodible) -- (3.23)

Where:

t_f =breach formation time (hr.) and h_w=depth of water above the bottom of the breach (m)

The second set was also breach development time as a function of water depth above the bottom of the breach and average breach width, which was formulized as follows:

$t_f = \frac{B_{avg}}{4\ h_w}$ (Erosion resistant)--- (3.24)

$t_f = \frac{B_{avg}}{4\ h_w+61}$ (Easily erodible)--- (3.25)

Where: B_{avg}= average breach width (m)

Von Thun and Gillette (1990) noticed that, their formula was more accurate than MacDonald and Langridge-Monopolis (1984) (Wahl, 1998).

3.7.7. Comparison of Breach Parameter Equations with Guidelines

Before proceeding to compute unsteady flow analysis, selecting the best breach parameter calculator equations was carried out. To do this, first the breach parameters of these equations was calculated in the HEC-RAS breach parameter calculator tool by selecting the breach scenario weather overtopping or piping. Next, to undertake the unsteady flow analysis the formulas' result should standardize, validate or compared with the published guidelines of breach parameter characteristic range serves as upper and lower bound (table 2-5). These guidelines were dependent on the specific geometry of the dam to be analyses like dam height (h_d) and side slope (S). As a result, this made it better than other methods like envelope curve.

As Adnan (2017) and USACE (2016) recommendations, these guidelines were prepared for small dams and hence Gerhu-sirnay dam was small dam. Also as it referred from many other papers it was time saver than any other methods like envelope curve. Because envelope curve judges the accuracy of the regression equations after all equations' breach parameters and outflow discharge were calculated and the guide line judges at the first parameterization stage.

As it reviews many researchers were used envelope curve, but the fact is that: envelope curve was developed only for fourteen failed dams and due to its data limitation it has inaccuracy especially in its outflow upper bound and also it not accounts the specific property of the dam to be analyzed. Therefore due construction material, dam's geometry, air conditions, geologic nature… and many other differences the outflow results constituted for envelope curve may not represent the outflow results of the dam under analysis.

Therefore the guideline method of breach parameter characteristic range has more accuracy and was applied in this study. To carry out this, first the bottom and top breach width, breach side slope and breach formation time of each equation would be estimated under HEC-RAS breach calculator tool. Next, average breach width (B_{avg}) will be calculated according to each regression equations' relation externally or out of the model. Accordingly, the prepared parameters which were average breach width, breach formation time and side slope would be checked if they found under the breach parameter characteristic range guidelines (table 2-5).

Finally the regression equation in which its parameters were within the guideline's rage will be the most valid and appropriate equation to estimate the breach parameters of Gerhu-sirnay dam.

3.7.8. Performing Unsteady Flow Analysis

Gerhu-sirnay dam breach unsteady flow analysis has performed after the following model's identity assessment was carried out (USACE, 2016):

A. Model Accuracy

Model accuracy represents the degree closeness of numerical solutions to the real solution. Model accuracy in HEC-RAS 2D depends on: any assumptions and limitations of the model, accuracy of geometric data (manning's n values fixed in figure 3-19), accuracy of flow data and boundary conditions and numerical accuracy of the solutions.

Therefore to develop accurate dam breach analysis modelling of this research, all the listed items having probability to cause model inaccuracy and must first fixed and maintained to have accurate model.

B. Model Sensitivity

The main problem for developing 2D unsteady flow analysis was model unstability. Performing sensitivity analysis was the best method of understanding model's accuracy and stability. This may occur due to unwise use of: model computational time step, weir and piping flow factors (see table 2-8). In this study's unsteady flow model development these points were adjusted according to the specification of the case study to develop stable model.

During unsteady flow analysis, HEC-RAS 2D needs the selected breach parameter regression equation to expand the breach progress automatically in a linear relation with time. The starting and ending of simulation time window should also filled respectively with the current date and hour regarding to the time of inflow hydrograph. Next to this the computation setting like computation interval, mapping output interval, hydrograph output interval and detailed output interval was feed and at the end compute the plan (USACE, 2016). Next the model starts to run the unsteady flow analysis step by step as geometry pre-processor and postprocessor, unsteady flow simulation and flood plain mapping.

C. Model Stability

The developed dam breach analysis model of Gerhu-sirnay dam was checked its stability before the fixing the results of unsteady flow analysis for next processes. Its stability was checked in terms of water surface elevation (WSE) of the 2D flow area, initial boundary condition, date time interval, extrapolation beyond storage volume versus elevation curve, simulation time window , maximum number of iterations and computation time interval setting and initial conditions of the 2D flow area in terms of elevation. Therefore as checked the model was safe and stable regarding the listed stability terms above

D. Unsteady Flow Calculations

The Gerhu-sirnay dam breach unsteady flow calculation was carried out to estimate geometry processor, unsteady simulation, post processor and flood plain mapping. The breach out flow hydrograph estimation was its earlier result and has carried out by the parametric hydraulic model's regression equations. This means the regression equations used to estimate breach parameters in HEC-RAS breach calculator tool have corresponding peak outflow discharge estimator regression equation integrated with the model. Therefore, once the appropriate equation for breach parameter estimation has been selected, then the HEC-RAS 2D model automatically estimates the breach outflow hydrograph as a function of the breach parameters when performing the unsteady flow analysis. In section 3.7.6 above Von Thun and Gillette (1990) has been selected to calculate breach parameters then know the outflow discharge would be also estimated by Von Thun and Gillette (1990) regression equation. Von Thun and Gillette (1990) regression equation of peak discharge was a function of average bed width, breach formation time, material erosion resistivity and side slope of horizontal component of breach (FEMA, 2013).

Therefore, Gerhu-Sirnay dam unsteady flow analysis simulation has been computed for both overtopping and piping scenarios according to the above method. This helps to create a professional view on how the reservoir drains during the instant of breach for the selected scenarios.

3.7.9. Viewing Breach Outflow Hydrograph

To extract the estimated breach outflow from the above method's result, the following procedures should follow:

- Opened the HEC-RAS main window
- Open View
- Open Stage and flow hydrograph
- Go to type and select the storage area connections
- Finally breach out flow hydrograph was displayed in tabular and stage flow forms

3.7.10. Preparation of Downstream Flood Inundation Mapping

After the HEC-RAS model has run in between both upstream and downstream boundaries, the 2D flow area inundation would be accomplished as the function of Gerhu-sirnay dam failure in unsteady flow condition. In order to prepare the downstream flood inundation maps, both HEC-RAS and Arc Map would use interchangeably (FEMA, 2013).

After the breach simulation was accomplished for both breach scenarios in HEC-RAS, the possible results for dam breach analysis and downstream flood mapping was stored under GIS tool of HEC-RAS which known as RAS Mapper. Then according to the data requirement to address the objectives, the targeted results would export to the prepared directory.

For this study the required results from HEC-RAS were: inundation extent map, inundation depth detail, water surface elevation, detail of flow velocity, flood arrival time and inundation duration. Next to this, import them to Arc Map and prepare each hydraulic inundation result in map format as much as readable. Finally by exporting the map as shape file it can be placed to the prepared document directory.

3.7.11. Identification of the Worst Scenario

Identifying of Gerhu-sirnay dam breach worst scenario was aimed to estimate the flood impact, develop emergency action plan and prepare the flood damage recovery facilities in order to minimize the damage from the worst scenario. As it was checked in HEC-RAS model Gerhu-sirnay dam would be subjected to both overtopping and piping breach scenarios.

From the prepared inundation maps in the previous objective, the required inundation data like peak outflow, velocity, inundation depth, and the others have been quantified. Then the breach scenario with the higher inundation data: inundation extent, inundation depth, flow velocities, inundation duration and faster flood arrival time would cause higher flood impact or damage. Therefore estimation of flood damage in this study was only for the worst scenario. According to this study the identified worst scenario can be called as the source of input data to develop emergency action plan.

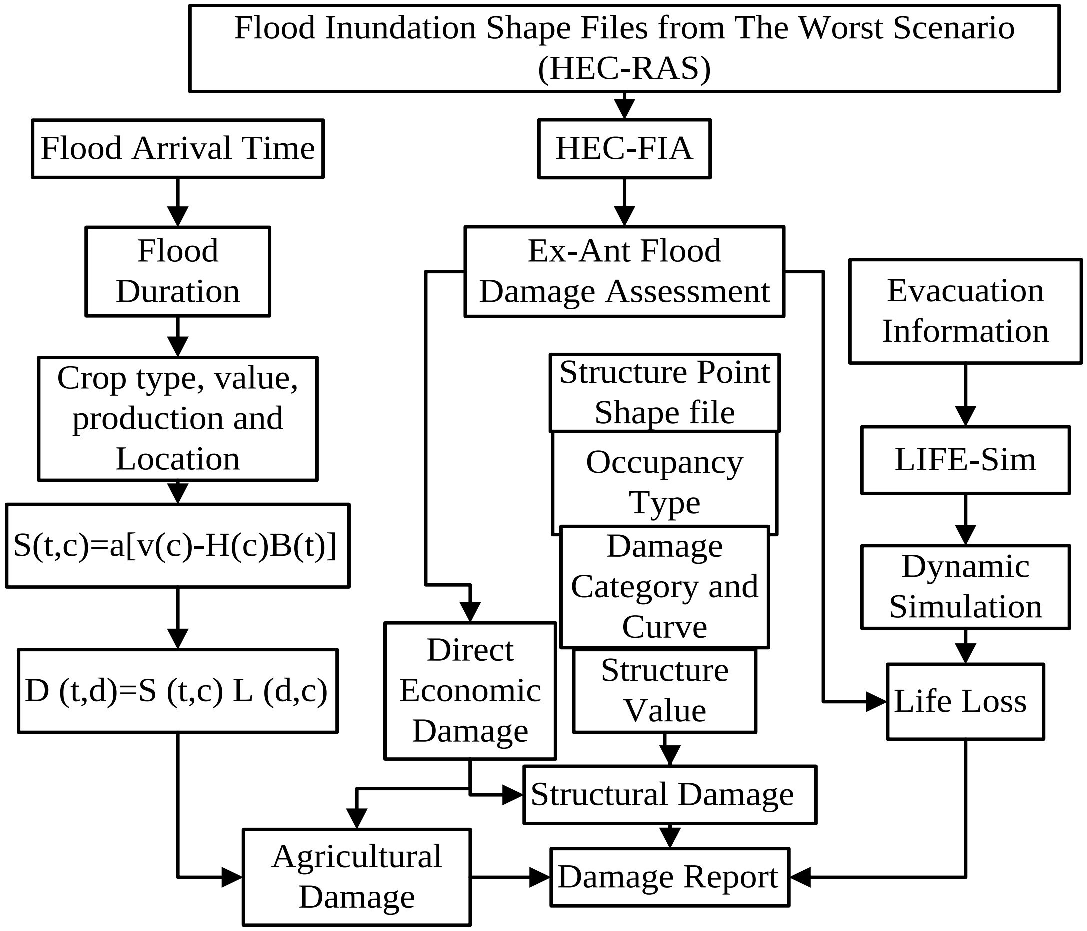
3.8. Assessment of Downstream Flood Damage Using HEC-FIA

Lehman and Light (2016) said that, HEC-FIA has the capability of estimating many flood consequences during a single flood event. But according to the objective of this study only ex-ant direct economic damage and life loss flood damage will be assessed here.

According to USACE (2016), Hydrologic Engineering Center Flood Impact Analysis (HEC-FIA) was an integrated system of software designed to enable users in calculating flood damage and benefit accomplishment attribute to flood control projects. Development of LIFE-Sim has been sponsored by the United States Army corps of Engineers and ANCOLD to calculate dynamic life loss in combination with HEC-FIA. HEC-FIA has flexible properties with much hydraulic software and GIS for modeling and assessing the level of damage in flooding situations.

This model can estimates the damage consequence or flood damage in terms of both direct and indirect economic damages and life loss. In this research the only tasks were estimation of both direct economic damage and life loss. Direct economic damage includes damage of structure and agricultural practices around the flood affected area and life loss includes assessment of the number of lives they would die on the 2D flood area.

The overall modeling of HEC-FIA 2.2 to calculate the mentioned above flood damages were tried briefly in the following flow chart:



Source: Buchanan (2016)

Figure 3-21: The HEC-FIA Modeling

3.8.1. Ex-Ant Direct Economic Damage

To identify Gerhu-sirnay dam probable flood damage potential, ex-ant direct economic damage involving both agricultural concerns and structural (treatment plant house with costly treatment machines and other content) will be carried out. The tool used to estimate the damage potential was HEC-FIA version 2.2. To carry out this, the input data and procedures followed were given in below section:

3.8.1.1. Agricultural

To estimate agricultural flood damage from the dam failure monetarily, HEC-FIA needs the following input data.

Input data:

- Terrain grid
- Total inundated agricultural area
- Crop geospatial location up on the 2D flow area
- Crop type with its coverage
- Area-production capacity of each crop type
- Crop values, harvest cost
- Inundation data grids from HEC-RAS: flood arrival time, flood depth, and its duration.

From AWAB (2019), the crop types and their coverage for the maximum 2D flood area was Sorghum (67.46%), Barley (15.29%) and the rest 17.25% was also Millet. According to the EASSSA (2017/2018) Report, the standard for area- production of major crops with its yield was provided. For Sorghum 28.52 quintal per hectare, for Barley it is 17.88 quintal per hectare and for Millet that is 22.66 quintal per hectare. Based on that norm the areal production in quintal and yield of all crops included in this study inundation area were calculated to prepare the total crop value as function of area and crop type (a*v(c)).

Finally, HEC-FIA model uses the following equation to estimate agricultural direct economic seasonal damage (Nafari et al, 2015):

$$\mathrm{S}(\mathrm{t},\mathrm{c}) = \left(\mathrm{a} * \left(\mathrm{v}(\mathrm{c}) - \mathrm{H}(\mathrm{c})\right)\right) * \mathrm{B}(\mathrm{t}) \text{--} (3.27)$$

Where: S= seasonally based value as a function of date and crop type

t= date

a= the area of grid cell in acre and c= Crop type

v= the value as a function of crop type

H= the harvest cost as a function of crop type

B= the percentage of total crop value that is available to be flooded

$$D(t,d) = S(t,c) * L(d,c) \text{--} (3.28)$$

Where: D=damage to the crop as a function of date and inundation duration

d =duration of flooding

L= loss of crop as a function of duration and crop type

3.8.1.2. Structural

Structural flood impact estimation refers to the total structural damage calculation in Birr. According to TWRB (2016), the overall monetary cost of the water treatment plant (structures and treatment machines) was 54, 000,000 Birr. Therefore the structural impact assessment was based on this monetary specification and other additional future improvements mentioned in the design document.

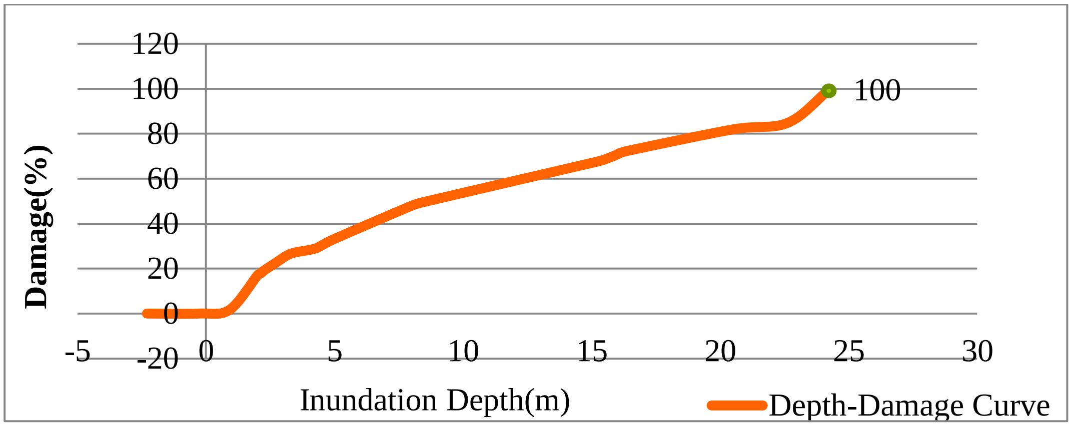
Input data:

- Individual structure's point shape file
- Occupancy type
- Damage category
- Structure value: content value (cars, other machines…. within the structure), structure cost, population at risk under and above 65 years separately, location, classification, address and attribute of individual structures.
- Damage curve: The ex-ant flood damage assessment purpose needs a standard approach damage function referred as stage-damage curve based on the type of structure and its level of damage. Damage curve refers to the function that represents the relationship between inundation data like flood depth and velocity with its damage availability measured in Birr.

The damage curve used in this study was depth-damage function which realizes the damage in terms of percentage and Birr. The depth used to develop this curve was the depth difference between each structure's foundation height and corresponding inundation depth, for more detail see appendix H (i).

According to the concept of this curve, the damage value would vary from 0% at 0 m and below inundation depth difference up to 100% at the maximum inundation depth difference.

The depth-damage curve used in this study was given below:



Source: HEC-FIA User's manual (2012)

Figure 3-22: Depth damage curve (DDC)

3.8.2. Assessment of Life Loss

The LIFE-Sim tool integrated with FIA software was a dynamic simulation system to prepare the potential of life loss (Aboelata et al., 2004). This tool prepared the report of life loss potential by using warning diffusion theorem. This principle needs to consider population activity distribution of the working time. So as it was checked, Gerhu-sirnay dam water treatment plant has equally distributed two times-day-scenario (day and night) working time.

3.8.2.1. Input data

- Terrain grid
- Location of impact area
- Population at risk (PAR)
- Inundation data from HEC-RAS
- Location of safe zone
- Warning-mobilization model
- Warning alert type

Based on these concepts the currently available population at risk (PAR) of the impact area collected from TWWCE (2019) was about 70 stakeholders for both working time (appendix H, ii).

3.8.2.2. Sensitivity Analysis of Dynamic Simulation

In order to obtain accurate life loss report, the dynamic simulation of HEC-FIA model needs sensitivity analysis for the parameters those can change the report or death result rapidly. As the recommendations of Aboelata et al. (2004), the accuracy of HEC-FIA dynamic simulation model has dependent on its analysis for sensitive.

The sensitivity analysis for dynamic simulation of this study would carry out for the pillar parameters which were effective warning duration time, warning alert type and location of safe zone. As the value of these parameters would vary, the life loss result from the dynamic simulation would also vary rapidly due to that reason all these parameters were selected to be tested for their accuracy as in below sections.

A. Effective Warning Duration

Effective warning duration is the time taken by the populations at risk (PAR) to exposure and flood damage. During this time, the PAR takes all possible measures like transportation, lifting and evacuation to save their life (Australian Guideline for Flood Disaster Resilience, 2017). The life loss estimation of Gerhu-sirnay dam under the worst scenario depends on the development of effective warning duration time. The main factors that determines effective warning duration time were breach initiation time, breach formation time up to the instant of peak outflow and the flood arrival time of the peak outflow discharge to the impact area (Aboelata et al., 2004). These parameters were discussed in below paragraphs:

Warning initiation time is the time at which warning was first released to the population at risk relative to the dam failure time (Aboelata et al., 2004). In this research the worst scenario used to estimate life loss was overtopping, therefore as a function of this scenario the warning initiation time would be started by over embankment water flow due to the heaviest rainfall. From this point of time up to the upstream face of the dam will start to erode is called breach initiation duration. Breach formation time is the time duration from the end of breach initiation duration up to the instant of peak breach outflow would occurred.

Finally the peak outflow arrival time was the time duration extends from the end of breach formation time up to the peak breach outflow would reached at the probable area of population at risk (PAR) would occurred.

Then according to these needed imput data for LIFE-Sim dynamic model, those parameters were calculated in HEC-RAS version 5.0.3 hydraulic model and feed to the dynamic model. Then accordingly the calculated parameters were 39.2 minutes, 20 minutes and 31 minutes respectively. This notifies that, starting from the initiation of breach time up to the inundation or arriving of maximum breach outflow at the impact area there would a duration of 90.2 minutes.

Due to the LIFE-Sim dynamic analysis capability it uses emergency warning diffusion theorem to calculate the percentage of population warned and mobilized (Sorensen, 1991). This theorem uses the available warning time duration as 30 minutes duration for one-warning diffusion round (USACE, 2012).

The General formula of the theorem is:

$$\frac{\mathrm{d_n}}{\mathrm{d_t}} = k[a_1 * a_1 f(N-n)] + (1-k)[a_2 n(N-n)] \text{-----------------------------------} \quad (3.29)$$

Where:

k= the portion of the population alerted by the warning system who understand the warning message.

a_1= the alerting parameter that defines the efficiency of the broadcast process in reducing uncertainty of a given warning system.

