



Addis Ababa University
Addis Ababa Institute of Technology

School of Graduate Studies
School of Civil and Environmental Engineering

Dam Breach Analysis and Emergency Action Plan
Case Study of Mhtsab Azmati Dam

A thesis Submitted to the School of Graduate Studies of the Addis Ababa Institute of Technology in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering Major in Hydraulic Engineering

By

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April, 2019

Addis Ababa, Ethiopia

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Dr.Ing. Asie Kemal (Advisor)

Date

Declaration

I declare that I have elaborated my M.Sc. thesis aimed at “**Dam Breach Analysis and Emergency Action Plan: Case Study of Mhtsab Azmati Dam**” independently under the leadership of my advisor, Dr. Ing. Asie Kemal. I have already used only the literature and other information sources that were cited in the work and listed in reference at the end of this work. As the author of the thesis, further I declare that I am related to its creation and did not infringe the copyright of the third parties.

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Abstract

This study mainly describes the different dam failure modes which can affect the downstream societies and property in the world wide and was aimed to analyze the over topping and piping mode of dam failures for the Mhtsab Azmati embankment dam due to their highly occurrence from the others. The hydrological assessment, the dam breach dimensions, the downstream flood hydrographs, the development of the inundation map and the emergency action plan were among the specific tasks done in this paper. The meteorological data, physical features of the dam, DEM, land use and land cover and stream flow from other gauged river were the data collected for this study. The hydrologic model HEC – HMS version 4.2.1, the regression equations MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990), the hydraulic model HEC – RAS version 5.03 and the Arc GIS tools HEC – GeoHMS and HEC – GeoRAS were also the methods which used for the analysis. The probable maximum flood (PMF) with its peak run off $833.33 \text{ m}^3/\text{sec}$ from the catchment was found and used as input for the unsteady flow simulation in the HEC- RAS model. The dam breach model was analyzed for both overtopping and piping mode of failures using the four methods of dam breach regression equations for each cases. Depending on the maximum probable flood generated from the catchment and the reservoir capacity, the result obtained using MacDonald and Langridge Monopolis (1984) for the overtopping mode of failure case was selected to the downstream flood analysis with the results of side sloe (0.5H: 1V), bottom breach width 15m, breach development time 1.18hr and peak flood of $727.2 \text{ m}^3/\text{sec}$. The downstream inundation map of the flooded were also for the identification of the more risked area. An integrated emergency action plan was also provided by this paper in order to safeguard the downstream population and conclusion and recommendation were also included in this study.

Key words: Dam breach modeling, DEM, ArcGIS, HEC – GeoRAS, PMP, PMF, HEC – HMS, HEC – RAS, Inundation map and EAP

Acknowledgement

I would like to express gratefulness thank to my advisor Dr.-Ing. Asie Kemal for his continuous encouragement and supportive advice during my study period. His critical advices and comments were critically useful on my progress work and I appreciate his approach and way of understanding the problems in my paper work.

I would like to acknowledge my sponsor, Ministry of Education for its financial support and providing higher education opportunity for me and many other students. Also, my thank goes to Addis Ababa University, Addis Ababa Institute of Technology (AAIT) for accepting and providing the Master Programm.

In addition also I would like to acknowledge the Ministry of Water, Irrigation and Electricity of Ethiopia, National Meteorological Agency of Ethiopia and Tigray Water Resource Bureau for their heart full helping me during the data collection time.

I would like to thank all my colleague and friends for giving me the strength to complete this paper and for supporting me any access regarding this task.

Lastly but not least, I would like to thank my families for their endless loving and supporting me in any aspect of my life.

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Abbreviations

1D/2D – One Dimensional/ Two Dimension

AnnAGNPS – Annualized Agricultural Non-point Source Model

Arc GIS – Geographic Information System

DEM – Digital Elevation Model

DHI – Danish Hydraulic Institute

EAP – Emergency Action Plan

FEMA – Federal Emergency Management Agency

GPS – Global Position System

GSSHA – Gridded Surface Subsurface Hydrologic Analysis

HEC – Hydrologic Engineering Center

HEC – GeoRAS - Hydrologic Engineering Center – Geomorphological River Analysis System

HEC – HMS - Hydrologic Engineering Center - Hydrologic Modeling System

HEC – RAS - Hydrologic Engineering Center's River Analysis System

HYPE – Hydrological Predictions for the Environment

IDF – Inflow Design Flood

Km – kilo meter

m – meter

masl – Mean above Sea Level

mm - mil meter

Mm³ – Million cubic meter

No – Number

NWS – National Weather Services

PMF – Probable Maximum Flood

PMP – Probable Maximum Precipitation

SCS – Soil Conservation Service

SDF – Standard Design Flood

S.I.U – System International Units

SMP – Surveillance and Monitoring Plan

Sq.km – square kilometer

St. No – Station Number

USA – United States America

UTM – Universal Transected Meter

Chapter 1

1. Introduction

1.1 Background

Dams are hydraulic structures which are constructed across the river to impound water in its upstream for different purposes like to supply water to irrigation, hydropower generation, domestic and industrial water supply, multipurpose reservoir serves as flood control and drought mitigation structures and storage place for arresting the sediment (Kumar et al., 2017).

In spite of many uses of the dams and reservoir, the disaster caused by the dam failure creates damages higher than the damages caused by the other disaster all together. Dam breach can be described as partial or catastrophic failure leading to uncontrolled release of enormous flood to the downstream (Fread, 1993). Flood generated from the dam breach is much higher than flood generated from the heavy precipitation. There are a number of dam breach causes and breach due to seepage, overtopping, foundation failure, failure due to poor construction quality in case of materials and procedure are among them. Failure of the dams may be gradual or instantaneous. Concrete dam generally fails by sudden failure due to overtopping and earthen dam fails due to erosion of material and piping action (Kumar et al., 2017).

Even though latest technologies in design methodologies and construction techniques were developed, failure of dams still occurs. Study conducted on various failure of dams occurred in India and all over world indicated the different hazard caused by the dam failures. Hence emphasis to be given on better management of the flood by preparing emergency action plan to minimize the hazards of the flood in the floodplain rather than prevention of the flood. It is necessary to analyze the behavior of the flood before suggesting flood management measures. This can be done by analyzing the flood based on the observed floods, numerical model simulation of dam breach and flood event based on the various parameters (Kumar et al., 2017).

Breach is defined as the opening formed in the dam body that causes the water to spread to the downstream location. Different Case studies show that dam failure may arise due to different reasons ranging from seepage, piping, overtopping due to insufficient free board and settlement due to side slopes on the upstream shells and liquefaction due to earthquakes (Xiong, 2011).

The study of dam breach is vital for disaster management as well as for development of plans in the vicinity of the dam. Dam breach modelling helps in making preparedness plans, issue of emergency warnings and planning downstream development. Dam breach modelling involves the

determining the dam breach parameter dimensions, determining the outflow hydrograph and the peak discharge, routing the peak discharge and prediction of hydrograph at different river cross section stations downstream up to the point of consideration on the river and mapping of inundation levels (Nema and Desmukh).