$a_1 f$= multiplier for warning system effectiveness adjusted for the location and activities of individuals throughout the day.

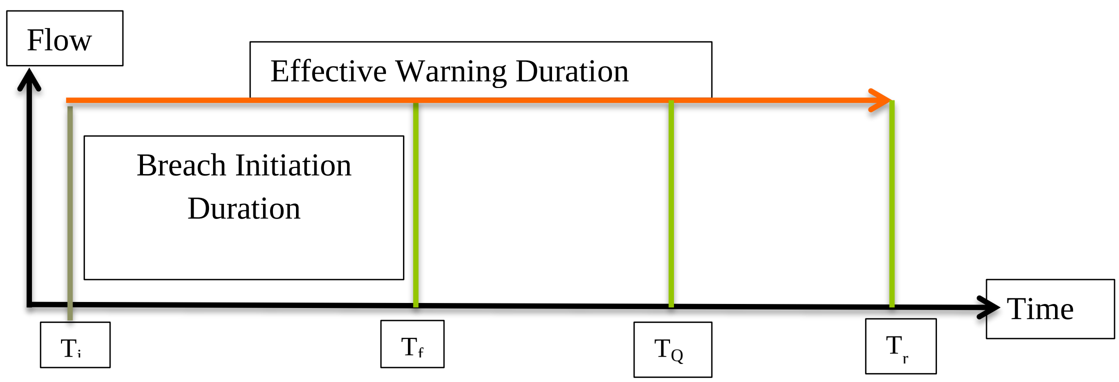
N= the limit of percentage of population warned within 30 minutes of initial warning time.

n= the value of N in the preceding time step.

$(1-k)$= the portion of the population that cannot be fully alerted or do not understand the warning (where, 1 represents the entire population).

a_2= multiplier for the effectiveness of the warning system in reducing uncertainty through secondary avenues or secondary chance like word of mouth.

Therefore inorder to save the life of the population at risk using the first 90 minutes for warning diffusion duration would optionless decision and used in this study. According to the model's warning diffusion performance this duration was applied as 30 minutes warning duration considering as first round of warnig term. Then the next 30 minutes duration also used as second round warning term for those being not warned in the first round. The last 30 minutes duration would then used for the third chance warning term for those not warned and mobilized in the previous warning rounds. Finally the model considered that, the people not warned upto the end of last warning chance term were couldn't mobilize and consequently subjected to death. The warning model was designed as below figure 3-20:



Source: Aboelata et al. (2004)

Figure 3-23: Effective warning duration time modelling

Where:

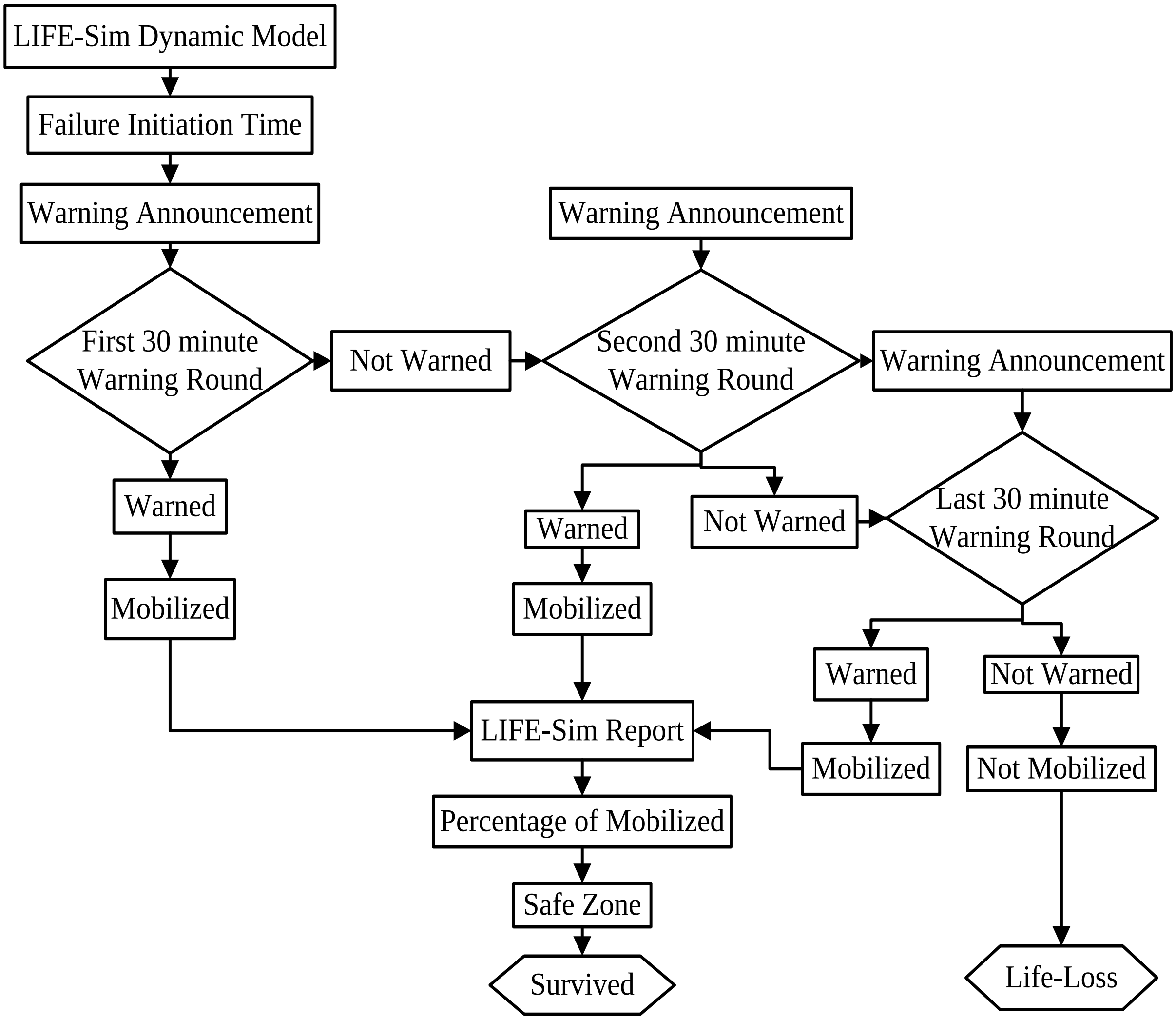
T_i = breach initiation time

T_f = initial failure time

T_Q= breach formation time up to the instant of peak outflow

T_r= flood arrival time of the peak outflow

The overall life loss Assessment steps in this study were summarized in flowchart below:



Source: USACE (2012)

Figure 3-24: The LIFE-Sim Dynamic Modeling Approach

B. Warning Alert Type

The HEC-FIA model supports more than six warning alerting types then by changing all these types the death report would have observed (USACE, 2012). But finally the best alerting type should be taken based on the awareness of the populations at risk. Therefore, as per Gerhu-sirnay dam stakeholders' awareness, the Tone-Alert Radios alerting system would be applied.

C. Location of Safe Zone

During warning time the population at risk were mobilized through the shortest and suitable road to save their life from the probable hazard. The probability of life loss in Gerhu-sirnay dam breach analysis would in the water treatment plant. So in this dynamic modelling analysis, the location of safe zone is to the left of the treatment plant at an average distance of 254 m. For the purpose of high mobility rate, the mobilization road would straight and short pass through the main gate of the treatment plat camp. Therefore according to the warning diffusion theorem, the stakeholders who would reach at the safe zone with in the given warning duration would save their life.

The shortest and safest zone prepared for the mobilized stakeholders was provided in the following map and marked by a green colour.

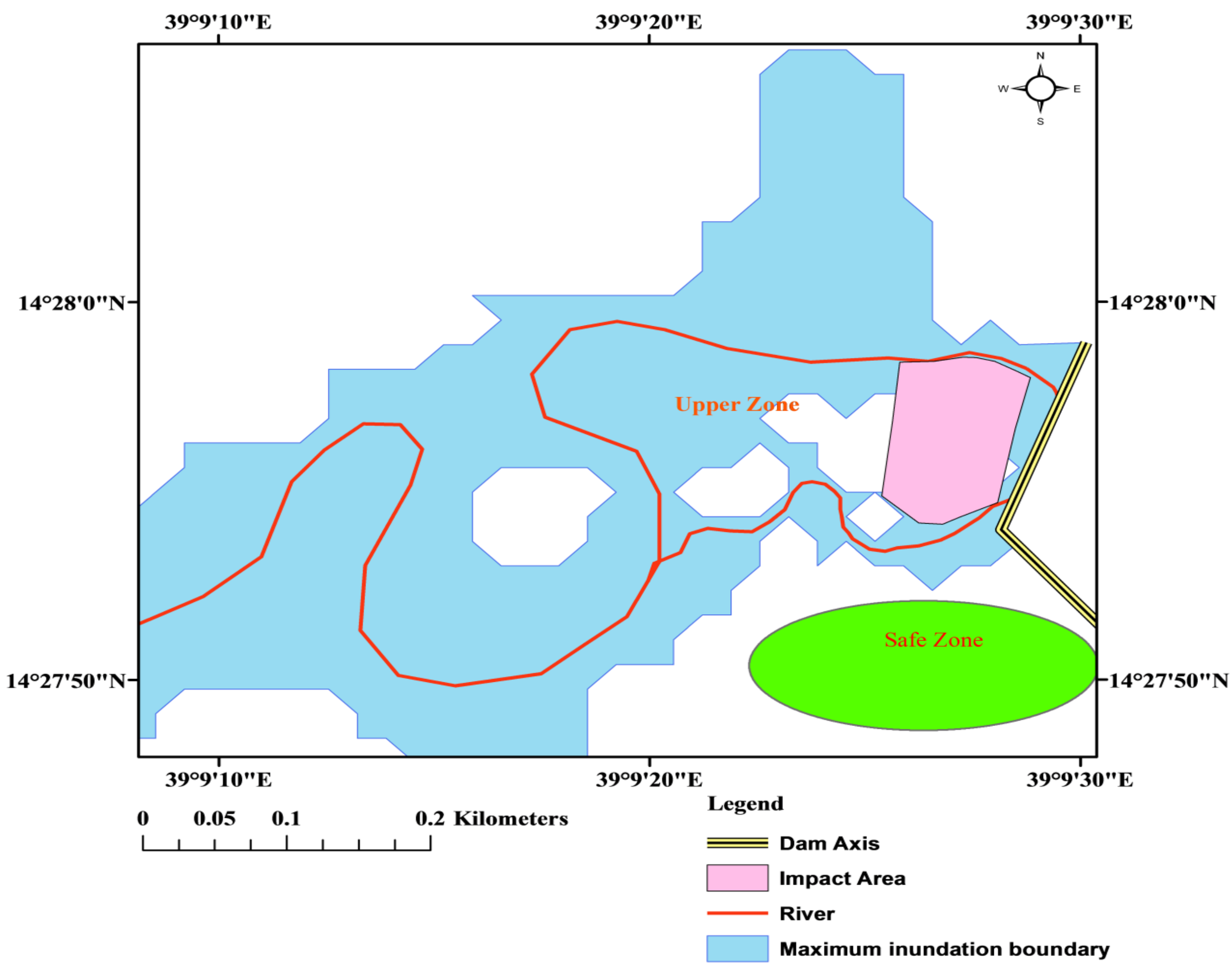


Figure 3-25: Location of safe zone

3.9. Emergency Action Plan (EAP)

The aim of Emergency Action Plan (EAP) as the last objective of this study would to minimize the life and property damage by the probable dam breach hazard as a function of Gerhu-sirnay dam. This objective was specifically dependent on the results like delineated hazard area, hazard potential category and local community dam breach hazard awareness.

To develop emergency action plan the methods included were: first the maximum flood extent was delineated by using HEC-RAS 2D and the flood properties also quantified. After that the Gerhu-sirnay dam flood hazard potential would specify. According to the risk severity, the EAP would prepare as much as possibly practiced by the local community. Finally awareness creation of local community and responsible risk organizations would build according to the validity of EAP and its operation procedures.

Procedures to prepare emergency action plan of Gerhu-sirnay dam failure hazard was summarized in the following figure:

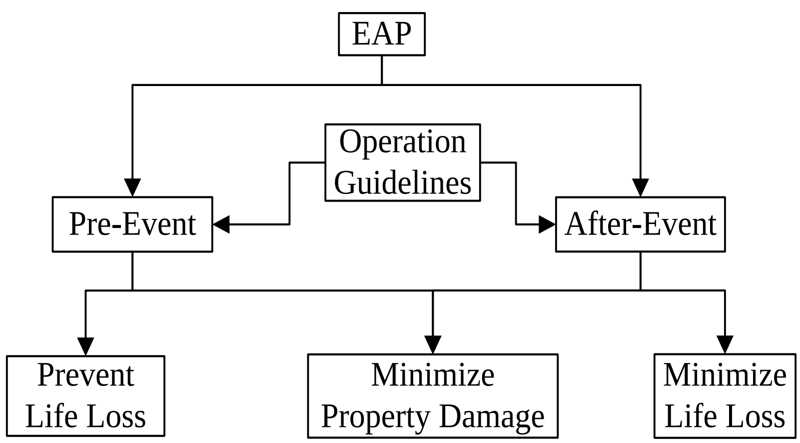


Figure 3-26: Operation procedures of EAP

4. RESULTS AND DISCUSSIONS

4.1. Inflow Hydrograph

The peak discharge of composite inflow hydrograph calculated from SCS method was, 329.1 m^3/s as labelled in figure 4-1 below. The spillway capacity of Gerhu-sirnay dam in the design document was, 105.5 m^3/s. Since the calculated PMF in this study shows greater flood magnitude, the dam has the probability to be faced by overtopping failure. Therefore for this study the input data as upstream boundary condition of unsteady 2D flow analysis was 329.1 m^3/s.

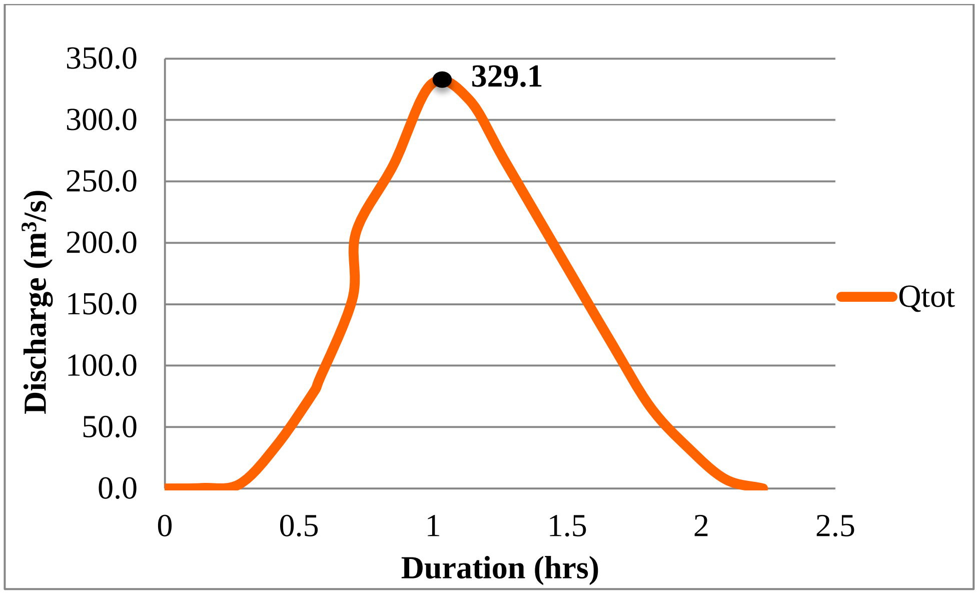


Figure 4-1: Reservoir Inflow hydrograph of the catchment area

The above hydrograph shows runoff magnitude change of Gerhu-sirnay dam watershed up on a given rainfall duration, 24 hrs. In the rising limb; the magnitude of runoff was increasing continuously, this shows the water retention capacity of watershed tends to be low. The maximum runoff was occurred at 0.99 hrs, this shows the soil of watershed was fully saturated and every rain droplet changes to runoff. The next rain fall duration has shown that, the intensity of rainfall decreases continuously and finally has completely stopped and runoff also become zero.

4.2. Breach Parameters

4.2.1. Overtopping Scenario

For the first case overtopping scenario was selected and the breach parameters calculation gives the following numerical values:

Table 4-1: Overtopping breach parameter for four methods in HEC-RAS version 5.0.3

Methods	Bottom Width, m	Side Slope (H:V)	Breach Top Width, m	Breach Formation Time (hrs)
MacDonald and Langridge-Monopolis(1984)	1	0.5	32.76	0.56
Froehlich(1995 a)	6	1.4	94.93	0.23
Froehlich(2008)	10	1	73.5	0.24
Von Thun & Gillete(1990)	54	0.5	85.75	0.73

Initially, the breach parameters of Gerhu-sirnay dam were calculated by using the above four regression equations. From the comparison of these equations' parameters result with the guidelines breach parameters range, the only Von Thun & Gillete (1990)'s breach parameter result was with in the guideline's range. Therefore, the best regression equation for this analysis's breach parameter estimation overtopping scenario was Von Thun & Gillete (1990). From this most valid equation, the extracted parameters for their validity were: breach bottom and top width as 54 m and 85.75 m respectively, breach side slope (H: V) as 0.5 and breach formation time 0.73 hrs.

Due to the existence of optional formula for predicting breach formation time Von Thun & Gillete (1990) mentioned that, their formula was better than MacDonald and Langridge-Monopolis (1984). This notifies that, the comparison was remained only for Froehlich (1995 a), Froehlich (2008) and Von Thun & Gillete (1990). But still as it was predicted initially, Von Thun & Gillete (1990) has become the most valid one.

From comparison of this result with Jiregna (2016) and Adnan (2017) under similar methodology and failure scenario, Von Thun & Gillete (1990) was being the accurate equation to estimate breach parameter of this research was consistent and advisable with dam geometry's proportionality.

Finally results of this equation for Gerhu-sirnay dam overtopping failure scenario embankment profile were provided in Figure 4-2 below:

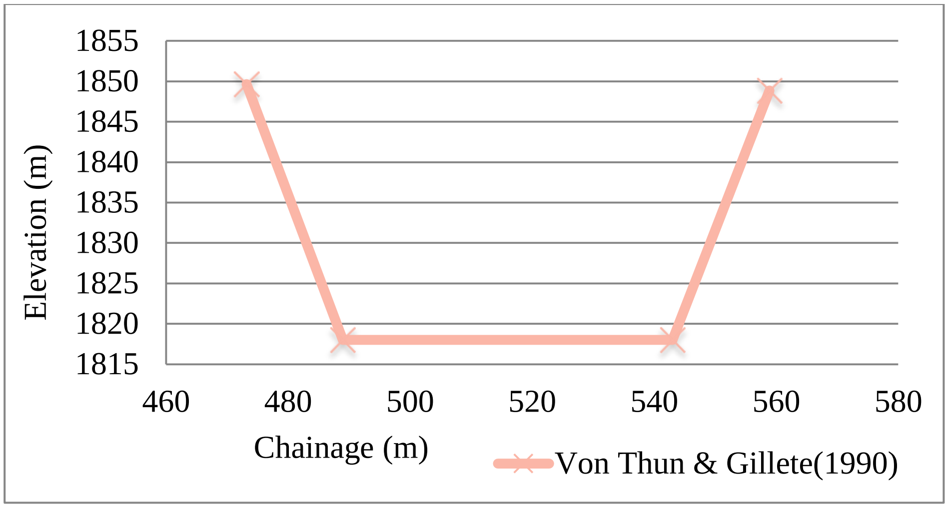


Figure 4-2: Breach parameter plot for overtopping scenario

4.2.2. Piping Scenario

Results of piping breach parameter calculation for the four regression equations were provided as follows:

Table 4-2: Piping breach parameter for the four regression equations

Methods	Breach bottom Width, m	Slope (H:V)	Breach top Width, m	Breach Formation Time (hrs.)
MacDonald and Langridge-Monopolis (1984)	1	0.5	26.75	0.43
Froehlich (1995 b)	7	0.9	53.35	0.24
Froehlich (2008)	10	0.7	46.05	0.26
Von Thun & Gillete (1990)	32	0.5	57.75	0.53

As overtopping scenario above piping case also used breach range criteria in order to select the best method. Then after comparison of this scenario's breach parameter calculator equations only Von Thun & Gillete (1990) was accepted. Due to accuracy of this equation, result from the comparison under piping scenario also the same as with that of overtopping scenario.

Therefore, the parameters profile visualization plot of this formula was as in figure 4-3 below:

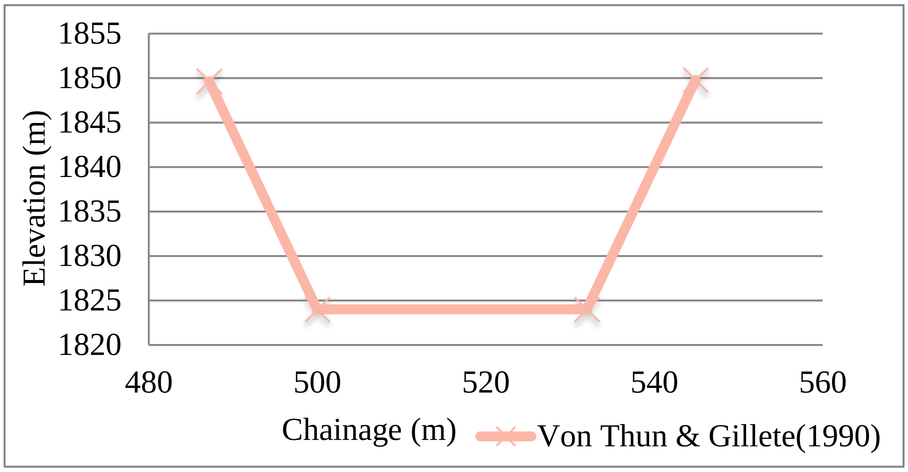


Figure 4-3: Breach parameter plot for piping scenario

Most of the reviewed authors were used envelope curve to get the accurate breach parameters out of the four equations' breach parameters result. But according to the HEC-RAS hydraulic reference (2016), envelope curve was developed only from fourteen failed dam data sets. This shows it was developed from a limited data hence it has a practical error on estimating upper limit of breach outflow discharge leading to unreliable and over damage estimation (USACE, 2016). Using envelope curve to identify accurate breach parameter equation also time consumer because it identifies the equation after breach outflows were calculated by all equations. But the breach parameter guideline range used in this analysis was identifying it after the breach parameters were estimated by all equation. This shows the guideline was one step faster than envelope curve.

The guideline prepared by many governmental organizations considers dam height and side slope of the dam to be analysed but envelope curve has no any consideration. Therefore using guideline breach parameter range to identify the accurate regression equation breach parameter estimator has so many accuracies and consistency over envelope curve. As a result of this it has applied in this study.Therefore for both failure scenarios of Gerhu-sirnay dam Von Thun & Gillete (1990) was best equation to estimate accurate breach parameters and the next procedure which is computation of unsteady flow analysis to obtain breach out flow was already with the specified equation here.

4.3. Breach Outflow

The computed unsteady flow analysis were both overtopping and piping based. Finally, the outflow routing was accomplished by wave diffusion equation of HEC-RAS 2D and detailed as below sections:

4.3.1. Overtopping Breach Outflow

After the program has run, the dam was collabsed for Von Thun & Gillete(1990) breach parameter dimensions consequently under the maximum volume of 5.34 Mm^3 and the peak breach outflow was reached up to 2,239.7 m^3/s at the 20th minute simulation times as labeled in figure 4-4 below. This means the maximum embankment material was erroded at the instant of this breach formation time.

The outflow hydrograph draining profile looks: 0 m^3/s outflow discharge at the duration of 0.0 hr., 2,239.7 m^3/s maximum outflow discharge at 0.33 hrs and at the end of breach simulation time (1 hr), the river flow has been remained 462.95 m^3/s. This means the breach progression was already completed but due to the effect of extreme rain event (PMP) wich was 129.7 mm during overtopping scenario (rainy day failure) the reservior was still draining. For this result it has a supportive idea in HEC-RAS Hydraulic Reference (2016) that, ‘‘the breach development ending time should not include the time to completely drain the rervoir’’.

This result was supported by hydrograph plott as below:

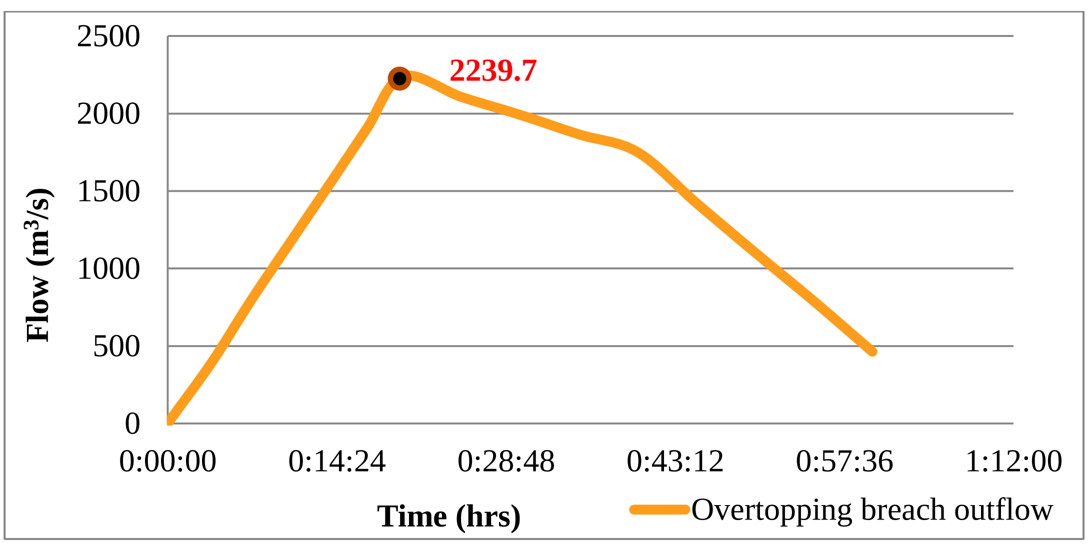


Figure 4-4: Overtopping breach outflow hydrograph

Ekaningtyas (2017) on the same scenario and model analysis with this study provided that from 56 m high and 20.15 million m^3 reservoir capacity, the outflow hydrograph was 15,022 m^3/s peak discharge over the dam body. Therefore, from the dam height of 24 m and reservoir capacity of 1.08 million m^3 the estimation of 2,239.7 m^3/s peak outflow discharge was acceptable and consistent due to geometric and analysis proportionality.

4.3.2. Piping Breach Outflow

Unsteady flow analysis for piping also performed in HEC-RAS 5.0.3. According to result of the model, the maximum breach outflow was 1,282.68 m^3/s at the 39th minute simulation times. At the end of breach progress, the river flow remains with 146.94 m^3/s flow. This indicates erosion of embankment material was stopped but still the water in reservoir was not completely outflowed or drained.

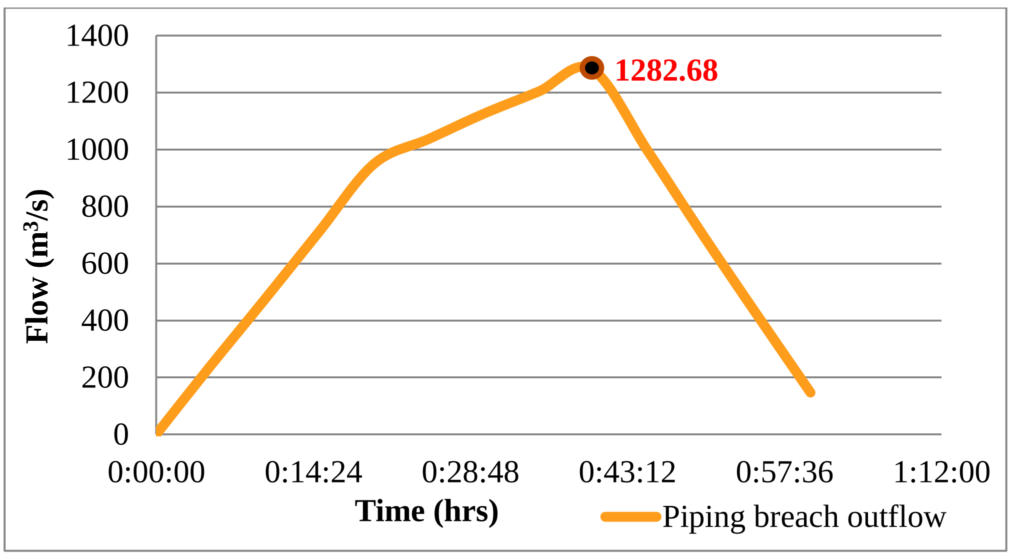


Figure 4-5: Piping breach outflow hydrograph

From both scenarios result, the total simulation duration was equal. This was because of that the inflow hydrograph ordinates, computational time intervals, breach simulation starting and ending date, and time were the same for both scenarios. However the maximum outflow point of time and magnitude of breach outflow for both scenarios was different at the same breach simulation interval. The reason for this situation was due to the discharge during failure instant of both scenarios were different in magnitude.

Overtopping scenario was rainy case then occurs after the inflow hydrograph was reached beyond the reservoir and spillway capacity whereas piping was sunny day scenario. Piping happens during the instant of maximum internal seepage up on the dam and may occur during the rainy or sunny environment. In this study's breach simulation piping was assumed to be occurred at the sunny day or the maximum internal seepage would be developed during a sunny day. Therefore, the cause of failure for both scenarios were different in magnitude consequently overtopping has higher inflow failure cause which results more than piping by 957.02 m^3/s. This leads to the expectation that, overtopping would cause higher flooding and exposure.

4.3.3. Comparison of Dam Profile after Breach

From the result of hydraulic model, the embankment profile for both breach scenarios analysed was plotted in figure 4-6 below. According to the expectation from breach outflow discharge and breach dimension sizes, overtopping scenario would create the wider breach embankment profile.

Therefore, the breach profile plotted below shows the same fact as the expectation above. According to the arrangement of the figure below, the width starting from the center of the river up to both sides of red graph represents the overtopping breach width that was 54 m at the bottom and up to the green graph which was 32 m also represented for the width of piping scenario. As a result of this overtopping scenario causes the wider breach profile.

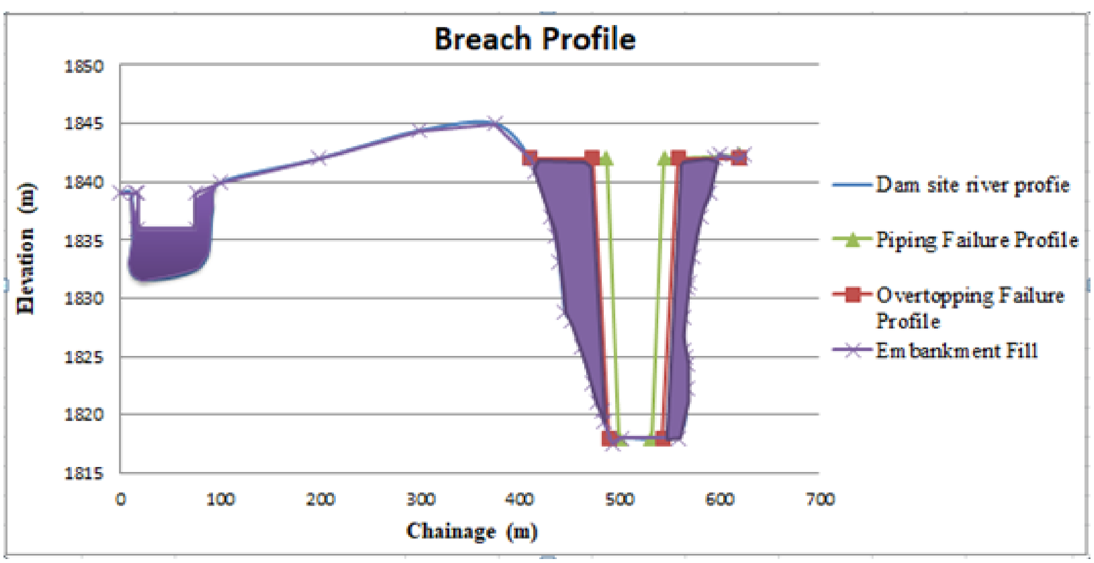


Figure 4-6: Dam profile after breach

4.4. Preparation of Downstream Flood Inundation Mapping

The unsteady flow analysis inundation mapping results were computed in HEC-RAS and arranged in ArcGIS according to the mapping standards. The inundation map was prepared for inundation boundary, depth, velocity, water surface elevation, flood duration and flood arrival time with respect to each breach scenarios according to requirement of the objectives.

4.4.1. Overtopping Scenario

4.4.1.1. Flood Inundation Boundary

Maximum flood inundation boundary map refers the maximum areal extent exposed by flood. The map was prepared by overlying the flood boundary map over the Ethiopian kebele map. The purpose of developing this map was to detect the total area exposed by flood and to carry out the flood damage analysis.

For this study overtopping failure scenario's maximum longitudinal inundation length was 11.04 km and its maximum width at the flattest flood plain zone was 0.472 km, provides 5.21 km^2 total inundated areas. According to the field survey sample measurement carried out on the average width of the river, it has 0.03 km. This shows for about 0.495 km flood inundation width was out of the river banks in both sides of the river separately, therefore it shows much area will be submerged out of the river banks.

As Faudzi1 et al (2019) have predicted, the 4 km by 0.2 km of maximum inundation on the same scenario from 300 m^3/s peak breach outflow. Therefore the 11.04 km by 1.02 km or 11.3 km^2 inundation extent from 2,239.7 m^3/s peak outflow discharge was reliable or consistent inundation extent.Therefore any flood analysis activity for this scenario should be carrying out with in this flood exposure.

According to the carry out field survey the inundation extent delineated in figure 4-7 below was narrow and deep at the steep slope and gorgeous flood plain areas and have wider extent at the flattest area. As shown in the figure the surface flood extent was comparatively increasing as it goes to the downstream end, this shows rivers has got flattened as it goes to downstream far. Generally it was given as in below figure:

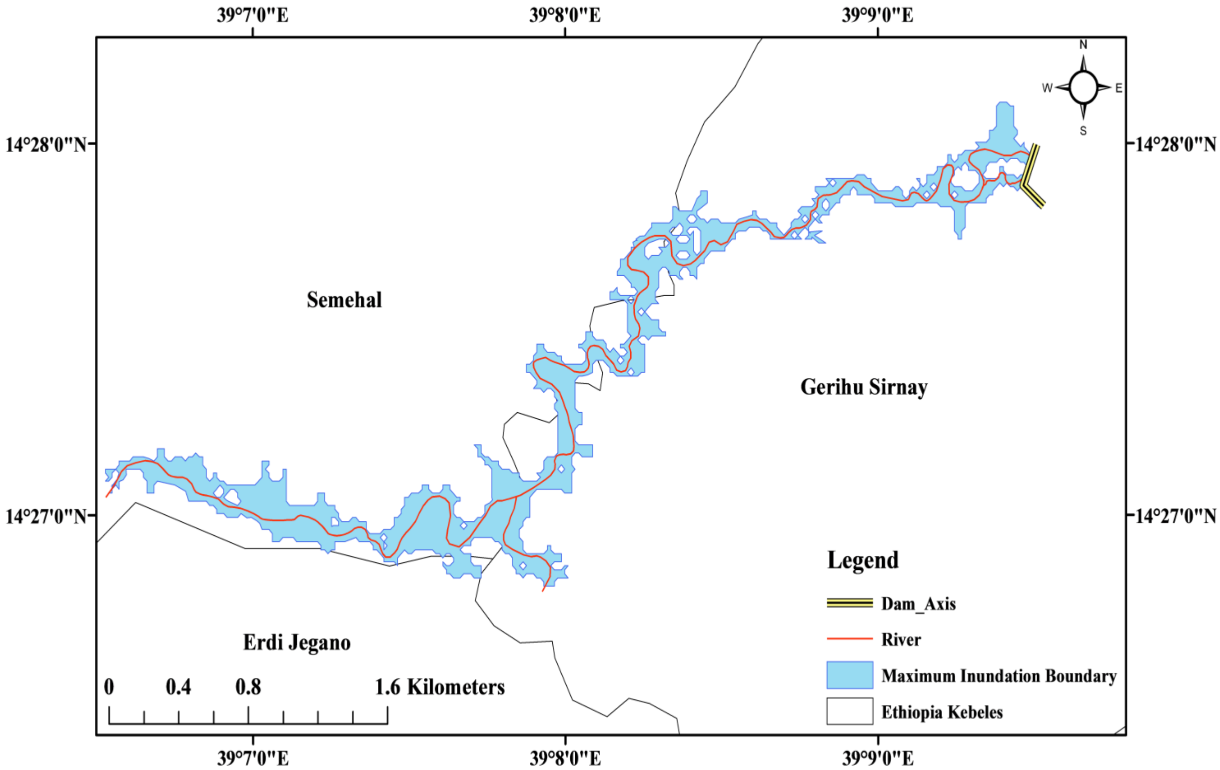


Figure 4-7: Maximum flood inundation boundary

4.4.1.2. Depth

Flood inundation depth map shows the area which has been affected in terms of depth detail. The depth detail result of this study's overtopping scenario varies between 0 m and 24.07 m. The maximum flood depth, 24.07 m was located at the area having topographical characteristics of narrow river width, steepest river bed slope and gorgeous banks. This area up on the 2D flood plain was represented by dark coffee colour in the map below and according to the analysis of this map the maximum flood depth would go to cause maximum damage.

Ekaningtyas (2017) predicted maximum depth from 15,022 m^3/s peak outflow discharge was 55 m on the same scenario with this study. According to Yakti et al (2018) conclusion on their HEC-RAS 2D overtopping scenario flood modeling study, the maximum flood depth was happened at the topographically narrow and cliff flood plain area. Therefore due to comparison of this study with this research, the depth detail result with its location of this study was realized and supported.

Therefore 24.07 m maximum depth estimated here in this study was acceptable with proportional value and location. The map detail was provided below:

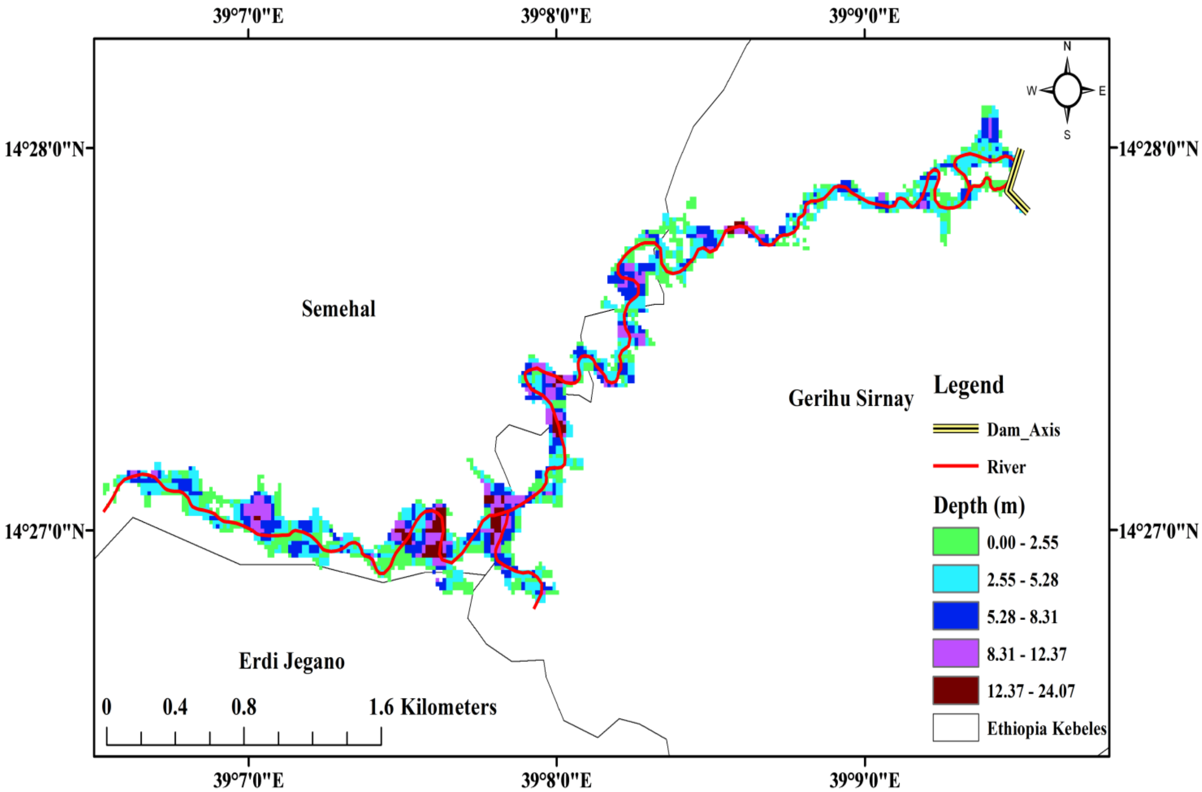


Figure 4-8: Inundation depth map

Due to depth variations, flow types may categorize as subcritical, critical and supercritical. Subcritical flow characterizes by a slow or stable flow condition (Chow, 1959). The driving force for this type of flow is gravitational force and mainly occurred at the flattest flood plain. Therefore the inundation area marked by the green colour in figure 4-8 above was the area of subcritical flow. The main features for supercritical flow were flow instability or presence of turbulent flow and having Froude number greater than one (Chow, 1959). This type of flow was occurred at the steepest flow area, meandering river nature and narrow river widths. These geographic features were represented by dark coffee and purple colours in figure 4-8. Therefore supercritical flow has occurred at the flood plains marked by the red and purple colours.