Currently, the exploitation of irrigation and hydropower potentials has been recognized by the Ethiopian government as key issues in the economic development of the country. To meet the strong increase of food and energy demands for the future, the Ethiopian government has made large-scale investments of large to small dam projects like Mhtsab Azmati irrigation embankment dam which is located in the Northern Ethiopia which is among the tributary of the Mereb river basin around 23km north of Adwa town. This dam is planned to store surface-runoff during the rainy season and used for supplementary irrigation during the wet season to fulfil the shortage of rain and for dry season irrigation purposes.

1.2 Statement of problem

Dams are considered "installations containing dangerous forces" under International Humanitarian Law due to the massive impact of a possible destruction on the civilian population and the environment. Dam failures are comparatively rare, but can cause immense damage and loss of life when they occur. In 1975 the failure of the Banqiao Reservoir Dam and other dams in Henan Province, China caused more casualties than any other dam failure in history. The disaster killed an estimated 171,000 people and 11 million people lost their homes.

https://en.wikipedia.org/wiki/Dam_failure.

Dams store large amount of water but due to lack of understanding of hydrological parameters, capacity of the reservoir and spillway, the structural failure of materials used in dam construction, movement and/or failure of the foundation supporting the dam, settlement and cracking of concrete or embankment dams, piping and internal erosion of soil in embankment dams and inadequate maintenance of an embankment dam breach is the most challenging in most countries in the world. Such uncontrolled or excessive release of a huge amount of water has great potential for loss of life and damage to property in the downstream areas due to excessive flooding. It is, therefore, necessary to have a thorough and consistent planning for any such eventuality to help save lives and reduce property damage in areas that would be affected by dam failure or operation and putting in place action plans to cope with such an emergency.

Human encroachment into floodplains and increase of damage potential appear to be the major causes for increased flood-related damages in most areas. For instance, in Japan half the total population and about 70% of the total assets are located on flood plains, which cover only about 10% of the land surface. An important factor influencing the flood risk is an unjustified belief in the absolute security provided by structural defenses. However, even a perfectly maintained dike designed to withstand, for example, a 100-year flood (river flow with probability of exceedance in any given year being one in 100 years, i.e. 0.01) does not, by definition, guarantee absolute protection. It is likely to be overtopped and possibly breached by an extreme flood (e.g. one with return period of 1,000 years). When a dike breach, the damage may be greater than it would have been in a levee-free case, since existence of a dike is taken as a safety guarantee by the riparian population, so that the damage potential grows very strongly. Further, a short memory syndrome can be observed. During a flood-free interval, decision makers gradually reduce the funding of flood preparedness systems, and citizens become increasingly less risk-aware. This occurs, for river and coastal flooding, in developing and developed countries alike, including the United States, where the Katrina event unveiled the general inadequacy of the emergency preparedness system (Zbigniew W et al., 2016).

Nowadays, Ethiopia is developing a planning and strategies regarding the water resource management by constructing different hydraulic structures in different rivers which have a great role on storing of the water during the rainy season and using at the dry time for different purposes like hydropower generation, irrigation, water supply etc. Among these hydraulic structures an embankments dam are the commonly used for storing of water for irrigation activities to increasing the agricultural crop production at different country sides.

Mhtsab Azmati dam is zoned embankment dam, which was constructed by Tigray Water Resource Bureau and designed to irrigate around 2000 ha. There is a town called Rama located at the downstream of the dam 12km far apart with total population of 7,824. Currently, the dam has an observable of high seepage with high flow velocity, high flow rate and turbulent flow over the downstream face of the dam aside one abutment and above the irrigation outlet pipe. It is a critical issue as the existed seepage is the one which is at the location of unexpected point and this condition may lead to the probability of occurrence of dam breach due to piping. Therefore, it is sensitive to analysis the downstream damages caused by the dam breach and providing

recommendation of an emergency action plan which can be used to reduce its downstream impact on life, property damage and different structural facilities of the community considering the volume of water, water level, velocity and arrival time of flood wave propagation that could be developed from dam to downstream area.

1.3 Objective of the study

1.3.1 General objective

The general objective of the study was to analyze the dam breach using hydraulic models and Arc GIS and providing an appropriate emergency action plans for case study of Mihtsab Azmati dam.

1.3.2 Specific objectives

- To assessment hydrological data for the Mhtsab Azmati catchment using HEC- HMS model
- To determine dimensions of dam breach parameters
- To compute downstream flood hydrographs after the dam was breach by both overtopping and piping mode of failure
- To develop an inundation map of downstream area
- To provide an appropriate emergency action plan

1.4 Significance of the Study

The studying of the Mhtsab Azmati dam breach analysis helps to give effective and urgent action plan that requires accurate prediction of inundation levels. Thus the results of the study were important for the following reasons:

- The research result can be used to inform the flood prone area coverage to the downstream population if the dam breaches.
- The research result can be used to notify the effect of dam breach on the human life, irrigated area, constructed infrastructures and other developed economic activity.
- The research outcome can be used to develop emergency action plan for the Mhtsab Azmati dam based on the flood inundation map developed and the potential risk that can exist.
- The research finding will support other researchers to do analysis of similar dam breach.

1.5 Scope of the Study

The scope of the study is limited to 16.68 km downstream river of the Mhtsab Azmati dam. In this study the analysis was focused on the development of the river characteristics i.e stream centerline, cross sectional cut lines, flow path line, bank line, inline structure etc using HEC - GeoRAS tool in the Arc GIS software, determination of probable maximum flood generated from the upstream catchment using HEC – HMS, estimation of dam breach parameters using the regression equations methods and the prediction of outflow hydrograph of dam breach, water surface elevation, top width of the flood, flood velocity and arrival time of the flood wave using HEC – RAS model due to overtopping and piping mode of failure on the main river excluding the other non-perennial rivers which were located to downstream of the dam. The inundation map of the flooded was also developed using the Arc GIS to identify the downstream risk area and the appropriate emergency action plan was also analyzed.

Chapter 2

2. Literature Review

2.1 Introduction

Flood is one of the most common hazard in the world and cause loss of lives, livelihood and property destruction. So while thinking to construct a dam, as it provides benefit to the society like as hydropower, irrigation, water supply, flood control and recreation, it should be also give a consideration the downstream impacts that can be developed during catastrophic flooding occurs when a dam fails and the impounded water escapes through the breach in the downstream valley.

As Dam Safety Training Module (December, 2014) describes a Dam Safety Surveillance and Monitoring Plan (SMP) shall be developed and implemented for all dams with a safety risk and a Potential Consequences Classification of low or higher. The SMP shall be developed to apply to the particular phases of the dam life cycle, such as construction, initial filling, normal operation, etc. and updated periodically as the operation and conditions evolve. Developing and applying of such programs in all Ethiopian dams insures the well conditional living of the downstream community and expansion of different infrastructures which have a great role on the economic development of the country.

2.2 Dam Breach History

The history of water retention structures coexists with the history of their failures. Around the world thousands of dams have been constructed over many centuries. But also, hundreds of dams have failed and every year many dikes breach due to high flows in the rivers, sea storm surges, etc. often leading to catastrophic consequences. Some of the major dams failed and which caused high loss of lives are listed as below (https://en.wikipedia.org/wiki/Dam_failure).

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

Table 1: list of major dam failures in the world.

Dam	year	Location	Fatalities	Details
Machchu-2 Dam	1979	Morbi, India	5000	Heavy rain and flooding beyond spillway capacity. Old estimates were 1,800–25,000 but a 2011 book by Sandesara & Wooten refined the bracket to 5,000–10,000.
Banqiao and Shimantan Dams	1975	Zhumadian, China	171,000	Extreme rainfall, beyond the planned design capability of the dam, dumped on China by Typhoon Nina. 11 million people lost their homes. Dam would later be rebuilt between 1986 and 1993.
Sempor Dam	1967	Central Java Province, Republic of Indonesia	2,000	Flash floods over-topped the dam during construction.
Swift Dam	1963	Monte Toc, Italy	2,000	Strictly not a dam failure, since the dam structure did not collapse and is still standing. Filling the reservoir caused geological failure in valley wall, leading to 110 km/h landslide into the lake; water escaped in a wave over the top of dam. Valley had been incorrectly assessed stable. Several villages completely wiped out.
Kurenivka mudslide	1961	Kiev, Ukraine (then Ukrainian SSR of the Soviet Union)	1,500	Impoundment of the clay slurry reservoir (storing the waste of the local brick factories) failed after heavy rains, inundating the Kurenivka neighborhood with meters of mud, up to 2,000 fatalities.
Panshet Dam	1961	Pune, India	1,000	Dam wall burst due to pressure of accumulated rain water.
Möhne Dam	1943	Ruhr, Germany	1,579	Destroyed by bombing during Operation Chastise in World War II. 11 factories were destroyed, 114 seriously damaged.
Tigra Dam	1917	Gwalior, India	1,000	Failed due to water infiltrating through foundation. Possibly more fatalities.

2.3 Dam Breach Analysis

Dam breach analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting potential flood hazards at the downstream region. This generally deals with simulation of assumed failure for dams and analyzing the resulting consequences (Pandya and Jitaji, 2013).

Dam breach inundation analyses include the following four elements (FEMA, 2013a)

- Estimation of the dam breach parameters,
- Estimation of the dam breach outflow hydrograph
- Routing of the dam breach hydrograph downstream and
- Estimation of downstream inundation extent and severity.

2.3.1 Purpose of Dam Breach Analysis

As flood generated from the dam breach is much higher than flood generated from the heavy precipitation, the way and rate at which a dam breaches can affect the timing of the breach, the rate and magnitude of the flood water released and the size of the breach itself. Therefore, breach affects the analysis of flood risk and can change the way in which flood events might be managed.

After knowing the results which obtained from the dam breach analyses, it may help to accommodate the downstream impacts that can occur by developing:

- Inundation maps
- Emergency action planning
- Emergency response
- Mitigation planning and
- Consequence assessment.

Each use has unique information requirements and may be used in different manners ranging from multi-year office-based planning efforts by mitigation planners and dam safety officials to field-based emergency responders responding to a developing or imminent dam breach.

2.3.2 Dam Breach Parameters Definition

The term breach parameter is commonly used to describe the parameters needed to physically describe the breach (breach depth, breach width, and side slope angles) as well as parameters that define the time required for breach initiation and development. These parameters

are key in calculating the dam breach outflow amount and the hydrograph that can develop on the downstream of the breach location. Those are as described as below.

- Breach formation time (t_f): the duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached its full geometry.
- Breach depth (h_b): the breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.
- Breach width (B_{ave}): is the average of the final breach width, typically measured at the vertical center of the breach.
- Bottom width (W_b): is the breach width developed when the breach formation reaches its maximum.
- Breach side slope ($zH: 1V$): the breach side slope is a measure of the angle of the breach sides represented as z horizontal to 1 vertical.

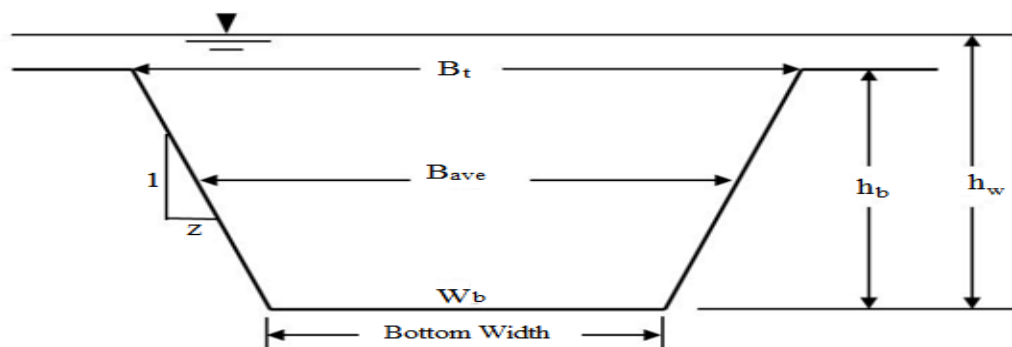


Figure 1: Dimensions of Dam Breach

A dam breach usually occurs in two distinct phases starting with the breach initiation followed by the breach formation.

Breach initiation: During the breach initiation phase, flow through the dam is minor and the dam is not considered to have failed. It may be possible to prevent a dam breach during this phase if flow is controlled.

Breach formation: Breach formation (defined above) begins when the flow through the dam has increased and progressed from the upstream face to the downstream face of the dam. It is uncontrolled, and will result in the failure of the dam (FEMA, 2013a).

2.3.3 Dam Breach Parameters Estimation

The geometric description of a dam breach must be estimated to simulate the resultant flood wave and downstream consequences. There are four critical elements of any breach analysis:

- Breach parameter estimation (breach size/shape and time of failure)
- Breach peak discharge and breach hydrograph estimation
- Breach flood routing and
- Estimation of the hydraulic conditions at critical locations.

The most commonly used approaches for the required elements of the analysis are as described below (CDWR, 2010).

2.3.3.1 Comparative Analysis

The simplest approach to dam breach flood estimation is comparative analysis. This method compares a given dam of interest with those in a database of well documented dam failure case histories. A given dam geometry, height, slope angles, and reservoir areas and volumes are compared with a list of similar sized dams that have failed. Dam breach parameters and peak discharges reported from the failure case histories of similarly configured dams are then directly applied to the dam being analyzed.