4.4.1.3. Water Surface Elevation

Water surface elevation map was the other flood inundation component that shows the summation of both flood depth and terrain elevation (USACE, 2016). The main purpose of this inundation result in this study was to predict the overflow depth of water during the instant of maximum breach outflow. As the result of this analysis, overtopping failure scenario will cause 2,239.7 m^3/s peak breach outflow as generated in section 4.4.1 and results with 0.89 m overtopping depth or height above the elevation of dam axis (1842 m a.m.s.l).

When this result was compared with Delhi dam in Delhi which was failed due to same scenario on July 24, 2010 it had 1, 955.4 m^3/s peak breach outflow with 4.6 m overflow depth and Walnut Grove Dam with 1.09 m overtopping depth. They have a proportional result that helps to ensure the overtopping depth result of this research analysis has good acceptability.

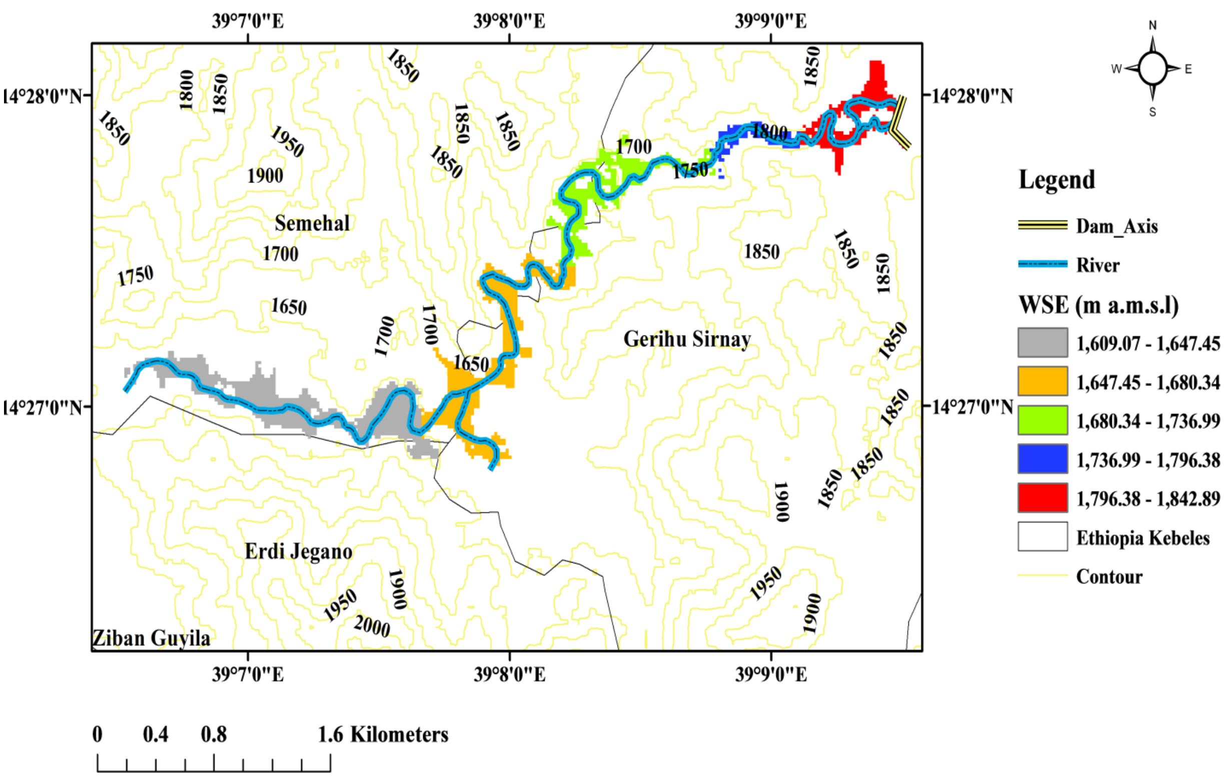


Figure 4-9: Water surface elevation map

4.4.1.4. Velocity

Analysis of inundation area in terms of flood velocity category was shown in the following velocity map. The map shows the 2D flow area due to velocity detail with a maximum velocity of 16.89 m/s at steepest area represented by a red colour.

Because of its steepest slope or maximum gravity agent driving force, the flow velocity has becomes maximum. In this type of location and velocity magnitude supercritical flow was created. Therefore the maximum risk on the 2D flood area would be the function of this flow velocity and flow type location.

Ekaningtyas (2017) has estimated 39 m/s maximum flood velocity from 56 m dam height and 20.15 million m^3 reservoir capacities on the same scenario, dam constituent materials and modelling approach with this study. In this research the maximum flood velocity was 16.89 m/s from 24 m dam height and 1.08 million m^3 reservoir capacities. Therefore when both studies were compared regarding to dam geometry and modelling similarity, the maximum velocity estimated in this research was reasonably proportional and consistent.

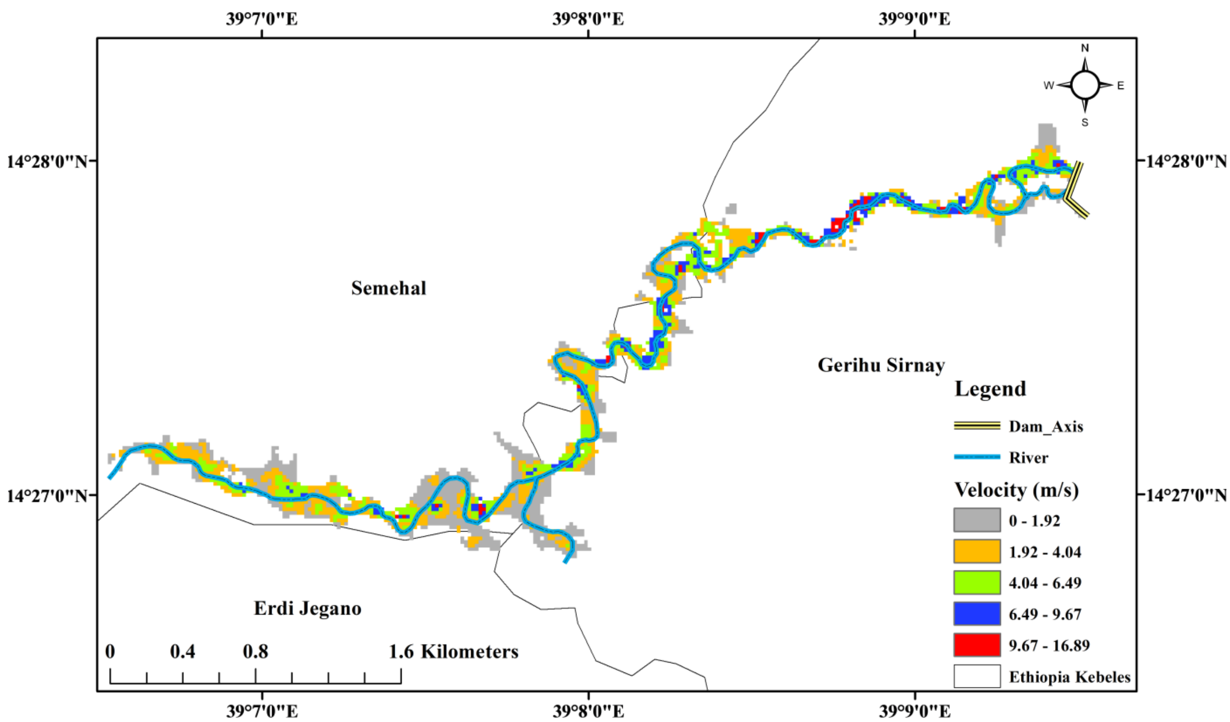


Figure 4-10: Velocity map

Australian Guideline for Flood Disaster Resilience (2017) has recommended that a flow velocity of 2 m/s and above was dangerous for people, crops and vehicles. Therefore as the developed velocity map above shows, overtopping scenarios can cause damage over property and life.

4.4.1.5. Arrival Time

Arrival time is the flood travel time to the specified hazard area. The main purpose of preparing flood arrival time map was to identify the targeted hazard points' flood contact time in relation to breach formation time. The longest arrival time overtopping scenario of this study was 1 hrs as prepared in below map but the shortest arrival time has higher flood damage due to massive flood exposure. This duration has a relation with breach simulation duration. When the breach outflow simulation has stopped the driving force for flood routing up on the 2D flow area would be zero and consequently the flooding water tends to sustain its equilibrium condition.

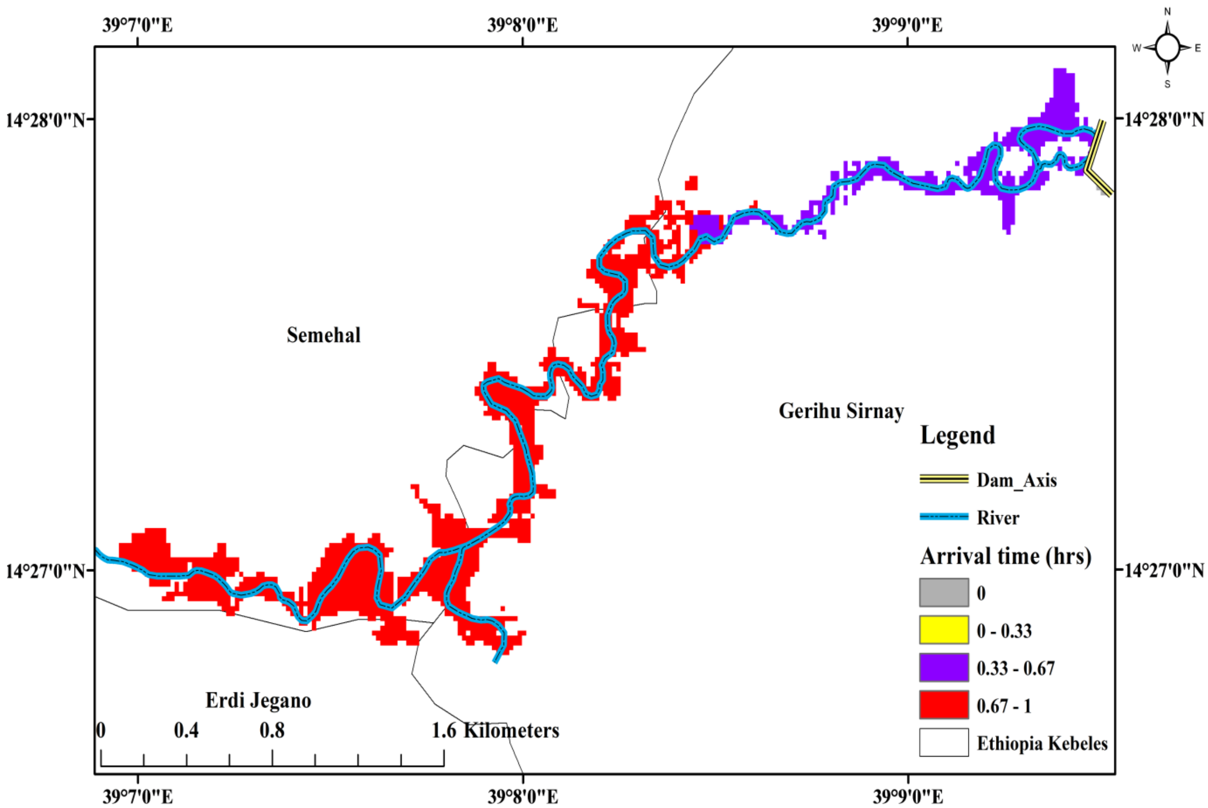


Figure 4-11: Arrival time map

4.4.1.6. Flood Duration

The flood inundation duration result of overtopping scenario was given below as 1 hrs with worst duration. According to this map interpretation, flood area having maximum flood duration would have large damage. From the details in the map below, the highest duration was appeared at the upper part of the flood area near the dam axis but due to its small surface extent it was not visible at the map hence it was symbolized by the red colour:

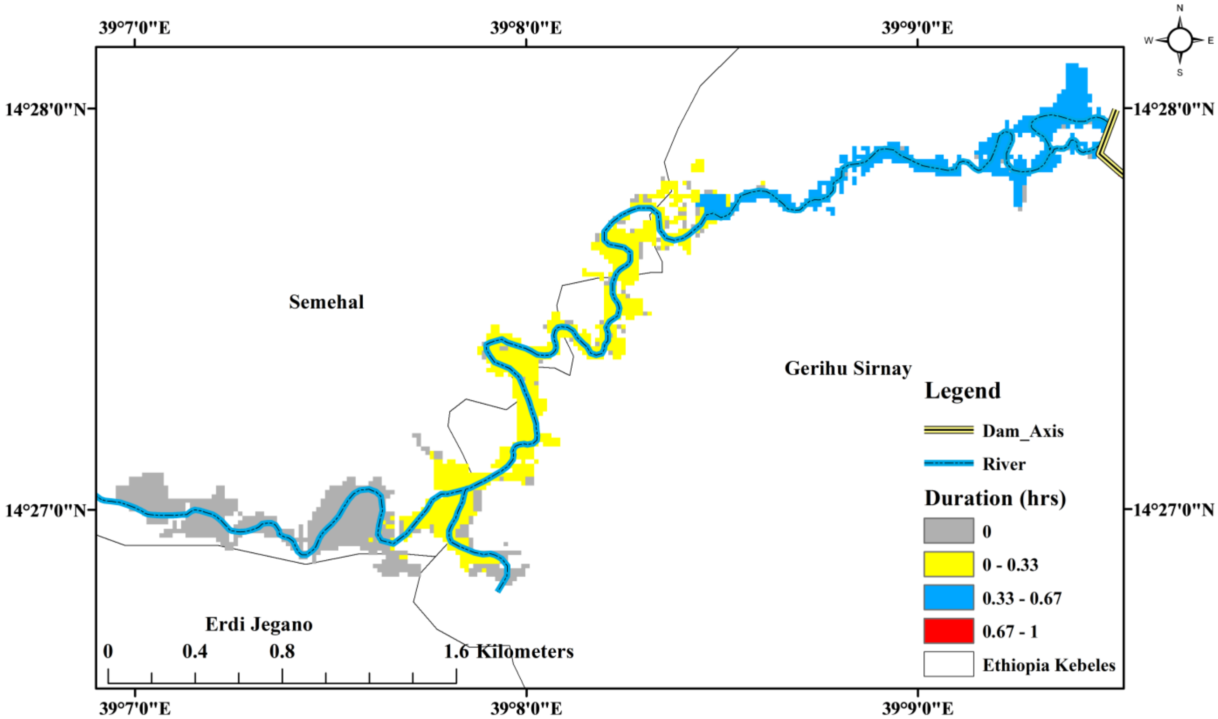


Figure 4-12: Flood inundation duration

4.4.2. Piping Scenario

4.4.2.1. Flood Inundation Boundary

Maximum flood inundation boundary map in piping failure scenario also refers to the maximum areal extent which was exposed by flood hazard. The maximum longitudinal inundation length was 9.65 km and its maximum width was 0.411 km with areal extent of 3.96 km^2. When it was compared with overtopping scenario, piping was sunny day hence it has lower breach outflow. Consequently every inundation data of this scenario has lower magnitude and cause lower hazard.

The map detail of this extent was prepared below:

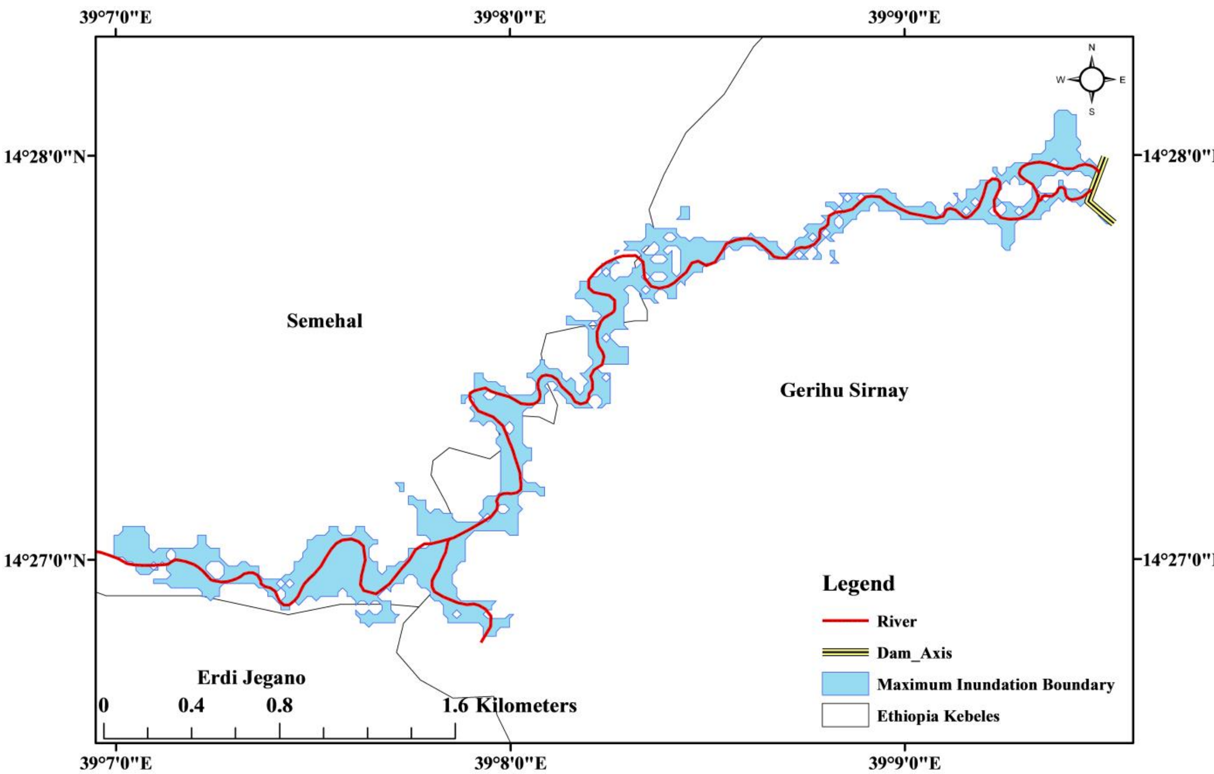


Figure 4-13: Maximum flood extent for piping

4.4.2.2. Depth

As it tried to discuss in the overtopping scenario above, there has been also the same analysis with piping case. But the only difference was in depth magnitude, total exposure and its level of risk. Therefore the maximum depth in this scenario was 24 m and its detail was prepared as below figure 4-14:

According to Amini et al (2017) study on peak flood estimation under piping and overtopping conditions shown that, a maximum of 37 m depth was appeared with the same scenario and proportional dam geometry with the case study of this research.

Adnan (2017) has also 22.5 m flood depth under similar methods and failure scenarios. Due to this a maximum inundation depth of Gerhu-sirnay dam due to its dam size would consistent with the findings mentioned here.