2.3.3.2 Parametric Regression/Empirical/ Equations

Parametric regression equations are empirically derived using case study information to estimate the time-to-failure and ultimate breach geometry, then simulate breach growth as a time dependent linear process to compute breach outflows using principles of hydraulics. Numerous equations to predict breach parameters have been developed based on analyses of case studies.

Table 2: Published Parametric Regression Equations for Predicting Breach Parameters (FEMA, 2013a)

Reference	No of Studies	Relations Proposed (S.I. units, meters, m ³ /s, hours)
Johnson and Illes (1976)		$0.5h_d < B < 3h_d$ for earth-fill dams
Singh and Snorrason (1982, 1984)	20	$2h_d < B < 5h_d$ $0.15\text{m} < d_{\text{ovtop}} < 0.61\text{m}$ $0.25\text{hr} < t_f < 1.0\text{hr}$
MacDonald and Langridge Monopolis (1984)	42	<u>Earth-fill dams:</u> $V_{er} = 0.0261(V_{\text{out}} * h_w)^{0.769}$ [best-fit] $T_f = 0.0179(V_{er})^{0.364}$ [upper envelope] <u>Non-Earth-fill dams:</u> $V_{er} = 0.00348(V_{\text{out}} * h_w)^{0.852}$ [best-fit]
FERC (1987)		B is normally $(2-4) * h_d$ B can range from $(1-5) * h_d$ Z = 0.25 to 1.0 [engineered, compacted dams] Z = 1 to 2 [non-engineered, slag or refuse dams] $t_f = 0.1-1$ hours [engineered, compacted earth dam] $t_f = 0.1-0.5$ hours [non-engineered, poorly compacted]
Reclamation (1988)		$B = 3 * h_w$ and $t_f = 0.011 * B$
Singh and Scarlatos (1988)	52	Breach geometry and time of failure tenancies $B_{\text{top}}/B_{\text{bottom}}$ averages 1.29
Von Thun and Gillette (1990)	57	$B_{\text{ave}} = 2.5 * h_w + C_b$ (C_b factor depends on the storage capacity of Reservoir) $T_f = 0.02 * h_w + 0.25$ (for erosion resistance materials) $T_f = 0.015 * h_w + 0.25$ (for easily erodible materials)
Froehlich (1995)	63	$BR = 0.1803 * K_o * V_w^{0.32} * h_b^{0.19}$ and $t_f = 0.00254 * V_w^{0.53} * h_b^{-0.90}$

		$K_o = 1.4$ for overtopping; 1.0 otherwise
Froehlich (2008)	74	$BR = 8.239 * K_o * V_w^{0.32} * h_b^{0.04} \& T_f = 3.664 * \left(\frac{v_w}{gh_b^2}\right)^{0.5}$ $K_o = 1.3$ for overtopping; 1.0 otherwise

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time depending actual failure of dams. MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) were among empirical regression equations which to calculate the breach parameters with the help of HEC-RAS software for this study.

(Wahl, 2010) suggests that one of the main advantages of using empirical parametric regression equations is that the user can exhibit some control over the breach parameters used in the model, and thus account for site - specific factors.

2.3.3.3 Predictor Regression Equations

Predictor regression equations are empirically developed equations used to estimate peak discharge based on actual case study data. These equations are used as a prediction method to determine a reasonable outflow hydrograph shape.

Table 3: The empirical relationships developed by various authors for predicting peak breach discharge (FEMA, 2013a)

Reference	Case Studies	Relations Proposed
Babb and Mermel (1968)	> 600 incidents	
Kirkpatrick (1977)	16 (plus 5 hypothetical failures)	$Q_p = f(h_w)$
Soil Conservation Service (SCS) (1981)	13	$Q_p = f(h_w)$
Hagen (1982)	6	$Q_p = f(h_w * S)$
Reclamation (1982)	21	$Q_p = f(h_w)$
Graham (undated)	19	$Q_p = f(h_w, S)$

MacDonald and Langridge Monopolis (1984)	42	$V_{er} = f(V_{out} * h_w)$ $t_f = f(V_{er})$ $Q_p = f(V_{out} * h_w)$
Costa (1985)	31 constructed dams	$Q_p = f(h_d)$ $Q_p = f(S)$ $Q_p = f(h_d * S)$
Evans (1986)		$Q_p = f(V_w)$
FERC (1987)		Guidance for B, Z, t_f
Reclamation (1988)		B, t_f guidance
Singh and Scarlatos (1988)	52	Guidance for B, Z, t_f
Von Thun and Gillette (1990)	57	Z guidance $B = f(h_w, S)$ $t_f = f(h_w, \text{erosion resistance})$
Froehlich (1995b)	63	B, Z, t_f relations
Froehlich (1995a)	22	$Q_p = f(V_w, h_w)$
Froehlich (2008)	74	$Q_p = 3.1 B_{avg} H_w^{1.5} \left(\frac{\gamma}{\gamma + T_f * h_w^{0.5}} \right)^3$

MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) were among empirical regression equations which to calculate the peak discharge for the given dam breach analysis with the help of HEC-RAS model.

2.3.3.4 Physically Based Erosion Models

A physically-based model (also referred to as a “process” or “causal” model) utilizes generally accepted relationships based on physical principles to establish the framework of a model. The model then attempts to solve those relationships for a given input. This is a relatively simple concept, but it can become very complex when the input is changing with time. In the case of dam breach analysis, both the input and physical constraints are changing with time as the dam erodes and the reservoir evacuates.

Although several physically-based models have been reported as being in the development stage for research purposes, the National Weather Service's BREACH program (NWS BREACH or BREACH) is currently the only widely available model. BREACH predicts the development of a breach and the resulting outflow using an erosion model based on principles of hydraulics, sediment transport and soil mechanics. It was initially developed in 1987, but has had several upgrades in 1988, 1991, and 2005. The model takes into account several components of a dam and reservoir that are not considered in the empirical methods, such as area versus elevation, dam dimensions, soil properties of the dam, and tail water effects downstream. It is relatively simple to run and is widely used within the United States (FEMA, 2013a).

Unfortunately, BREACH is no longer supported by the National Weather Service and significant advances in the understanding of the complex mechanics of a dam failure have not been incorporated (Wahl, 1998). Also, the model has only been calibrated with a very limited number of cases.

2.3.4 Embankment Dam Breach Mechanisms

The failure of a dam can be either accidental or deliberate, natural or man-made. Accidental failures may occur as a result of natural deterioration of the structure, extraordinary natural events such as extreme rainfalls and floods, earthquakes, differential settlement, rockslides, piping, seepage, overtopping, wave action, etc. Man-made causes include bombing, sabotage, demolition for public good, poor construction, faulty design, improper reservoir operation, improper location, animal burrowing, etc (Singh, 2013).

The most common causes of dam failure between January 1975 and January 2011 are summarized in table below. Flood or overtopping was the most common cause of dam failure, followed by piping.