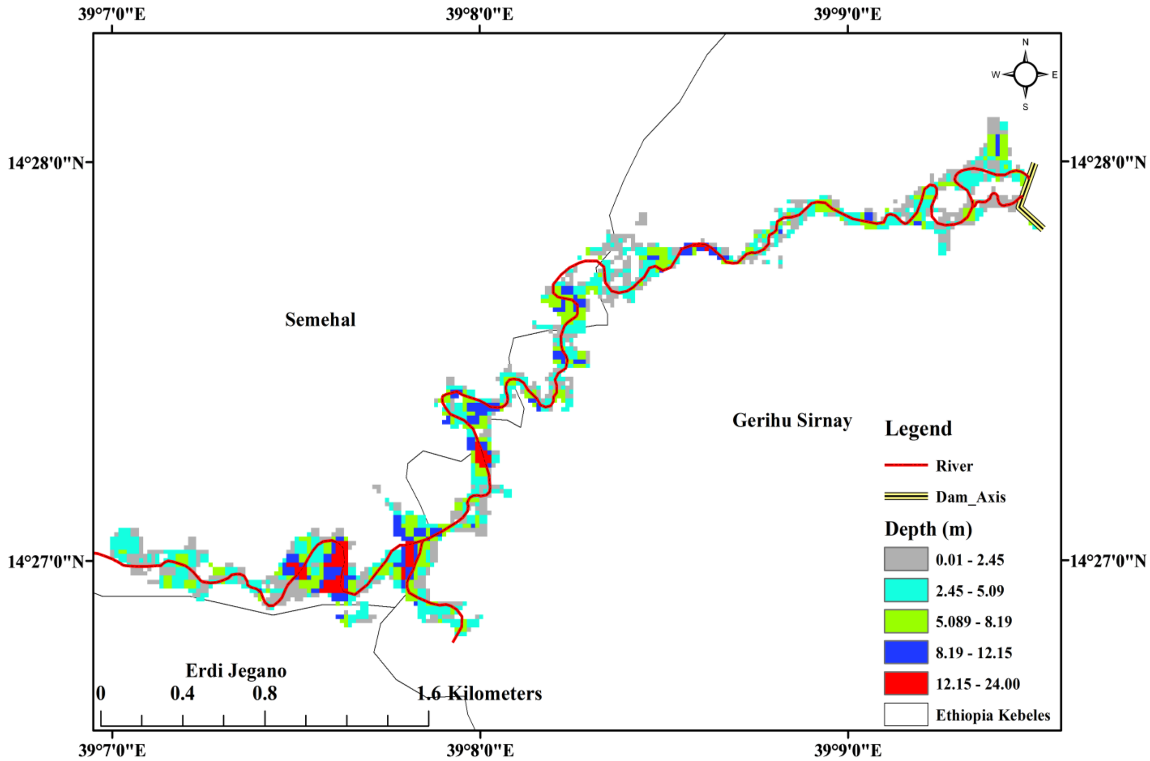


Figure 4-14: Flood depth of piping scenario

Based on Australian Guideline for Flood Disaster Resilience (2017) the flood depth above 1.2 m was unsafe for vehicles, children and elders. Therefore the depth detail piping scenario of Gerhu-sirnay dam shown in the above map has the capacity of damaging the whole 2D flow area's life and property.

4.4.2.3. Water Surface Elevation

The maximum water surface level of piping scenario up on the dam axis was 1836.41 m; this was 1.59 m below the reservoir normal pool level. Since the piping scenario of Gerhu-sirnay dam was assumed to be occurred at the sunny day, the maximum water surface elevation has become below the normal pool level of the reservoir. According to the analysis of this map, the location with the highest water surface elevation level owns the highest impact.

Its detail up on the whole flood plain was prepared in map below figure 4-15:

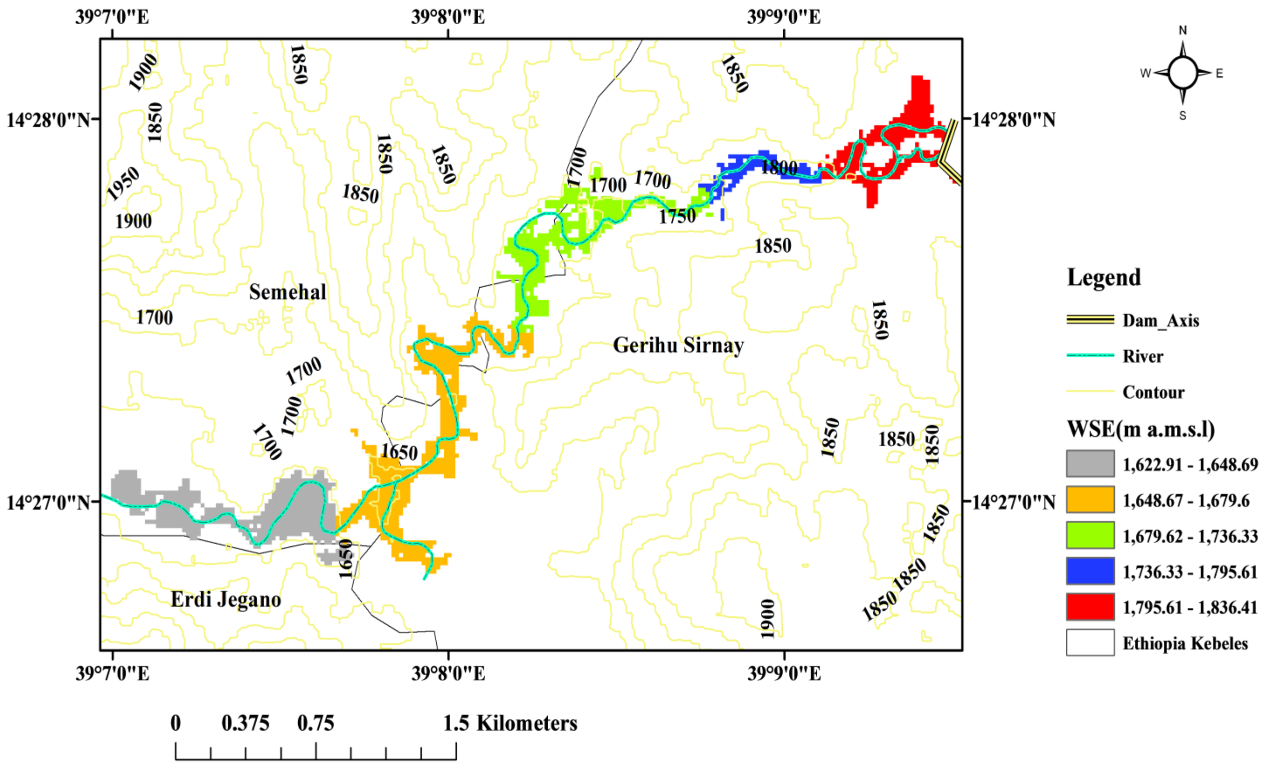


Figure 4-15: Water surface elevation map

Water surface elevation discussion was already proved in the inundation depth result above. Because water surface elevation result was already the summation of both inundation depth and terrain elevation. So if the inundation depth result was already accepted and discussed, the water surface elevation would tend to acceptable range. Therefore no additional discussion would require for water surface elevation here.

4.4.2.4. Velocity

Velocity map was one of the inundation hydraulic data that shows velocity detail along the 2D flood area. It is an important parameter to decide the hazardous nature of the breach outflow. Therefore piping scenario flood velocity was estimated as in figure 4-16 below. The velocity map prepared for this scenario was displayed with 15.72 m/s maximum velocity. According to the Australian Guideline for Flood Disaster Resilience (2017) flood damage category due to flood velocity, the velocity of this failure scenario is dangerous for life and property and it was detailed in below ma:

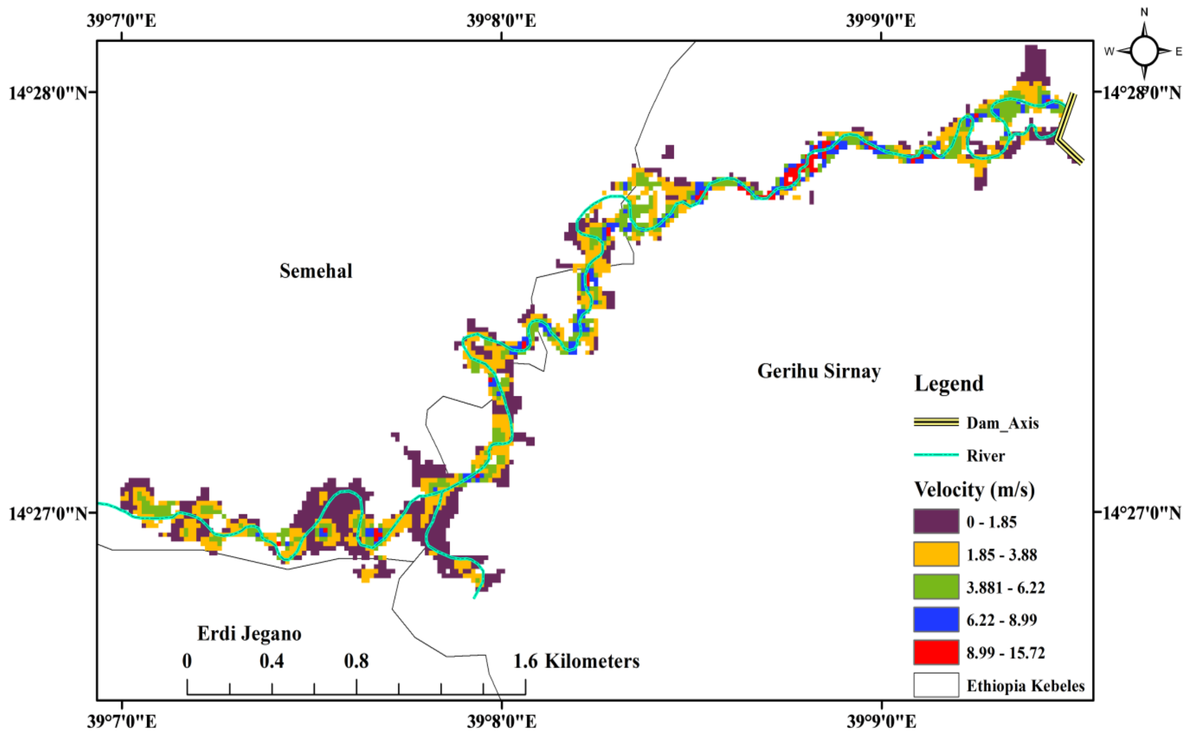


Figure 4-16: Velocity detail map

4.4.3. Inundation Mapping Conclusions

From both scenarios inundation mapping detail above, inundation data like depth, velocity and water surface elevation were reached their maximum value at the presence of complex channel irregularity or river meandering nature. In such river zones the flood driving forces like gravity force and flood wave actions may increase and the development of flow turbulence with supercritical type of flow will be appeared. In this study's 2D flow area, the channel meandering nature was observed nearly at the intermediate part of the flood zone.

Therefore correspondingly with the inundation data type the maximum values were appeared at such river natures. For example, when the depth map detail of overtopping scenario was considered, the maximum depth value region was marked by blue, purple and dark coffee colours. Comparatively with other part of the flood area these were the areas with higher channel irregularity (check figure 4-8).

The flood depth, velocity, duration and water surface elevation values were changed at the points where manning's n regions changed. When the value of n increase, the flood natures mentioned above tends to decrease due to the reason that manning's n value is measure of flow resistance. Then accordingly the inundation data variation continues through the 2D flow area.

4.4.4. The Worst Breach Scenario

To remove repetitive analysis, flood damage assessment for Gerhu-sirnay dam failure was carried out for the worst failure scenario. Therefore to identify the worst scenario, both scenarios must compare in accordance with the flood mapping results quantified in respective of both scenarios in above. The worst scenario should to mean that, the scenario with higher inundation data at corresponding location and time. Since the 2D flood area has 7,477 grid cells then it is difficult to compare every data throughout the flood extent. Therefore an easy way of comparison was with respect to the maximum inundation data's. Then the comparison data included were breach outflow discharge, flood inundation extent, flow depth and flow velocity. Therefore, both scenarios were compared in the following table:

Table 4-3: Comparison of Gerhu-sirnay dam breach scenarios

Breach scenarios	Peak breach outflow(m^3/s	Inundation extent (ha)	Depth (m)	Velocity (m/s
Overtopping	2,239.7	521.1	24.07	16.89
Piping	1,282. 68	396.6	24	15.72

According to the comparison in table 4-4 above, Overtopping scenario has higher inundation data including depth, velocity, inundation extent and breach out flow discharge. Then it expected that overtopping has more risky inundation data, hence it has higher damage. Therefore overtopping would be the worst scenario of this study.

From the list of failed dams in table 2-1, the failed dams due to Overtopping were more destructive than that of piping. Example Tigra Dam has failed due to piping with 1,000 fatalities whereas Machchu Dam has failed due to overtopping with 5,000 fatalities, St. Francis Dam failed due to piping with 600 fatalities and Banqiao Dam also failed due to overtopping with 171,000 fatalities.

Therefore overtopping was being the worst scenario of this study was supported by the previously failed and well documented dams in history as mentioned in the previous paragraph. So the next objective which was flood damage assessment to evaluate potential damage of the hazard was overtopping scenario based. That means every inundation map detail and any other requirements for assessment of downstream flood damage would be taken from the worst scenario.

The inundation extent in which flood damage assessment would be carried was already delineated in figure 4-7 above with GIS and its topographic inundation extent detail from the combination of RAS Mapper, GIS and Google earth also prepared below:

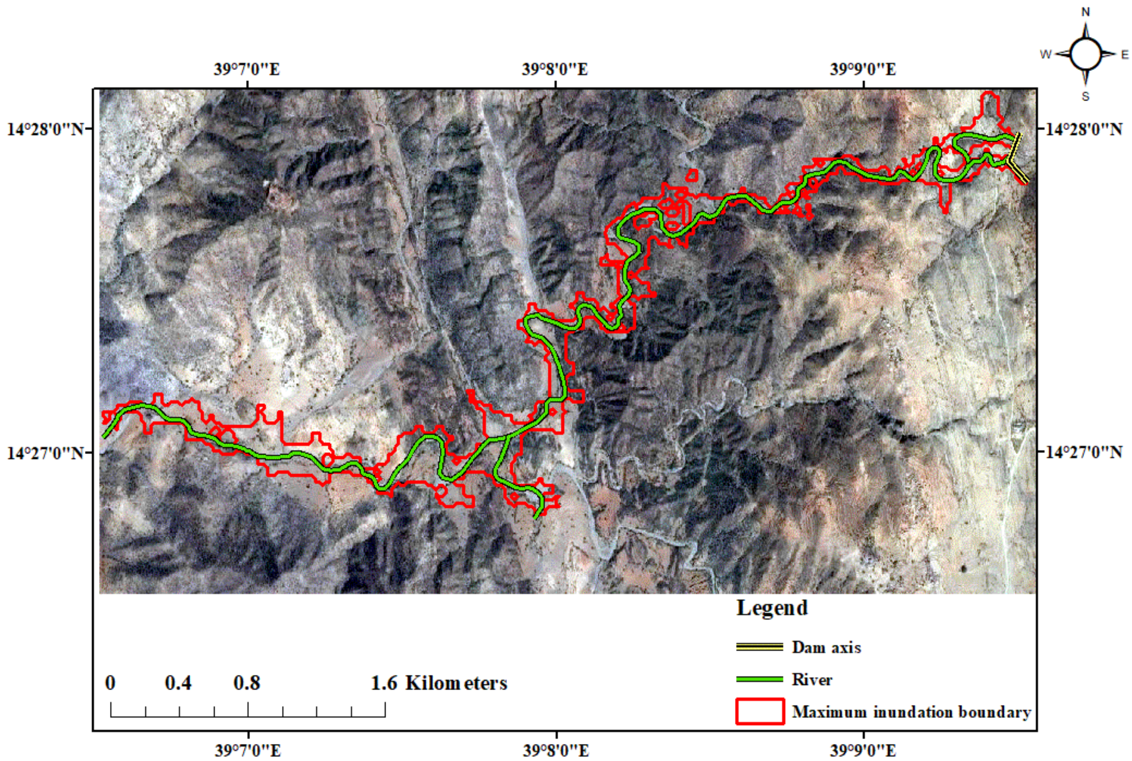


Figure 4-17: Inundation extent from RAS mapper and Google Earth

4.5. Flood Damage Assessment

4.5.1. Flood Exposure

As per result of the worst scenario, a total of 521.1 ha area will be inundated for the maximum duration of 1 hour along the flattest area. The flood arrival time and duration details primarily used for computing life loss and agricultural damage computations were detailed in figure 4-11 and 4-12 respectively.

Based on the probable damage type and consequence category which will be occurred, the 2D flood area was classified in to upper, middle and lower hazard zones. Out of the total inundated area 88.721 ha was agricultural lands with 68.3% of it was in the lower zone (from land 1 to land 20), 9.1% of it in the middle zone (lands 21& 22) and the rest 22.6.1% was also in the upper zone (lands 16 to 22). The 3.71 ha was the area up on which water treatment plant camp was constructed in the upper zone. This zone was the area of structural and life loss damage occurrence. The rest 428.7 ha was also grazing area, vegetation cover and natural habitat located on all zones (not included in flood damage assessment of this study).

Damage category of all hazard zones were quantified by using the magnitude of flood depth and velocity as in below table:

Table 4-4: Depth, velocity and both characteristics of each zone

Hazard Zone	Depth (m)	Velocity (m/s)	Damage category
Upper	0.00-24.07	1.747-16.89	High
Middle	0.00-24.07	1.747-16.89	High
Lower	0.00-24.07	0-16.89	High

According to the Australian Guideline for Flood Disaster Resilience (2017), the value of flood depth and velocity above 1.2 m and 2 m/s respectively was dangerous for human life, plants and property. In the above table, the maximum depth and velocity in all zones were the cause for high damage category but due to the exposure type on these zones damage may not necessarily high. Therefore the quantified depth and velocity (table 4-5) above up on all hazard zones of this study has the power to cause economic and life loss damages. Then according to the occurrence of damage types with respect to each zone the flood damage will be assessed in section 4.6.1 and 4.6.2 below.

The map below shows the detailed exposure of damageable items including agricultural areas, structures and human life as a function of each hazard zone. As the exposure detail of below map, for about twenty nine agricultural lands, nineteen treatment plant structures and seventy human beings will be exposed for flood hazard. The exposed agricultural areas were labelled in the legend below map from Land-1 to Land-29 but the exposed structures and humans were within the water treatment area represented by the red colour.

In the map below the exposure details of agricultural lands, structures and life loss were expressed in terms of their geospatial occurrence.

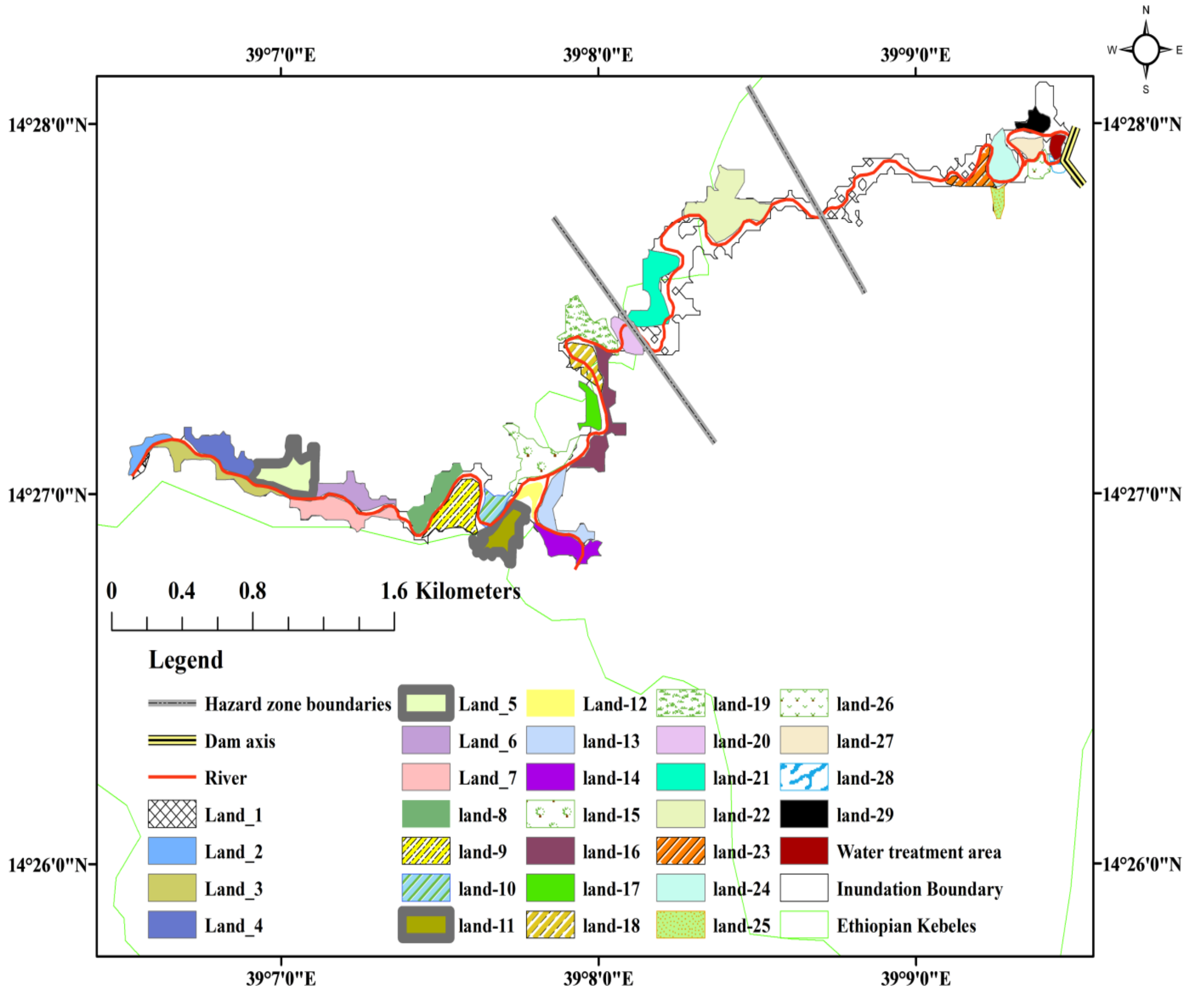


Figure 4-18: Zones of flood hazard

4.5.2. Ex-Ant Direct Economic Damage

The direct economic flood damage assessment of Gerhu-Sirnay dam failure was focused on agricultural and structural (water treatment plant) damage calculations:

4.5.2.1. Agricultural

As agricultural inundation detail stated in figure 4-18 above, the flood covers a total of 88.721 ha agricultural area. From this, the percent coverage of each crop was 59.85 ha sorghum, 13.57 ha barley and 15.3 ha millet for maximum duration of 1 hours. This crop submergence duration for a maximum of 24.07 m and 16.89 m/s depth and velocity respectively has enough power to destroy the productivity of crops as a function of crop types.