Table 4: Causes of Dam Failure (1975-2011)

Cause of Failure	Number of Dam Failures	Percentage of Dam Failure
Flood 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	100%

Sources: Yonatan Sisay (2016)

As listed in the above table, overtopping and piping are among the most series causes of dam failure and this document was more focusing on the given dam breach analysis depending on these failures.

2.3.4.1 Overtopping Failure

Overtopping is the most cause of embankment dams to fail and is happening when the water level exceeds the height of the dam crest, causing it to spill over the dam. After water begins to overtop the dam, erosion of the dam crest occurs and removes massive amounts of material starting from the downstream face of the dam and head cut will develop to words the upstream face of the dam. This material makes up the weight that holds the dam in place against the hydraulic forces acting to level the dam. These forces and subsequent events normally lead to the complete failure and washing away of the embankment dam.

Most dams overtop due to high water levels and heavy rain. Their spillway capacity may not be adequate relative to the inflow of water that occurs. If the spillway is only designed to handle a low percentage of that area's Probable Maximum Flood (PMF) it is likely that an extreme weather event could cause the dam to overtop. Spillway design standards differ from state to state and depend on the hazard level of the dam. Large settlements of the foundation can cause loss of

Freeboard and could also result in overtopping as shown in the following figures (Imbrogno, 2014).

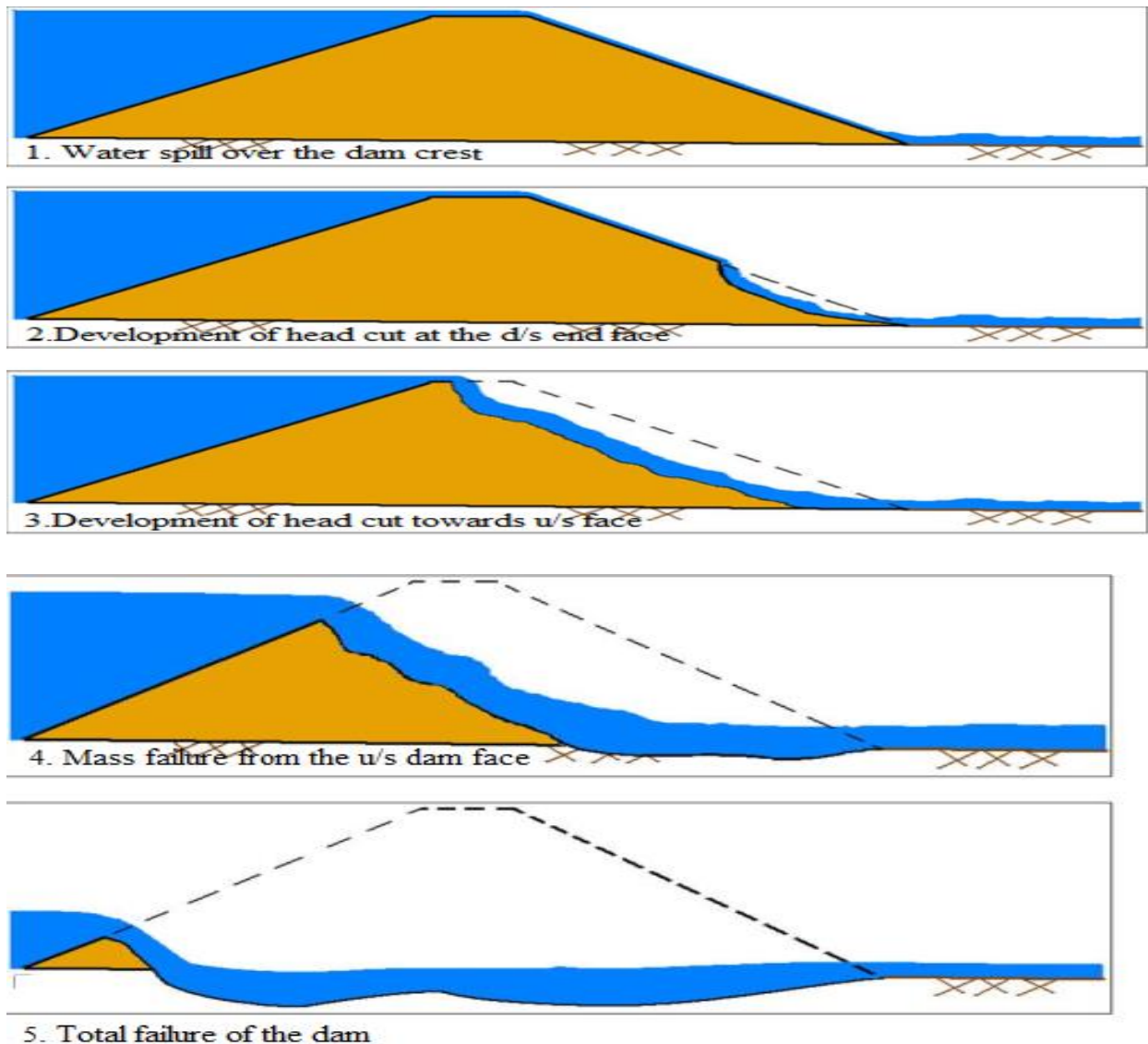


Figure 2: Examples Breach Process of Overtopping (Brunner, 2014))

Generally, breach due to overtopping is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent towards the bottom of the dam axis. If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the stream main channel. A generalized trapezoidal breach progression is shown below.

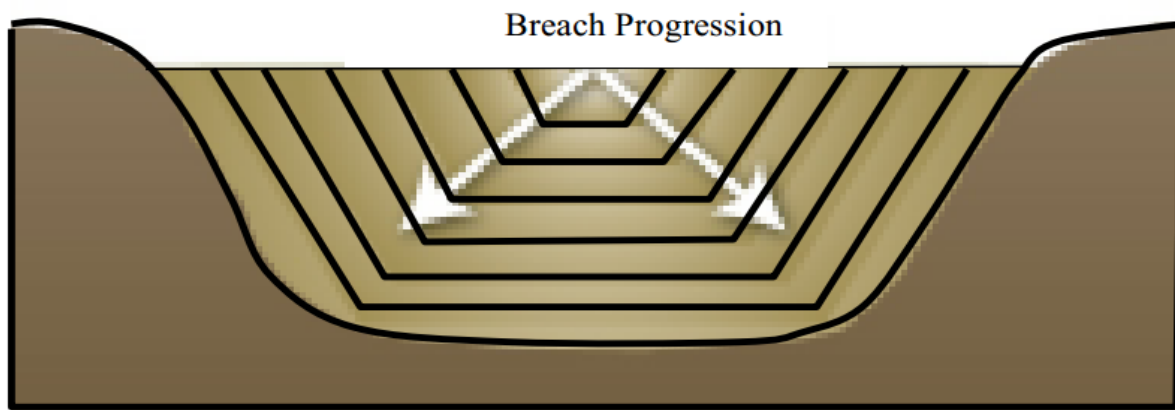


Figure 3: Overtopping trapezoidal breach progression (Gee, 2009).

The breach may stop growing when the reservoir has emptied and there is no more water to erode the dam or the dam has completely eroded to the bottom of the reservoir or has reach bed rock (FEMA, 2013a).

The breach regression may be modeled as either a linear progression or sine wave progression:

- Linear progression: rate of erosion remains the same for the duration of erosion development.
- Sine wave regression: breach grows very slowly at the beginning and end of the development and rapidly in between.

In a study by the State of Colorado Department of Natural Resources, no significant difference were found between linear and sine wave progression models when comparing one overtopping case study in HEC-Hydrologic Modeling System (HMS) and HEC-RAS (2010). As a result, this thesis was conducted with the Sine wave regression breach progression starting from the of center crest of the dam as shown in the figure below.

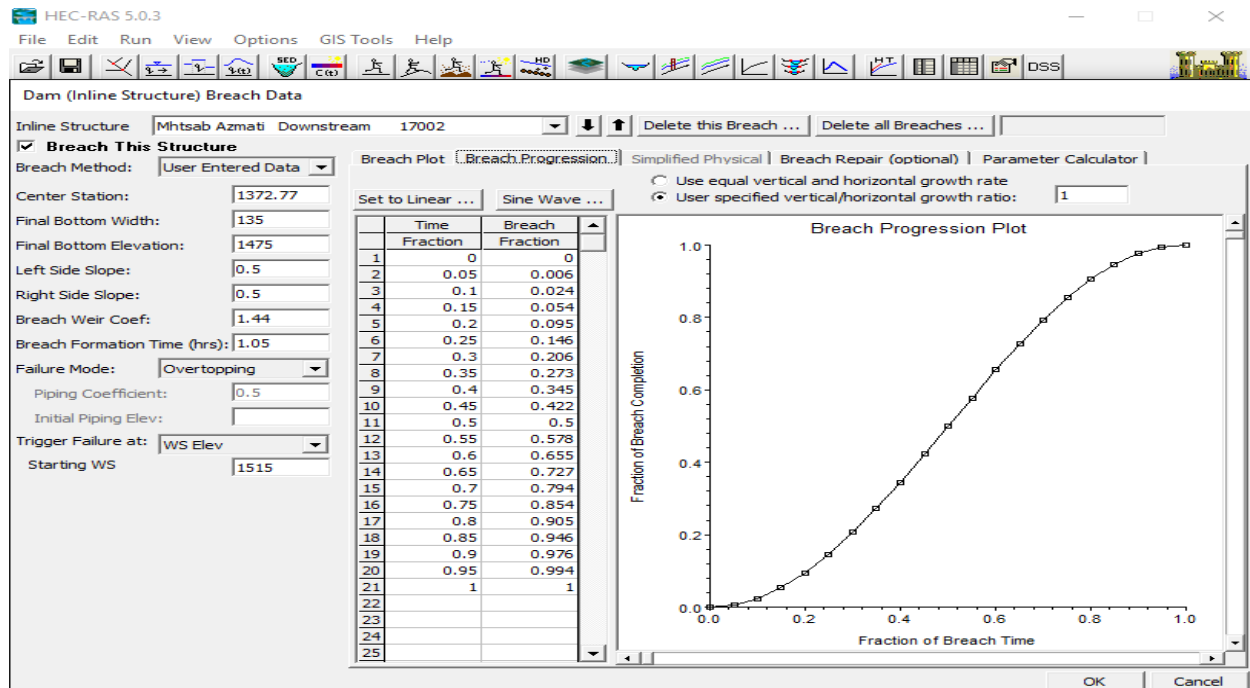


Figure 4: Sine wave regression breach progression

2.3.4.2 Piping/ Internal Erosion Failure

Piping and seepage are also major causes of embankment dam failures which cause erosion and saturation in the dam body or foundation material and cause it to lose strength. Piping is when the seepage of water through the foundation or embankment begins to cause internal erosion. Erosion generally begins in the downstream portion of the dam, and works backwards toward the upstream end. The water forms channels through or under the dam's embankment or foundation which follow the path of maximum permeability. Depending on the level and location, seepage and piping are sometimes hard to catch, and in some cases may not become an issue until many years after construction. Piping paths through to the embankment core is a serious cause of failure.