Crop damage estimation may be expressed in terms of quintal or money. The damage in this analysis was estimated in quintal and money but the cost of crops per quintal fluctuates overtime. Therefore it must consider that, the crop damage estimation in terms of money was the current Ethiopian crop value economic inflation based. As prepared in fifth column of table 4-6 below, the total crop damage in quintal was 2, 296.21; detailed as 1, 706.8, 242.6 and 346.8 for sorghum, barley and millet respectively.

Nebiyat (2016) on case study of Teji River, Ethiopia estimated 40,034.2 quintal crop loss from total 1,617.5 ha inundated area under a maximum depth of 0.75 m for Teff, Lenities and Chick pea. Then from comparison with this result, the estimated crop damage in this study was consistent regarding to depth, inundation area and crop type variation and proportionality.

From the static model simulation report (HEC-FIA 2.2 model) crop loss calculation detail, the economic damage estimation as a function of these crops were given in below table:

Table 4-5: Crop damage report summary

Crop type	**Crop land Area, ha**	**Yield, qt/ha**	**Product, quintal (qt)**	**Crop Value, Birr**	**Harvest Cost, Birr**	**Agricultural Damage, Birr**
Sorghum	59.85	37.14	1,706.8	2,474,902.9	150,750	2,324,152.9
Barley	13.57	23.28	242.6	412,465.8	116,100	296,365.8
Millet	15.30	29.51	346.8	572,182.6	133,425	438,757.6

As per the model's crop calculation concept, the total agricultural damage cost was calculated by subtracting the harvest cost from that of crop seasonal production value. Harvest cost was obtained from the product of total number of labors and their daily cost as a function of each crop. This was because that each crop has different harvest effort. From the local own knowledge, the standard of current labor cost and harvesting efficiency was 225 Birr and 0.05 ha respectively. According to the EASSSA (2017/18) report the seasonal production standard of each crop was provided (see appendix H, viii). Accordingly the total crop damage from the three crops calculated was 3,059,276.29 Birr. So as per Ethiopia's economy, this much agricultural damage was enough to cause seasonal famine over the localities.

4.5.2.2. Structural

The structures considered for damage assessment of this analysis was Gerhu-sirnay town water treatment plant with their corresponding content values. Content value means the detail content of damageable properties within each structure. According to (TWRB, 2019), the structural and content detail of the treatment plant includes about nineteen structures, four cars (one ambulance, one Tata transportation service and two double cup Hilux), twelve treatment machines and others (future) developments mentioned in the design document, for more detail of individual items see appendix H (i to vi). From the delineated detail of hazard zones in figure 4-1, the water treatment plant camp was located at the upper hazard zone.

According to the depth and velocity map detail of the worst scenario prepared in figures 4-8 and 4-10 respectively, all these structures will be submerged under 6.153 m to 9.229 m depth interval and faced to 3.449 m/s to 16.89 m/s flow velocity.

From the flood damage quantification guide for both depth and velocity of Australian Guideline for Flood Disaster Resilience (2017), flood depth above 1.2 m and velocity above 2 m/s was unsafe and dangerous for buildings, life and vehicles (see table 2-8). Kreibich et al (2009) standardized that inundation depth of 2 m was a critical impact level of flood depth and above it considers as destructive depth of structures or residential buildings.

According to the structures' foundation height detail in appendix H (i), all the structures have lesser foundation height than the inundation depth except only the generator house (5.61 m) and this gives an implication that all the structures would be damaged due to the flood from the Gerhu-sirnay dam failure.

The structural and its content detail was prepared in figure 4-20 below:

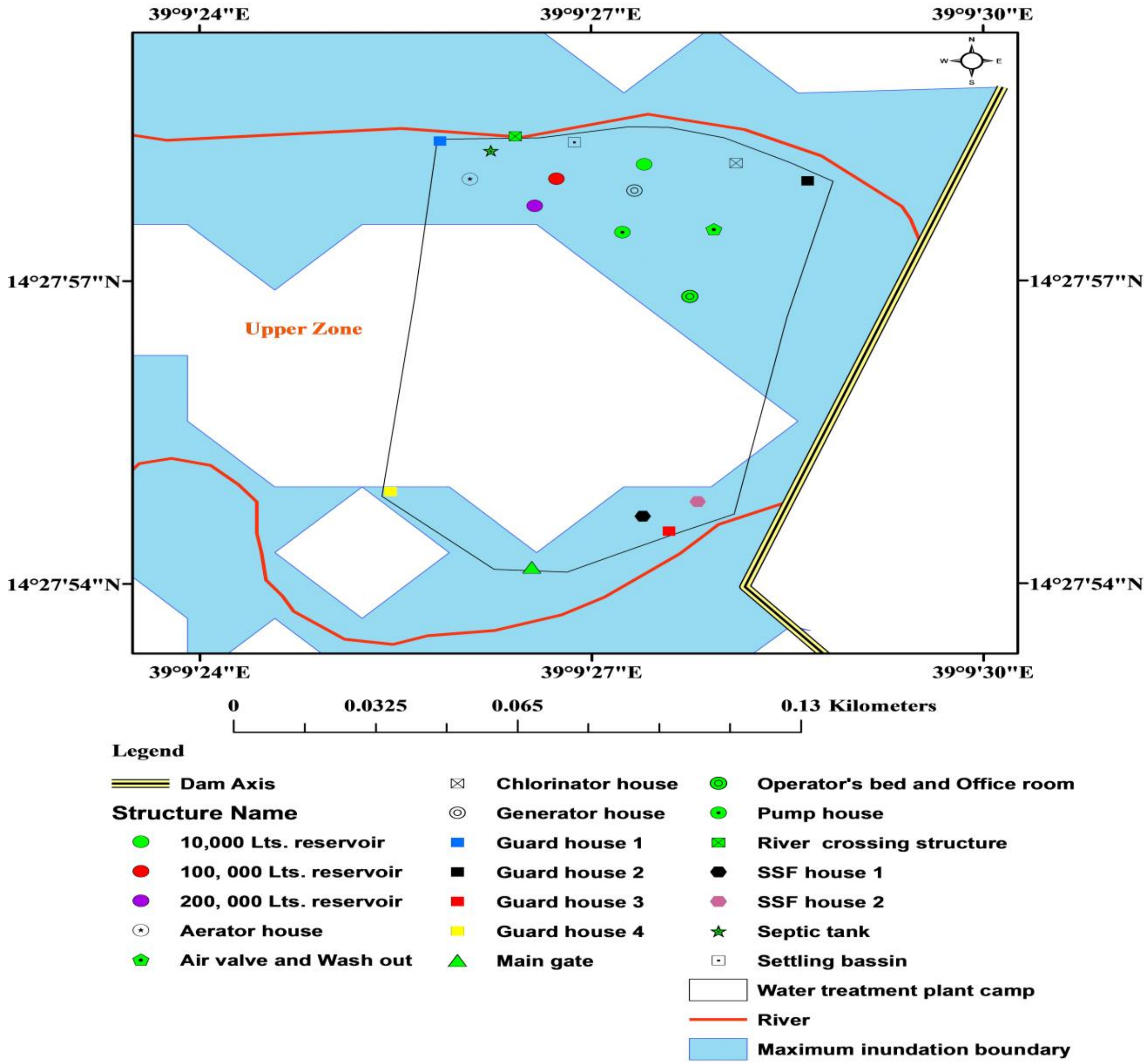


Figure 4-19: Location of damage for structures

The mapping result shows, the treatment plant was located at an average of 3.5 m distance from the downstream toe of the dam /downstream face. This distance was very short and leads to a fast flood time arrival, destructive energy head and velocity.

Therefore from the discussions above, all the structures, vehicles and content machines found in Gerhu-sirnay town water treatment plant would damage immediately when the dam would failed with the above inundation parameters.

The summarized economic damage report of HEC-FIA model would provide in below table:

Table 4-6: Estimated structural damage

S no.	Damage items	Cost, Birr
1	Structural	50,969,147
2	Treatment plant machines	3,030,853.2
3	Cars	6,492,942
4	Others (future works, one main pump)	98,275.149
		Total structural damage Cost = 60,591,217 Birr

Generally, the direct economic damage assessed from the worst failure scenario of Gerhu-sirnay dam was the total sum of damages estimated from agricultural and structural. Then as per the result of the model, the agricultural damage was 3,059,276.29 Birr and the structural was also 63,650,493.3 Birr.Therefore, the total direct economic damage from Gerhu-sirnay dam failure would be 63,650,493.3 Birr. Using this economic damage assessment result as input, the responsible organization for damage response should aware and prepare sufficient amount of recovery.

4.5.3. Life Loss

From the result of LIFE-Sim dynamic tool, after the three warning diffusion rounds, the total warned and mobilized people to safe zone was 95%. At the first 30 minute warning round 79.8% of the population at risk were warned and mobilized to a safe zone, at the second warning chance it reaches to 88% and at the final warning round totally the mobilized stakeholders were accounted to 95%.

Tariku (2015) evaluated total numbers of fatalities for precise and vague understanding of warnings was found 1,012 and 2,972 respectively for 6,084 m^3/s outflow discharge. Faudzi1 et al (2019) also reported the death of three lives under a maximum of 300 m^3/s breach outflow.

Then by comparison with these authors' result the life loss in this study was carried out with the specified LIFE-Sim tool and has proportional result, therefore it shows better accuracy.

The detailed mobilization scenario per each warning round was prepared in below figure:

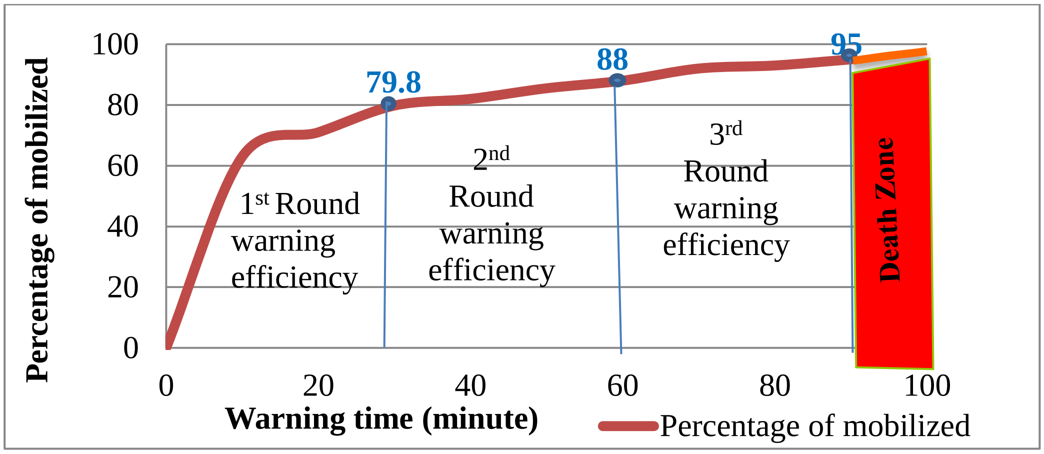


Figure 4-20: Warning-Mobilization curve

The detailed result of life loss from the LIFE-Sim model was provided in table 4-8 below:

Table 4-7: Life loss result for LIFE-Sim model

Warning terms	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**
PAR	29	100%	6	100%	35	100%	0	0
Warned	29	100%	4	67%	33	95%	0	0
Not mobilized	0	0%	2	33%	2	5%	0	0
Mobilized	29	100%	4	67%	33	95%	0	0
Life Loss	0	0%	2	33%	2	5%	0	0
Total lives could be died = 4								

Where: PAR= Population at risk

A=People Under the age of 65 years with day time working scenario

B= Percentage of people under the age of 65 years with day time working scenario

C= People Over the age of 65 years with day time working scenario

D= Percentage of people over the age of 65 years with day time working scenario

E= People Under the age of 65 years with night time working scenario

F= Percentage of people under the age of 65 years with night time working scenario

G= People Over the age of 65 years with night time working scenario

H= Percentage of people over the age of 65 years with night time working scenario

From the model's result above there are three analysis scenarios described as below: The day time workers under the age of 65 years (A) were totally 29 persons in number. So during the warning time all of them were warned and mobilized to a safer zone. Therefore no one would loss his/her life.

The total stakeholders of day time above the age of 65 years (C) were 6 persons. Out of these 67% of them were warned and the rest 37% of them meaning two stakeholders were not warned and mobilized, hence they loss their life.

The workers under the age of 65 years night time working scenario (E) were also 35 persons. During warning time 95% of them were warned and reach at a safe zone. Therefore the rest 5% or 2 workers would die.

Generally, from the mobilization curve above out of the PAR 70 people, the 95% of them were received warning and mobilized to a safe zone. But the rest 5% or four persons would be loss their life. Therefore, according to the dam breach hazard classification of Colorado state office of dam safety branch (2010), the hazard classification of this case study was high. This is because that if at least one human life was lost, automatically the hazard classification becomes high.

4.6. Hazard Potential Category

4.6.1. As per Gerhu-Sirnay Dam Breach

The hazard potential analysis result of Gerhu-sirnay dam breach was recorded the damaged elements as Gerhu-sirnay town water supply treatment plant with its costly treatment machines as public infrastructure, public seasonal agricultural areas and four human lives losses they were stakeholders of the treatment plant. From the definition of Colorado state office of dam safety branch (2010), if features like public infrastructures, public services and life loss damages were appeared from the hazard, automatically the hazard potential classification belongs to high hazard category. Therefore, the hazard potential classification of this case study belongs to high hazard category.

After this destructive flood damage analysis result, the design document of Gerhu-sirnay dam was checked if it includes dam breach flood analysis but it hasn't included. Therefore by using this study's result as input, Gerhu-sirnay dam breach analysis study should be carried out before the hazard would happened to minimize the lives and economic damages.

4.6.2. As Per hazard Zone

Based on the Hazard Potential Classification, the three zones in the impact area of this study were categorized as low hazard potential zone, intermediate hazard potential zone and high hazard potential zone.

A. **The low hazard potential zone (middle zone):** the middle flood hazard zone would cause only the private owners agricultural damage for about 258,405.2 ETB and some forested environmental damage. Therefore based on Colorado state office of dam safety branch (2010) hazard potential classification, the hazard potential from this zone would be low.

B. **The Intermediate hazard potential zone (lower zone):** the lower zone flood hazard was located at downstream end of the inundation area with inundation data characteristics like: maximum velocity, maximum depth, inundation area and arrival time as 16.89 m/s, 24.07 m, 60.3 ha and 1 hour respectively. From the results recorded on this zone: damage of public grazing areas, disorder of many private owner seasonal agricultural activities which measured in 2,080,307.9 ETB could be destructed and damage of some environments due destruction of some forest area will occur. As per the rule of Colorado state office of dam safety branch (2010) hazard potential category, the damage potential would occur in this zone were belongs to intermediate hazard.

C. **The High hazard potential zone (upper zone):** in the upper zone of flood hazard area, the damage records will: the loss of four lives, public infrastructures (water supply treatment plant) destruction and crop damage will be observed. According to the Colorado State Guidelines for Dam Breach (2010) definition; even if one life loss was happened the hazard classification would automatically high. Therefore as discussed above sections, the upper flood hazard zone of this paper case study would cause four life losses and then the hazard potential up on this zone would high potential hazard category.

Generally, the hazard potential up on each flood hazard zone would summarize in below table:

Table 4-8 : Summary of hazard potential category on each hazard zone

Hazard zone	Damage type	Hazard potential category
Upper	Four (4) lives were lost	High
	Public infrastructures (60,591,217 ETB)	
	Public seasonal crops (634,267.4 ETB)	
Middle	Private seasonal crops (258,405.2 ETB)	Low
Lower	Public seasonal crops (2,080,307.9 ETB)	Intermediate
	Damage of public grazing areas	

4.7. Emergency Action Plan (EAP)

The results to emergency action plan were just preparing of efficient guidelines and procedures on operating emergency action plan (EAP) to minimize the life loss and property damage. So the earlier results for efficient preparation and documentation of this objective were the identified lives and properties damaged for the probable hazard of Gerhu-sirnay dam breach in section 4.6.

Based on its effective purpose, emergency action plan in this thesis has two operation phases prepared as below:

- Pre-Event operation phase
- After-Event operation phase

4.7.1. Pre-Event Operation Phase

The pre-event operation of emergency action plan includes the procedures and guidelines for operating emergency action plan before the dam breach hazard will be happened. This type of emergency action plan operation aims at completely preventing life loss and minimizing movable property damage:

If the dam failure situation would have known before the prepared warning duration, it would possible to mobilize all the populations at risk (PAR) to a safer zone hence no life loss will be happen. This means all the population at risk would mobilize and reach at a safe zone before the warning duration rounds would finish.

To do this all the stakeholders should take awareness about the hazard severity and warning alerting type in the form of short training and newspaper written materials. This notifies that, the proper pre-event emergency action plan operation has the power to minimize the high hazard category of Gerhu-sirnay dam breach hazard to intermediate hazard category. This is because that, the populations at risk can be mobilized to a safer zone due the existence of long warning and mobilization duration.

In addition to the warning rounds duration, there would be long time duration before the initiation of dam failure. Therefore the movable properties like: the costly machines with in the treatment plant, the cars in the treatment plant area and other movable properties subjected to the probable hazard could be transported to the safer zones. Then the property damage would minimize in an extent corresponding to the mobilized properties.

Therefore from the above paragraphs' justification, the efficient pre-event emergency action plan would be the key and important tool to minimize the hazard potential of this case study. This conclusion provides the answer for the last research question in section 1.4 of this study.

- **Operation Guidelines**

To have efficient emergency action plan, the local community and responsible organization should get a good awareness of its procedures. They should arrange a periodic schedule to supervise the dam for the purpose of maintenance if possible and otherwise for warning.

The supervisors should be professionals on dam safety and well responsible on the hazard that will happen. They should carefully observe the signs of dam failure weather for piping or overtopping. The major sign for piping scenario could be seepage of muddy fluid through the downstream face of Gerhu-sirnay dam and the major signs for overtopping scenarios could also spillway malfunctioning, wet season time to time increasing of both reservoir level and the local probable maximum precipitation (PMP). As it has checked in the HEC-RAS model, Gerhu-sirnay dam would subject to overtopping failure scenario at the probable maximum precipitation values equal to or greater than 106 mm.

Decision, if the mentioned above failure signs would appear, the client organization of the structure should maintain the structure and prevent the dam from failure hazard. But if the failure signs would impossible to control and the dam failure will to happen, the organization should warn to the areas exposed for hazard delineated in Figure 4-17 especially the hazard zone with movable properties and PAR.

4.7.2. After-Event Operation Phase

The after-event operation of emergency action plan includes the procedures for operating emergency action plan immediately after the hazard has happened. The purpose of this type of emergency action plan operation would to minimize life loss, property damage and to prepare recovery budget from the concerned body.

- **Operation Procedures**

This type of operation could operate by the client organization in combination with other emergency response organizations and federal concerned agencies. Immediately after the hazard has occurred, those organizations should use different machines, equipment and skillful humans to minimize life and property damage.