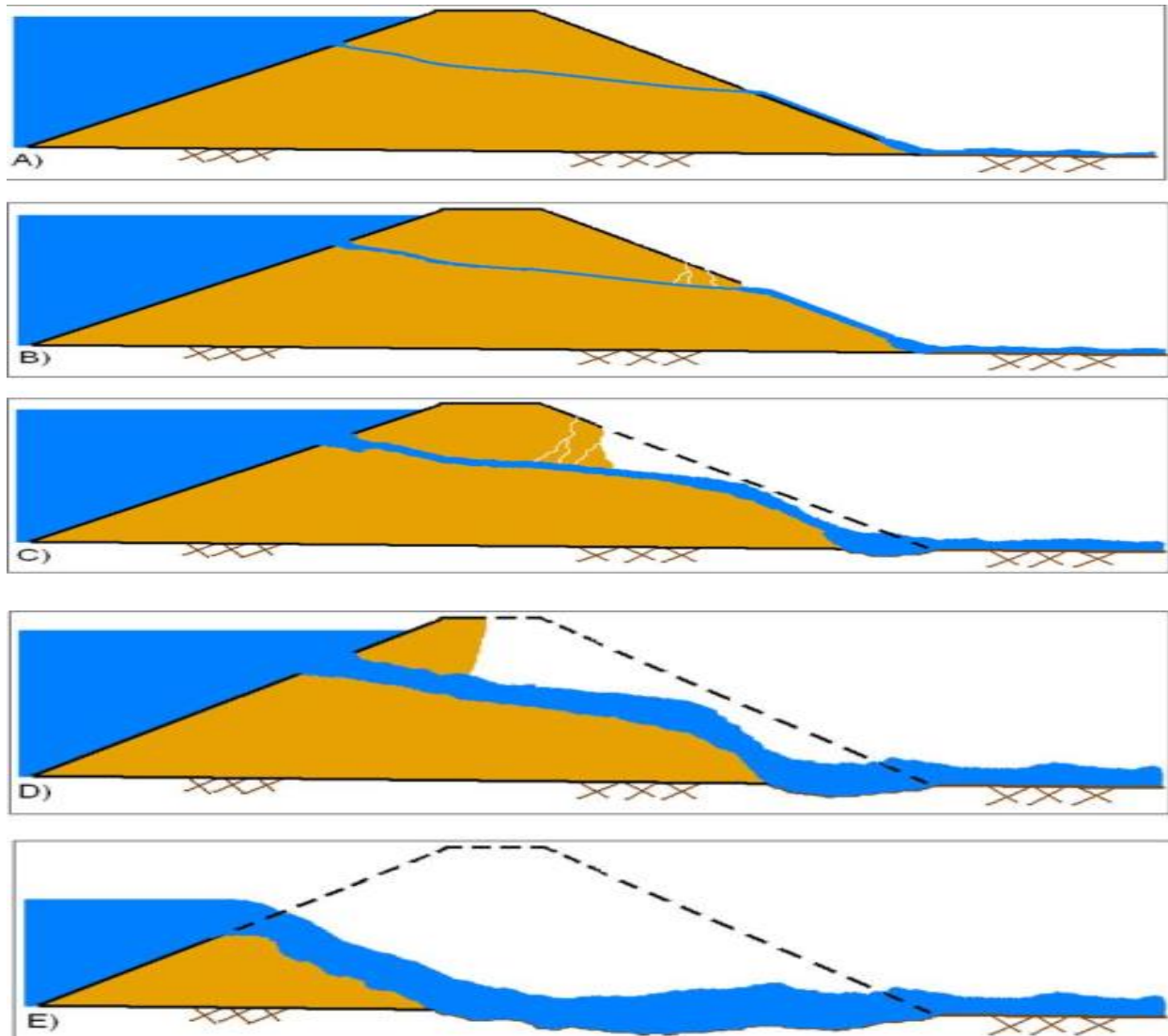


Figure 5: Examples Breach Process of piping (Brunner, 2014)

Mhtsab Azmati embankment dam has an irrigation fixed outlet on the left side at an elevation of 1494.4m (inlet) and sill level of pipe outlet at 1488.77m.a.s.l and two outlet pipes with a diameter of 0.6m each in parallel and the total length from inlet to outlet of 150m. In the anchorage portion, often a problem of seepage is common due to less compaction during construction time for saving the pipe from crashing case and currently there was high observable seepage below and above the irrigation outlet. For this study, this portion was considered as the starting elevation for the piping failure mode and was developed to the other portions of the dam.

2.3.5 Dam Breach Analyzing Models

Diverse hydrologic and hydraulic models of varying complexities have been proposed in the past few decades to accurately predict the water levels and discharges along rivers. Numerical models used to carry out the dam breach study and flood routing which may be one-dimensional, two-dimensional or combination of both. When the channel width increases or the flow becomes non-channelized, accuracy of one-dimensional models decreases hence two-dimensional models are more reliable while carrying out floodplain study. Some of the models used for dam breach study and flood inundation mapping are described below.

1. **DAMBRK MODEL** was developed by NWS, USA in 1984 and it was updated by BOSS International. This model predicts dam breach wave formation and its progression in downstream. The important three features of DAMBRK are ability to describe dam failure mode temporarily and geometrically, computation of the outflow hydrograph through the breach section and its ability to route the outflow through a downstream channel. This model is good in determining potential influenced area allowing users to input geometric and temporal data to accurately predict the initial breach wave. This includes both piping and overtopping failures. The updated BOSS DAMBRK has many improvements including faster calculations and it has graphic interface (Fread, 1984).
2. **FLO-2D** model is developed based on MUDFLOW model in 1989. This predicts the flood hazard, mudflow and debris flow over alluvial river and this uses grid system to determine floodplain based on elevation, roughness coefficient and it is good in predicting flow path and area. In this model, sediment laden and without sediment flow can modelled. In this model, discharge is estimated based on depth of flow over each sector and summing up all sector on all the four sides of grid. Accuracy of the model is dependent on the density of grid system and the data available (O'brien et al., 1993).
3. **FLDWAV** model was developed by the National Weather Services, USA as a replacement to DAMBRK model. This model has wave front tracking for more accuracy and computational time required is less. This allows dam breach prediction and potential concerned area are calculated. This model is designed to model rapid flood events from large precipitation event or dam breach occurrences predicting flow through single stream or network of streams using real forecasting technology and considers terrain and properties of material at different time interval and adjust flow pattern. This model also has secondary function to predict flow

through hydrological structures and river basins and it performs dam breach analysis, flood predictions, pumping stations and other rapid flow scenarios. It allows flow to change from subcritical to supercritical vice versa based on location and time and can model one dimensional unsteady debris flow also(Fread and Lewis, 1988).

4. **SMPDBK** was model is developed by National Weather Services (NWS), USA as simplified version of DAMBRK. This is good for obtaining dam classification and potential dam breach risk. Unlike DAMBRK this model is quick and easy to use and does not require many parameters but provides similar results compared to DAMBRK when performed on simpler cases. The model produces the information needed to find out the downstream inundation areas. The SMPDBK model does not require a computer with high specification to run the model and it is efficient model to simulate the dam breach scenario and for preparing emergency action plan (Wetmore and Fread, 1981).
5. **Danish Hydraulic Institute (DHI) MIKE 11** is a new generation software comprising of fully windows integrated graphical user interface. It is fully dynamic, user friendly, one dimensional tool for the breach analysis which simulates flood waves, sediment transportation, water quality in channels or water bodies, etc., The main feature of MIKE 11 is that it is an integrated modular structure with various add on modules like hydrology, advection – dispersion, water quality model, cohesive sediment transport, non-cohesive sediment transport, hydrodynamic model, rainfall – runoff model and flood forecasting model. Hydrodynamic module is one of the core module of MIKE 11 package simulating unsteady flow in open channel. It uses implicit finite difference method for simulation (Patro et al., 2009).
6. **Hydraulic Engineering CENTRE'S River Analysis System (HEC-RAS)** is developed by USACE's Hydrologic Engineering Center. In the previous versions of HEC-RAS, it was possible to simulate the flow as one-dimensional flow only using full Saint Venant equation to simulate the flow. In February 2016, RAS model with ability to perform 2D hydrodynamic unsteady flow routing using Saint Venant equation or Diffusion wave equation was introduced. HEC - RAS model simulate the flow in river channels and floodplains and the model was considered as efficient model for predicting downstream flooding effects from an upstream event i.e., by dam breach study. HEC - RAS model uses the breach information and breach geometry as input data to simulate the dam breach model. The model simulates the resulting flood wave generated based on the consequences of an upstream event and models downstream

effects based on the results of dam breach studies. It works with both off channel and on channel reservoir storages, dam breach modeling and mixed flow analysis.

According to Kumar et al. (2017), HEC-RAS is efficient tool which provides more reliable results in dam breach study and results obtained from the simulation model can be exported to GIS platform through HEC- GeoRAS to prepare the inundation mapping and also it was selected for this study on the dam breach analysis of Mhtsab Azmati dam.

2.4 Hazard Potential Classification

A system that categorizes dams according to the degree of adverse incremental consequences of a failure or mis-operation of a dam. Impoundments and dams are evaluated relative to their hazard potential classification, which affects the design criteria incorporated in the planning, development, and construction of dams.