This emergency action plan operation system mainly would practice to minimize majorly the life of population at risk and somehow damages for easily movable properties like transportation service. Finally, the saved people should transport to the prepared shelter and medical treatment stations for recovery purposes. To make the prepared emergency action plan will efficient, the local community and responsible organizations should get a good awareness of its procedures.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

This research investigates Gerhu-sirnay dam breach analysis and quantification of its flood damage. Embankment dams may have subjected to many failure scenarios however results of this study shows Gerhu-sirnay dam will subjected to only overtopping and piping .Therefore downstream of the dam there are elements to be damaged from the outflowing flood. The dam breach parameters for both scenarios were calculated by the four regression equations available in HEC-RAS breach calculator tab. But due to the comparison of all these equations, Von Thun & Gillette (1990) was the most accurate. Therefore, for overtopping scenario the breach bottom width and breach formation time were 54 m and 0.73 hrs respectively, and for piping these parameters were 32 m and 0.53 hrs respectively.

The breach outflow simulation was modelled by HEC-RAS 2D version 5.0.3 under the upstream and downstream boundary conditions as analysis limit. The upstream boundary condition was inflow hydrograph with peak flow of 329.1 m^3/s obtained from standard dimensionless Soil Conservation Service (SCS) method and its downstream boundary condition also normal depth computed from manning's n formula with a magnitude of main river slope, 0.02189. Then the model has delivered peak breach outflow of 2,239.7 m^3/s and 1,282.68 m^3/s for overtopping and piping scenarios respectively.

The downstream flood inundation mapping maximum values including inundation extent, inundation depth, flow velocity and water surface elevation for both scenarios have been prepared by RAS mapper in combination with Arc Map. For overtopping scenario the specified inundation data according to the above order was 521.1 ha, 24.07 m, 16.89 m/s and 0.89 m above dam axis. For piping scenario, these results were also 396 ha, 24.0 m, 15.72 m/s and 1.59 m below dam axis.

Flood damage assessment of Gerhu-sirnay dam failure was carried out by using HEC-FIA version 2.2 to the worst dam breach scenario. The assessment includes static and dynamic simulations for direct economic damage and life loss estimations respectively. Then, the estimated damage was about 63,650,493.3 Birr and the life loss report also shows that four lives will be subjected to death. Finally emergency action plan was developed as pre-event and after-event operation phases to minimize the degree of flood damage.

5.2. Recommendations

As per result of this study's analysis Gerhu-sirnay dam will be failed due to overtopping at the incidence of maximum probable precipitation (PMP) which was 129.7 mm. To prevent this failure the measurements may to take would be: by considering more metrological and hydrological studies provide emergency spillway, increase safe and economical freeboard regarding to the maximum overtopping depth, and provide dam's slope erosion protection materials like stone riprap, grass plantation and other appropriate materials.

The water treatment plant of Gerhu-sirnay dam was the place of high hazard category then it must be out of the areas affected by flood. From the maximum flood extent result, there is safe and suitable area for installing the water treatment plant to the left side of its current location at least 240 m apart from it. For those they can't be resettled properties like agricultural areas, the responsible organization must prepared replacement cost regarding to their damage assessment result of this study.

From the warning diffusion dynamic analysis result, the stake holders having age above 65 years have less mobilization performance and will be subjected to death. Due to this reason, the age of all Gerhu-sirnay dam treatment plant's stake holders must be below 65 years and then they will have good mobilization performance to save their life. In addition to these measurements, the stake holders should get a good awareness about the hazard's destructivity and operation of the prepared emergency action plan to minimize the damage.

REFERENCES

Abimeal Leul. (2015). Dam Breach Analysis Using HEC-RAS and HEC- GeoRAS the Case of Kesem Kebena Dam. M.Sc. Thesis. Addis Ababa University: Ethiopia, 92p.

Aboelata M., Chen A. and David S Bowles. (2004). Transportation model for evacuation in Estimating dam failure life loss. ANCOLD 2004 Conference, p2 to8.

Ackerman C. T. and Brunner G.W. (2006). Dam failure Analysis Using HEC-RAS and HEC-GeoRAS. Hydrologi engineering center, CA-95616, 8p.

Addisalem Eyob. (2019/20). Dam Breach Analysis and Flood Inundation Mapping for LowerAwash Multipurpose Dam. M.Sc. Thesis. Addis Ababa University: Ethiopia, 48p.

Adnan Arega. (2017). Dam breach analysis and flood inundation mapping a case study of. MSc Thesis. Arba Minch University: Ethiopia, p90 & 91.

Agha M. B., Bagcaci S.C. and Slark N. (2017). Homogeneity Analysis of Precipitation in North Iraq. IOSR Journal of Applied Geology and Geophysics (IOSR-JAGG), 59P.

Alzahrani S. (2017). Application of Two-Dimensional Hydraulic Modeling in River Systems Using HEC-RAS. M.Sc.Thesis. University of Dayton: Ohio, p37 to 61.

Amini A., Arya A., Eghbalzadeh A.& Javan M. (2017). Peak flood estimation under overtopping and piping conditions at vahdat Dam, kurdistan Iran. Arabian Journal of Geoscience, 12p.

Arakelyan L. (2018). Geghi Reservoir and Geghanoush Tailing Dam Break Studies. Yerevan, p10.

Australian Guideline for Flood Disaster Resilience. (2017). Attorney-General Department.Flood hazard Management. Australian Institute for Disaster Resilience: South Wales, p11.

AWAB. (2019). Ahferom Wereda Agronomy and Rural Development Agricultural. Agricultural Survey Report: Enticho, Ethiopia.

Bashar K. E. (2005). Micro Dams Nile Basin Capacity Building Network for River Engineering (NBCBN-RE) River Structures Research Cluster Group II. UNESCO-IHE: Sudan, Egypt, Ethiopia and Uganda.

Buchanan K. (2016). Consequence Assessment for Dam Failure Simulations Using HEC-FIA. Mapping, Modeling, and Consequences Center, p2-20.

Chow V. T., Maidment D. R. and Mays L. W.(1988). Applied hydrology. New York: McGraw Hill.

Chow V. T. (1959). Open-Channel Hydraulics. New York, Mc Graw-Hill Book Co., 680 p.

Christian H. (2014). Digital terrain models. Department of Spatial Sciences Curtin University Perth, WA, Australia Institute for Advanced Study. Springer International Publishing, TU Munich, Germany.

Colorado State Guidelines for Dam Breach. (2010). Department Of Natural Resources, Division of Water Resources. Office of the State Engineer Dam Safety: Colorado, 68p.

Cook S., Timberlake T., Murphy D. & Stahl J. (2015). Selection of dam breach inundation. The journal of dam safety Volueme 13, p48 to 51.

Daniel M. (2010). Lake Delhi Dam Breach - Two Perspectives. Lowa Department of Natural Resources, P1 to 2.

Deal C., Parr D. and Young B. (2017). A Comparison Study of One and Two-Dimensional Hydraulic Models for River Environments. Lawrence, Kansas: the University of Kansas No KS-17-02.

DEFRA, Welsh A., Halcrow, HR Wallingford and FHRC. (2004). Modelling and Decision Support Framework (MDSF). User Guide.

EASSSA. (2017/2018). Ethiopian Agricultural Sample Survey. Addis Ababa, Ethiopia.

Ekaningtyas T. R. (2017). Flood Inundation Prediction of Logung River due to the Break of Logung Dam. Journal of the civil engineering forum, Volume.3 and No.2, p61.

EMA. (2019). Meteorology Department. Hydro-Meteorological Data. Addis Ababa, Ethiopia.

ERA. (2002). Department of Hydrology. Ethiopian Roads Authority Drainage Design Manual. MECE/01: Addis Ababa, Ethiopia, p 4 to 22.

Faudzi1 M., Abustan I., Kadir A., Khairi M., Wahab A. & Razak A. (2019). Two Dimensional Simulation of Sultan Abu Bakar Dam Release Using HEC- RAS. Internation Journal of GEOMATE Volume16 and No. 58, p12.

FEMA. (2013). Federal Guidelines for Dan Safety (Hazard Potential Classification System For Dams. United States, Homeland Security: Federal Emergency Management Agency and Interagency Committee on Dam Safety, 145p.

Fread D. (1988). An Erosion Model for Earthen Dam Failures. National Weather Service NOAA, Silver Spring 20910, p2.

Froehlich D. (1995a). Embankment Dam Breach Parameters Revisited. Journal of Water Resources Planning and Management Volume 121 No 1, p 90 t0 97.

Froehlich D. (2008). Embankment Breach Parameters and Their Uncertainties. Journal of Hydraulic Engineering Volume 121/1, p1708 to 1721.

Hadush Fiseha. (2019). Dam Breach Analysis and Emergency Action Plan Case Study of Mihtsab Azmati Dam. MSc Thesis. Addis Ababa University: Ethiopia, 122p.

Haile Belay. (2018). Dam breach analysis the case study of Gidabo dam. M.Sc. Thesis. Hawassa University : Ethiopia, 71p.

Hayimanot Legisa. (2015). Dam Breach Modeling and Downstream Risk Analysis for Arjo-Dedessa Dam. M.Sc. Thesis. Addis Ababa University: Ethiopia, P135.

Hershfield D. (1995). Method for Estimating Probable Maximum Precipitation. Journal of American Water Works Association Volume 57, p965 to 972.

Jiregna Negusa. (2016). Dam Break Analysis The case studies of Fincha"a Dam. M.Sc.Thesis. Addis Ababa University: Ethiopia, 85 p.

Joshi M. and Shahapure S. (2017). Two Dimensional Dam Break Flow Study Using HEC-RAS for Ujjani Dam. International Journal of Engineering and Technology (IJET) Volume 9 and No 4, p1 to 3.

Juliastuti J. and Setyandito O. (2017). Dam breaks analysis and Flood inundation map of Krisak Dam For emergency action plan. American Institute of Physics Conference Preecidengs, p11.

Kreibich H., Piroth K., Seifert I., Maiwald H., Kunert U., Schwarz J., Merz B. and Thieken A.H. (2009). Is velocity a significant parameter in flood damage Modeling. Copernicus Publications on the behalf of European Union, p1 to 14.

Kumar S., Jaswal A., Pandey A. and Sharma N. (2017). Literature Review of Dam Break Studies And Inundation Mapping Using Hydraulic Models and GIS. International Journal of Research and Technology Volume 4 and No.5, P56.

Lehman W and Light M. (2016). Using HEC-FIA to identify Indirect Economic Losses. E3S Web of Conferences, 8p.

Liggett J. (2009/2010). Arizona"s Worst Dam Disaster from Walnut Grove Dam Failure (51662 US 60/89 ed.). Wickenburg, AZ 85390: Hassayampa, P36.

MacDonald T. and Langridge-Monopolis M. (1984). Breaching Characteristics of Dam Failures. Journal of Hydraulic Engineering Volume 110 No 5, p576 to 586.

Mahmut F., Dikbas F., Koc AC. and Gungor M. (2010). Missing data analysis and homogeneity test for Turkish Precipitation Series. Indian Academy of Science Volume 35, part 6, 14p.

Messner F., Penning R., Green C., Tunstall S. and Van der A. (2007). Evaluating Flood Damages: Guidance and Recommendations on Principles and Methods. FLOODsite Project Deliverable D9.1: Europe, Report No.06/01.

Mohammod A., Hamedi V. and Nazariha M. (2014). Flood damage Estimate (quantity), Using HEC-FDA Model a case study of Neka River. ELSEVIER Volume 10, Iran, p1173.

Motuma Shiferaw. (2015). Dam Break Analysis (The Case Study of Nashe Dam). M.Sc.Thesis. Addis Ababa University: Ethiopia, p88.

MoWIE. (2019). GIS department, Office Number 512. Ethiopian DEM. Addis Ababa: Ethiopia.

Nafari H. (2012). Flood damage assessment with the help of HEC-FIA model.MSc. Thesis. Politecnico Di Milano: P41 to 44.

Nebiyat Fekede. (2016). Flood Risk Analysis With Regards To Crop Yield in Upper Awash. Hawassa University: Ethiopia, p48.

Penning R., Johnson C., Tunstall S., Tapsell S., Morris J, Chatterton J., Coker A., Green C. (2003). The Benefits of flood and coastal defense: techniques and data for Flood Hazard Research Centre. Middlesex University, P60.

Pettit A. (1979). A non- parametric approach to the change-point problem. Appl Statist Volime 28, p126 to 135.

Quiroga V., Kure S., Udo, K., & Mano, A. (2016). Application of 2D Numerical Simulation For The Analysis of the Bolivian Amazonia Flood. ,Volume 3, P25-33.

Salas D., Gavilan G., Salas R., Julien Y. & Abdullah J. (2014). Uncertainty of PMP and PMF. California: State of Colorado University Libraries.

Samuel G. (1989). Backwater length in rivers, proceedings-Institution of Civil Engineers. Part 2, Research and Theory of 87, P571 to 582.

Schulze R. E., Schmidt, E. J., and Smithers, J. C. (1992). Pietermaritzburg: Department of Agricultural Engineering. SCS-SA User Manual PC Based SCS Design Flood Estimates for Small Catchments in Southern Africa. University of Natal.

SCS-USDA. (1986). Department of Agriculture, Simplified Dam Breach Routing Procedure. Technical Release No. 66.

Sorensen H. J. (1991). Factors affecting the timing of evacuation Departures. International Journal of Mass Emergencies and Disasters Volume 9 No. 2, p153 to 165.

Strahler A. (1957). Quantitative Analysis of Watershed geomorphology. Transactions of American Geo physical Union Volume 38, p913 to 920.

Tariku Tadesse. (2015). Dam Break Analysis and Risk Assessment case study of Tendaho dam. M.Sc. Thesis. Addis Ababa University: Ethiopia, 135p.

Tesfa Kefale. (2016). Dam breach modeling and inundation mapping, a case study on Gidabo Dam. M.Sc.Thesis. Addis Ababa University: Ethiopia, 119p.

Tesfay Haile. (2019/20). Dam breach Analysis and Downstream inundation the case study of Midimar Dam. M.Sc. Thesis. Mekelle University: Ethiopia, p9.

Thornton C. (2016). 2D HEC-RAS Model Development in Data Poor Areas of India the case study.M.Sc. Thesis. Tennessee Technological: India, p48.

TWRB. (2016). Water supply Section. Feasibility study of Gerhu-sirnay dam. Project Guide: Tigray, Ethiopia.

TWRB. (2019). Feasibility study of Gerhu-sirnay dam Water supply Project. Design Document: Mekelle, Ethiopia.

TWRB. (2019). Gerhu-sirnay dam site supervision during construction. Tigray, Ethiopia.

TWWCE. (2019). Gerhu-Sirnay Dam Water Treatment Plant Design Report. Project Guide: Mekelle, Ethiopia.

US Bureau of Reclamation. (1988). Simplified inundation maps for emergency action plan. US: p5 and 27.

US Bureau of Reclamation. (2007, March 2). US: Dam Failure Wikipedia. Retrieved Julay 13, 2019, from Wikipedia Web Site: htt://www. Dam Failure Wikipedia.com

USACE. (2012). HEC-FIA Version 2.2 User's Manual for Flood Impact Analysis. Hydrological Engineering Center: California,354p.

USACE. (2016). HEC-RAS, 2D Modeling User's Manual for Dam Break Studies. Hydrological Engineering Center: California, 171p.

Von Thun L. and Gillette R. (1990). Guidance on Breach Parameters. U.S. Department of the Interior Bureau of Reclamation.

Wahl L. (1998). Prediction of Embankment Dam Breach Parameters. U.S. Department of the Interior Bureau of Reclamation Dam Safety Office: United State, 67p.

Wehrheim J. (2006). Engineering Assessment of Breaching Failure of Ka Loko Dam. Star-Bulletin 11, p35.

Wijesekera T. and Perera H. (2012). Study on Key Issues of Data and Data Checking for Hydrological Analyses (case study of Attanagalu Oya Basin of Sri Lanka). xxxxv: p2.

WMO. (1986). Manual for Estimation of Probable Maximum Precipitation. WMO No. 332. Geneva, Switzerland.

Yakti P., Adityawan B., Farid M., Suryadi Y. and Nugroho J. (2018). 2D Modeling of Flood Propagation Due to the Failure of Way Ela Natural Dam. MATEC Web of Conference 147 03009. Bandung IOT: Indonesia, p4 to 5.

Yonatan Sisay. (2016). Dam Breach Analysis & Inundation Map for MelkaWakenaDam. M.S.c.Thesis. Addis Ababa University: Ethiopia, 82p.

Zagonjolli M. (2007). Dam Break Modeling, Risk Assessment and Uncertainty Analysis for Flood Mitigation. Ph.D. Thesis. Delft University: the Netherlands, p1.

APPENDIXES

Appendix A: Original Ground Level of the Dam Site

S No.	Chain age (m)	OGL(m)	S No.	Chain age (m)	OGL(m)
1	0	1839	21	482.17	1820.27
2	15	1839	22	484.47	1819.49
3	18	1839	23	493.23	1817.51
4	18	1836	24	501.42	1818
5	76	1836	25	558.29	1818
6	76	1839	26	567.8	1822.29
7	100	1839.9	27	568.1	1824.35
8	200	1842	28	565.9	1825
9	300	1844.4	29	564.32	1825.54
10	375	1845	30	564.1	1828.33
11	411	1842	31	568.89	1831.27
12	416	1840.88	32	573.69	1833.58
13	431	1837.01	33	567.73	1830.81
14	435.79	1835.31	34	581	1837.01
15	440.01	1833.05	35	591	1839
16	445.39	1828.87	36	596	1842
17	452.2	1828.01	37	601	1842.3
18	461.38	1825.87	38	621	1842
19	471.73	1822.73	39	624.3	1842.3
20	477.58	1821.1			

Appendix B: Constant Number Kn, for a Function of Precipitation Series Size, n

Sample Size, n	Kn	Sample Size, n	Kn	Sample Size, n	Kn	Sample Size, n	Kn	Sample Size, n	Kn
10	2.036	21	2.408	32	2.591	43	2.71	70	2.893
11	2.088	22	2.429	33	2.604	44	2.719	75	2.917
12	2.134	23	2.448	34	2.616	45	2.727	80	2.94
13	2.175	24	2.467	35	2.628	46	2.736	85	2.961
14	2.213	25	2.486	36	2.639	47	2.744	90	2.981
15	2.247	26	2.502	37	2.65	48	2.753	95	3
16	2.279	27	2.519	38	2.661	49	2.76	100	3.017
17	2.309	28	2.534	39	2.671	50	2.768	110	3.049
18	2.335	29	2.549	40	2.682	55	2.804	120	3.078
19	2.361	30	2.563	41	2.692	60	2.837	130	3.104
20	2.385	31	2.577	42	2.7	65	2.866	140	3.129

Appendix C: Reservoir Capacity Detail

S No.	Elevation (m)	Depth(m)	Area		D volume	S Volume	Capacity
			(m^2)	(ha)	(m^3)	(m^3)	(Mm^3)
1	1818	0	30.5	0	0.00	0.00	0.00
2	1819	1	627	0.06	265.26	265.26	0
3	1820	2	1699.5	0.17	1119.59	1384.85	0
4	1821	3	3335	0.33	2471.74	3856.59	0
5	1822	4	5385	0.54	4319.27	8175.86	0.01
6	1823	5	7734	0.77	6524.16	14700	0.01
7	1824	6	11356.1	1.14	9487.25	24187.3	0.02
8	1825	7	14956	1.5	13114.81	37302.1	0.04
9	1826	8	19293	1.93	17078.55	54380.6	0.05
10	1827	9	24122	2.41	21662.6	76043.2	0.08
11	1828	10	28835	2.88	26443.48	102487	0.1
12	1829	11	33987	3.4	31375.73	133862	0.13
13	1830	12	40016	4	36960.5	170823	0.17
14	1831	13	46147	4.61	43045.1	213868	0.21
15	1832	14	52156	5.22	49120.86	262989	0.26
16	1833	15	58905	5.89	55496.29	318485	0.32
17	1834	16	65708	6.57	62275.53	380761	0.38
18	1835	17	73141	7.31	69391.32	450152	0.45
19	1836	18	79044	7.9	76073.41	526225	0.53
20	1837	19	85374	8.54	82188.68	608414	0.61
21	1838	20	104702	10.47	94873.55	703288	0.7
22	1839	21	116639	11.66	110616.61	780817	0.78
23	1840	22	129268	12.93	122899.42	875999	0.88
24	1841	23	142588	14.26	135873.48	976673	0.98
25	1842	24	156599	15.66	149538.79	1082839	1.08

Appendix D: Annual maximum daily rain fall (X mm) of Gerhu-sirnay station (1998-2018)

Appendix i: Detail for outlier, adequacy and reliability tests

Year	X	Descending order (X)	Rank	Y=log Xi	
1998	93.9	93.9	1	1.97	
1999	75.7	75.7	2	1.88	
2000	15.1	60.1	3	1.78	
2001	33.4	55.4	4	1.74	
2002	45.1	46.5	5	1.67	
2003	32.5	45.5	6	1.66	
2004	55.4	45.1	7	1.65	
2005	27.7	40.0	8	1.60	
2006	60.1	37.4	9	1.57	
2007	45.5	34.8	10	1.54	
2008	31.0	33.4	11	1.52	
2009	40.0	32.5	12	1.51	
2010	46.5	31.0	13	1.49	
2011	37.4	30.7	14	1.49	
2012	29.1	29.1	15	1.46	
2013	30.7	27.7	16	1.44	
2014	22.5	24.5	17	1.39	
2015	24.5	22.5	18	1.35	
2016	20.4	21.0	19	1.32	
2017	21.0	20.4	20	1.31	
2018	34.8	25.1	21	1.40	
Sum		832.2		32.8	Parameters for outlier from Y
Mean, X_m		39.6		1.6	
Standard Deviation, δ_n		18.71		0.18	
C_S		1.60		0.69	
No. of data, N		21			Parameters for Adequacy and reliability from X
Standard Deviation, δ_n		17.87			
Standard error of mean, S_e		3.9			
Relative Standard,δ_e		9.84 %			

Appendix ii: Detail for homogeneity test by using Pettitt (1979)

Year	X	Rank	Vi	Ui	Year	X	Rank	Vi	Ui
1998	549.2	1	20	20	2009	291.2	12	-2	108
1999	787.6	2	18	38	2010	410.1	13	-4	104
2000	382.6	3	16	54	2011	468.7	14	-6	98
2001	516.0	4	14	68	2012	553.9	15	-8	90
2002	490.3	5	12	80	2013	331.4	16	-10	80
2003	480.7	6	10	90	2014	169.3	17	-12	68
2004	504.1	7	8	98	2015	200.2	18	-14	54
2005	407.2	8	6	104	2016	256.6	19	-16	38
2006	649.7	9	4	108	2017	613.4	20	-18	20
2007	694.7	10	2	110	2018	766.1	21	-20	0
2008	442.3	11	0	110					

Appendix iii: Annual precipitation of Gerhu-sirnay station (1998-2018) after data quality tests

Year	Gerhu-Sirnay	Year	Gerhu-Sirnay
1998	549.1	2009	600.4
1999	787.6	2010	310.8
2000	382.6	2011	508.6
2001	516.0	2012	430.3
2002	490.3	2013	482.1
2003	480.7	2014	358.0
2004	504.1	2015	446.1
2005	407.2	2016	554.0
2006	482.7	2017	313.4
2007	401.3	2018	563.6
2008	298.8		

Appendix E: SCS Method Parameters Detail

Appendix i: Inflow Hydrograph Parameters

S No.	Parameters	Symbol	Unit	Value
1	Watershed area computation using Arc Swat software(watershed delineator)	A	Km^2	14.27
2	Longest flow path measured by watershed delineator tool	L	m	4400
3	Level of head of the main water level at the upper point	E_2	m a.m.s.l	2,053
4	Level of head of the main water level at the lower point/watershed outlet	E_1	m a.m.s.l	1817.50
5	Slope of the main water course	S	m/m	0.05
6	Time of concentration, T_c=1/3000*(L/(S)0.5))0.77	T_c	hrs.	0.84
7	Rain fall excess duration, D=T_c/6 ; if T_c<3hrs & D=1hr if T_c>3hrs	D	hrs.	0.14
8	Time to peak, T_p=0.5D+0.6T_c	T_p	hrs.	0.57
9	Time base of Hydrograph, T_b= 2.67T_p	T_b	hrs.	1.53
10	Lag time , T_L= 0.6T_C	T_L	hrs.	0.50
11	Peak rate of discharge created by 1mm run off excess of the whole catchments Q_p=(0.21A)/T_p	Q_p	m^3/s/mm	5.23

Appendix ii: Detail for weighted CN

Land use types	**Hydrologic condition**	**Area, ha**	**Soil types**	**HSG**	**CN (AMC)**	**A*CN**	**CN= ∑A*CN/∑A**
Crop land	Good	899.01	Haplic Xerosols	B	86	77314.9	
Open grass land with sparse shrubs	Fair	527.99	Orthic Solonchaks	B	80	42239.2	
Total		1427				119554	**83.78**

Appendix iii: Inflow Hydrograph Ordinates

S. No.	Time, hrs	Q (m^3/s)	S.No.	Time, hrs	Q (m^3/s)
1	0	0	10	0.99	329.1
2	0.14	0.4	11	1.13	316.8
3	0.28	3.7	12	1.27	264.6
4	0.42	35.9	13	1.53	168.5
5	0.56	80.2	14	1.67	116.5
6	0.57	87.3	15	1.81	66.2
7	0.70	154.8	16	1.95	33.3
8	0.71	208.8	17	2.09	7.5
9	0.85	263.8	18	2.23	0

Appendix F: Manning Roughness regions (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy. deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. **Flood Plains**			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. **Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged**			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

Appendix G: Breach Outflow Ordinates for Von Thun and Gillette (1990)

S no.	Overtopping Scenario		Piping Scenario	
	Simulation Time	Flow (m^3/s)	Simulation Time	Flow (m^3/s)
1	9-Jul-19-0:01:30	0	12-Jul-19-0:00:04	8.08
2	10-Jul-19-0:05	220.77	12-Jul-19-0:05	102.31
4	10-Jul-19-0:15:17	630	12-Jul-19-0:14	304.23
5	10-Jul-19-0:20	834.62	12-Jul-19-0:20	409.23
6	10-Jul-19-0:24	1033.85	12-Jul-19-0:24	506.16
7	10-Jul-19-0:30	1248.46	12-Jul-19-0:30	609.898
8	10-Jul-19-0:34	1206.15	12-Jul-19-0:35	535.77
9	10-Jul-19-0:40	1157.69	12-Jul-19-0:39	471.15
10	10-Jul-19-0:43	1130.77	12-Jul-19-0:45	398.46
11	10-Jul-19-0:49	1071.54	12-Jul-19-0:50:00	331.15
12	10-Jul-19-0:54	1028.46	12-Jul-19-0:55	258.46
14	10-Jul-19-1:04	861.54	12-Jul-19-1:05	166.92
15	10-Jul-19-1:10	739.67	12-Jul-19-1:10	148.08
17	10-Jul-19-1:20	495.38	12-Jul-19-1:20	102.31
19	10-Jul-19-1:26	350.41	12-Jul-19-1:26	74.35
21	10-Jul-19-1:28	302.57	12-Jul-19-1:28	66.07
22	10-Jul-19-1:29:01	279.38	12-Jul-19-1:30	61.18
23	10-Jul-19-1:30	254.93		

Appendix H: Impact Area Data Details for HEC-FIA (TWRB, 2007)

Appendix i: Water Treatment Plant Location and Classification

Structure type	Location		Classification				
	Easting	Northing	Foundation Depth (ft)	Inundation Depth (ft)	Depth (ft)	% of damage	Damage (Birr)
Settling basin	516969.7	1599294.1	3.3	19.7	16.4	72.4	1683.78
10 m^3 RCC reservoir	516985.7	1599287.3	1.0	25.3	24.3	100	2481.36
100 m^3 RCC reservoir	516966.8	1599289.2	3.6	26.2	22.6	84.9	2227.32
200 m^3 RCC reservoir	516973.6	1599273.1	2.3	7.3	5.0	33	974.82
Septic tank	516946.6	1599293.5	0.0	21.8	21.8	81.9	2102.81
Chlorinator house	516991.3	1599267.5	5.9	6.8	0.9	12.7	257.00
Generator house	516995.1	1599258.1	18.0	26.5	8.5	49.7	1527.22
Pump house	516996.0	1599250.7	4.6	7.3	2.7	22.6	670.56
Operators' bed and office	517001.9	1599186.3	5.9	3.6	-2.3	0	70.90
Air valve and wash out	516994.5	1599243.1	1.2	3.3	2.1	17.9	528.77
River crossing	516970.9	1599173.3	1.0	3.3	2.3	19.2	567.17
Aerator house	516945.8	1599282.9	1.6	1.4	-0.3	0	186.10
Guard house 1	516945.8	1599275.5	1.0	16.5	15.5	68.6	2496.13

Guard house 2	516964.1	1599272.5	1.0	16.2	15.2	67.6	2451.82
Guard house 3	516950.2	1599288.7	1.0	9.0	8.0	48	1417.92
Guard house 3	516927.6	1599187.7	1.0	9.0	8.0	48.9	667.60
SSF house 1	516985.4	1599180.1	2.0	18.0	16.0	71.2	1654.24
SSF house 2	516994.2	1599181.9	2.0	18.0	16.0	71.2	1654.24
Fence	517007.6	1599259.1	2.3	4.3	2.0	17	502.18
Impact area	516964.2	1599272.6	0.0	4.3	4.3	29	856.66

Appendix ii: Structure Values and PAR

Structure (1000 Birr)	Content (1000 Birr)	Other (1000 Birr)	Car (10^3 Birr)	C	A	G	E	No. of Structures	Car No.	Total PAR
2682.59		1759.96						1		
2682.59		1759.96						1		
2682.59		1759.96						1		
2682.59		1759.96						1		
2682.59		1759.96						1		
2682.59	1541.82	1759.96			4		4	1		
2682.59	1035.05	1759.96			2		2	1		
2682.59	453.98	64835.87			3		3	1		
2682.59		1759.96	3691.32		15		15	1	2	
2682.59		1759.96			2		2	1		
2682.59		1759.96						1		
2682.59		1759.96			3		3	1		
2682.59		1759.96		1			1	1		
2682.59		1759.96		1			1	1		
2682.59		1759.96		1			1	1		
2682.59		1759.96		1			1	1		
2682.59		1759.96						1		
2682.59		1759.96						1		
2682.59		1759.96		2			2	1		
		1759.96	2801.62					1	2	**70**

NB: The representations of letters A, C, E and G were the same with table 4-8.

Appendix iii: Car Detail

Car Types	Car Height (m)	ETB
Pick Up Double Cabin (Moenco Vehicles), 2 cars	1.96	3,691,318.54
Ambulance	1.96	1,051,624
Tata Star bus, service with more than 66 seats	3.2	1,750,000
		Total Cost = 6,492,942.00

Appendix iv: Structure Elevation and Address

	Elevation (ft)		**Address**	
ID	**Ground elevation (ft)**	**Depth difference in cars (ft)**	**Structure Number**	**City**
34	10.8		G-001	Gerhu-Sirnay
35	10.8		G-002	Gerhu-Sirnay
36	33.5		G-003	Gerhu-Sirnay
37	44.6		G-004	Gerhu-Sirnay
64	17.2		G-005	Gerhu-Sirnay
6	10.8		G-006	Gerhu-Sirnay
7	12.1		G-007	Gerhu-Sirnay
8	20.7		G-008	Gerhu-Sirnay
9	11.8	0	G-009	Gerhu-Sirnay
10	26.2		G-010	Gerhu-Sirnay
11	19.7		G-011	Gerhu-Sirnay
12	34.8		G-012	Gerhu-Sirnay
13	9.8		G-013	Gerhu-Sirnay
14	9.8		G-014	Gerhu-Sirnay
47	9.8		G-015	Gerhu-Sirnay
16	9.8		G-016	Gerhu-Sirnay
17	11.8		G-017	Gerhu-Sirnay
62	9.2		G-018	Gerhu-Sirnay
50	6.9		G-019	Gerhu-Sirnay
29	10.5	4.1	G-020	Gerhu-Sirnay

Appendix v: Machine Detail

Machine group	Quantity and brand	Description	Power (KW)	HP	Cost (Birr)	Total Cost
Back Wash pump	2 (BW, GE-2600A)	Q=27.13 l/s, H=120 m	53229.06	72391.52	64835.87	129671.74
Main pump	3(Bosworth,GH-2600B 11/4F	Q=10 l/s, H=235 m	38422.5	52254.6	21656.07	64968.21
	4 (BW, GE-2600A)	Q=27.13 l/s, H=120 m	53229.06	72391.52	64835.87	259343.48
Dosing pumps	2(for 69.26 l/s)	118, 800 LPH	266114235	361915359.6	770910.34	1541820.69
Generator	1(28.41696 ft^2 seat area)		325.522	442.7099		35038.9

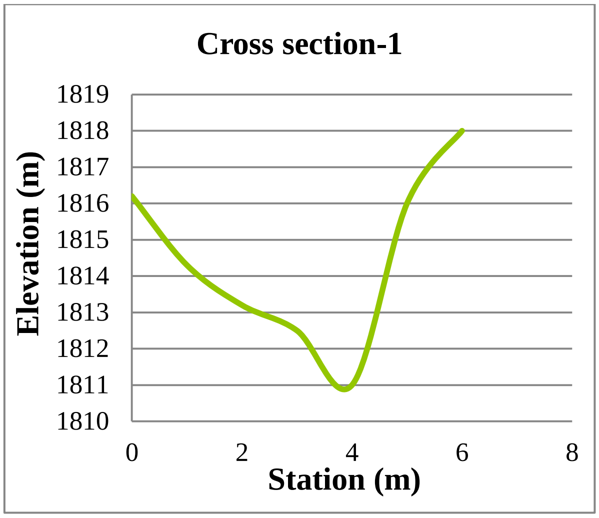
Appendix vi: Water Treatment Plant Attributes

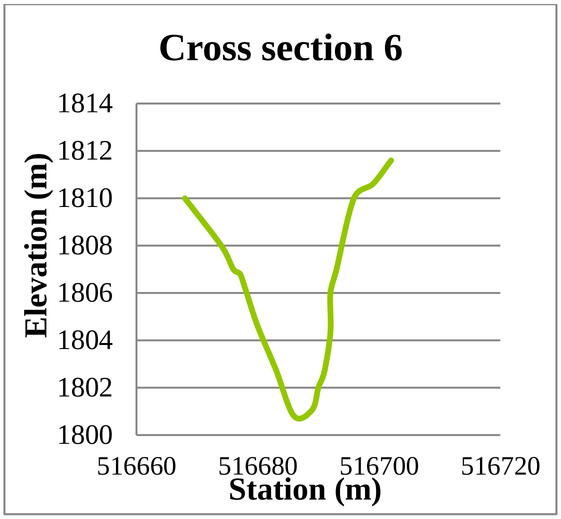
ID	Construction Type		Basic Parameters		
	Exterior Wall	**Foundation Type**	**Quality**	**Floor Area (ft^2)**	**Basement Area (ft^2)**
34	Hard Board Sheets/Masonry	Basement	Good	828.8	947.01672
35	Hard Board Sheets/Masonry	Basement	Good	221.5	150.1
36	Hard Board Sheets/Masonry	Basement	Good	438.3	438.3
37	Hard Board Sheets/Masonry	Basement	Good	654.7	654.7
64	Hard Board Sheets/Masonry	Basement	Good	142.1	239.6
6	Hard Board Sheets/Masonry	Basement	Good	703.5	703.5
7	Hard Board Sheets/Masonry	Basement	Good	430.1	430.1
8	Hard Board Sheets/Masonry	Basement	Good	653.6	653.6
9	Hard Board Sheets/Masonry	Basement	Good	532.2	532.2
10	Hard Board Sheets/Masonry	Basement	Good	871.9	911.1
11	Hard Board Sheets/Masonry	Basement	Good	40.9	245.4
12	Hard Board Sheets/Masonry	Basement	Good	981.2	981.2
13	Hard Board Sheets/Masonry	Basement	Good	127.9	127.9
14	Hard Board Sheets/Masonry	Basement	Good	127.9	127.9
47	Hard Board Sheets/Masonry	Basement	Good	127.9	127.9
16	Hard Board Sheets/Masonry	Basement	Good	127.9	127.9
17	Hard Board Sheets/Masonry	Basement	Good	2124.5	2310.5
62	Hard Board Sheets/Masonry	Basement	Good	645.8	753.0
50	Metal Frame	Basement	Good	0	79888.9
29	Metal Frame	Basement	Good	0	79888.9

Appendix vii: Impact Area Crop Data Detail

S no.	Crop Type	Crop land Area, ha	Area (acre)	Percent of Coverage
1	Sorghum	45.95	4595	67.46
2	Barley	10.42	1042	15.29
3	Millet	11.75	1175	17.25

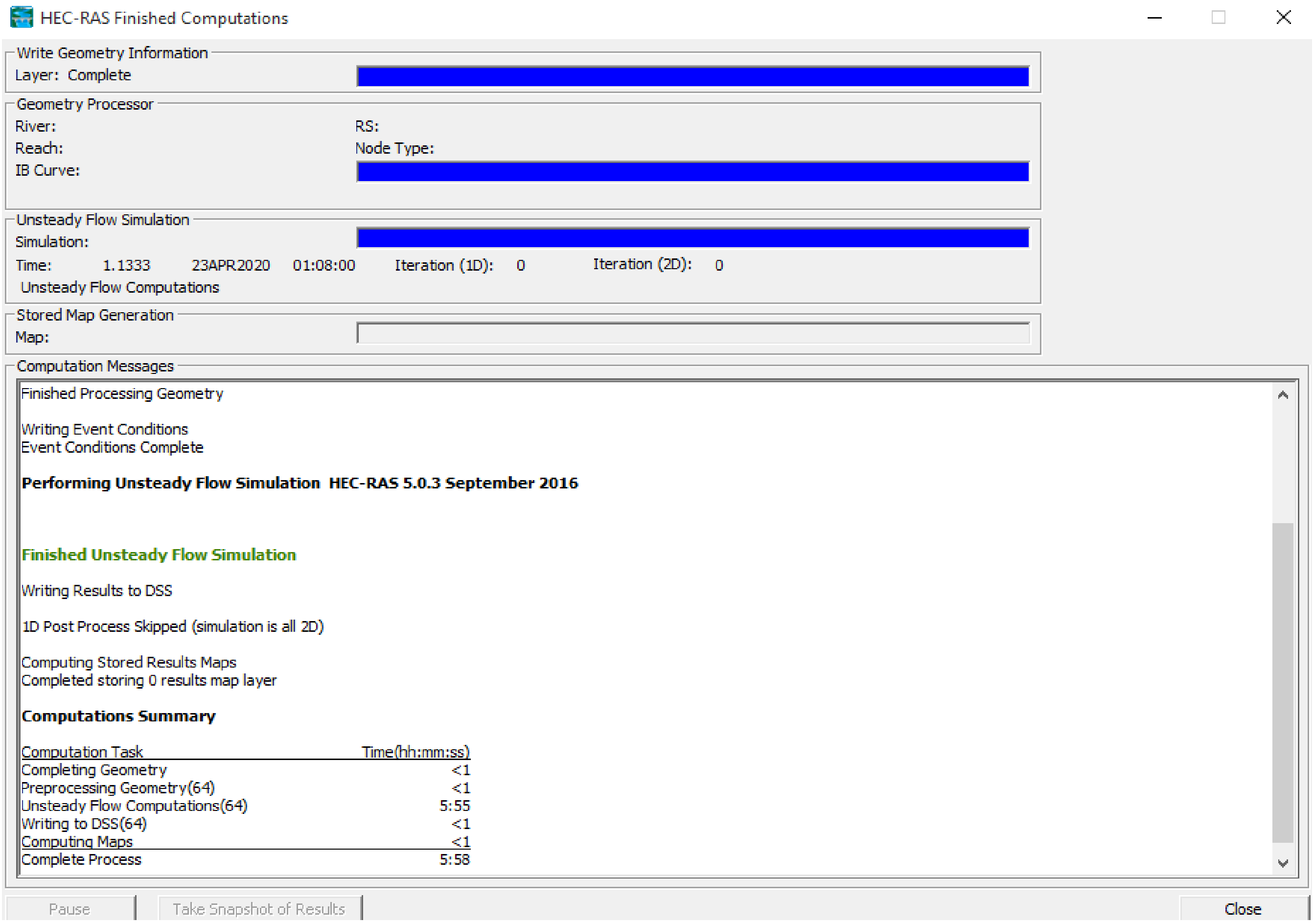
Appendix I:Sample River Cross Sections





Appendix J: Photos for Collecting River Cross Section

Appendix K: Unsteady flow analysis Computation message (Worst scenario)



Appendix L: Flood inundation result in RAS mapper (Worst scenario)