The purpose of hazard potential classification is not to determine the likelihood of a failure occurring, but rather to assess the potential impacts should a failure occur and to establish appropriate criteria for use in the design and operation of the facility. Thus, more conservative design and operations criteria apply as the potential for loss of life or property damage from failure increases. The three hazard potential classifications for dams are as follows (FEMA, 2004).

- A. **Low hazard potential:** Facilities where failure would result in no probable loss of human life and low economic and/or environmental losses. Such facilities are usually located in rural or agricultural areas where losses are limited principally to the owner's property or where failure would cause only slight damage to farm buildings, forest and agricultural land, and minor roads.
- B. **Significant hazard potential:** Facilities where failure would likely not result in loss of human life, but can cause economic loss, environmental damage, or disruption of lifeline facilities. Such facilities are generally located in predominantly rural areas, but could be in populated areas with significant infrastructure and where failure could damage isolated homes, main highways, and minor railroads or disrupt the use of service of public utilities.
- C. **High hazard potential:** Facilities where failure will probably cause loss of human life. Such facilities are generally located in populated areas or where dwellings are found in the flood plain and failure can reasonably be expected to cause loss of life; serious damage to homes, industrial and commercial buildings; and damage to important utilities, highways, or railroads.

2.5 Dam Breach Analysis Approaches

According to FEMA (2013a) , the two primary approaches used for dam breach analysis are an event-based approach and a risk-based approach as discussed briefly as below.

2.5.1 Event Based Approach

An event-based approach is a deterministic method that requires the use of a specific or series of specific precipitation and non-precipitation events for the evaluation of dam failure and downstream inundation mapping. Among the precipitation or hydrologic events extreme rainfall and runoff events that can lead to natural floods of variable magnitude and non-precipitations or fair weather or sunny day are typically analyzed.

The greatest advantage to using an event-based approach is that it is a direct approach, is less complicated to perform and regulate, and produces more conservative breach inundation zone mapping when compared to a risk-based approach. High-hazard potential dams are typically evaluated using a full PMF, and significant- or low-hazard potential dams are evaluated on a percentage of a PMF or some more frequent storm event.

2.5.1.1 Fair Weather Failures

A fair- weather breach is a dam failure that occurs during fair- weather (i.e., non- hydrologic or non-precipitation) conditions. Establishing an initial reservoir water level and commencing a breach analysis without additional inflow from a storm event analyze the breach. A fair - weather breach is typically used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes.

Base flow conditions for a fair weather failure are typically ignored because of the small discharge and volume compared to that of a dam breach. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow unless the stream flow is gauged. The three most common initial water level elevations for fair weather breach analyses are as follows:

➤ **Normal Pool Elevation (invert of the highest elevation of the primary outlet)**

A breach at the normal pool elevation of the reservoir is used to estimate the volume and associated breach discharge that would result from a failure event during fair weather conditions. For an embankment dam, this type of event is modeled as piping/internal erosion failure.

➤ **Invert of Auxiliary Spillway (lowest uncontrolled spillway)**

A breach of the dam with the reservoir water level set at the auxiliary spillway (also referred to as an emergency spillway) is common practice to simulate a breach during mis-operation of the primary outlet works. Initiation of dam failure is typically the same as for the reservoir level at normal pool.

➤ **Top of Dam / Maximum High Pool**

The reservoir level set to the top of the dam to represent the maximum amount of volume that may be stored in the reservoir. This condition may be selected to evaluate the most conservative non-hydrologic event. In practice, dams without adequate spillways or pump storage facilities, where the water level during non-hydrologic events is maintained at the top of dam, are unique situations subject to this conservative assumption. A breach event when the water level is at the top of dam may be modeled as a piping / internal erosion failure or as an overtopping failure with the water level just above the top of dam invert. This fair weather failure approach helps to compute and analysis the piping case in this study.

2.5.1.2 Hydrologic Failure

Hydrologic breaches that occur with extreme precipitation and runoff are termed “rainy day” or hydrologic failures. Hydrologic failures that cause dam breach events are generally analyzed based on the IDF established by the dam’s hazard potential and hazard size classification. Typically a PMF is used for high-hazard potential dams; whereas, values ranging from 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF is used for significant hazard dams. This condition is commonly used to simulate a breach during overtopping of the dam. Likewise the above hydrologic failure approach helps to compute and analysis the overtopping case in this study.

2.5.2 Risk-Based Approach

A risk-based approach to dam design and dam safety evaluations has been developed to account for the downstream consequences of a potential dam failure. The consequences evaluation is not based on the probability of failure, but instead on the potential loss of life or increase in economic losses caused by a potential dam failure.

A benefit of the risk-based approach is that it may demonstrate, via an incremental damage assessment, that areas located downstream of a dam may be marginally affected by the reduction

in the SDF or IDF design standard for a dam. By lowering the SDF or IDF requirements, limited funding for needed rehabilitation measures can be used for more dams, resulting in an overall increase in dam safety.

A disadvantage of the risk-based approach is that by reducing the SDF or IDF to less than the full PMF based on downstream consequences, new development in the downstream breach inundation zone could alter the consequences, resulting in the need for future dam rehabilitation measures to increase spillway capacity. Effective risk communication as a component of the local development approval process can assist in reducing the occurrence of “hazard creep,” an occurrence where new downstream development in a dam breach inundation zone increases the dam’s hazard potential classification or SDF/IDF design requirement.

2.5.2.1 Inflow Design Flood and the Incremental Hazard Evaluation

According to FEMA 946- 2004, IDF is defined as “the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining structure is no longer considered to present an unacceptable additional downstream threat.”

The selection of the IDF is based on the evaluation of the magnitude of several flood events. The incremental hazard evaluation begins with simulation of a dam failure during a hydrologic flooding condition, typically beginning with the PMF or percentage of the PMF as specified by the State hazard potential classification requirements. The same hydrologic event is then run for non-failure conditions. The water surface elevations for both the breach and non-breach events are compared to determine the increase in the water surface elevation resulting from the dam breach. If the incremental increase in downstream water surface elevation between the failure and non-failure scenarios results in an acceptable increase in consequences, (as defined by State requirements) a smaller percentage of the PMF flood inflow or other magnitude flood is then used to repeat the process. The process is repeated until the incremental increase in consequences due to failure falls within acceptable requirements specified by the State.

Table 5: Recommended IDF Requirements for Dams Using Prescriptive Approach (FEMA, 2013b).

High	Probable loss of life due to dam failure or mis- operation	PMF
Significant	No probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities due to dam failure or mis-operation	0.1- percent- annual- chance exceedance flood (1,000-year flood)
Low	No probable loss of human life and low economic and/or environmental losses due to dam failure or mis-operation	1-percent-annual- chance exceedance flood (100-year flood)

2.5.2.2 Population at Risk

Probable loss of life is an important factor used in hazard potential classification systems and emergency action planning.

According to Graham (1999), from about 40 floods which were caused by dam failure, a risk based method was presented to estimate the number of fatalities that would result from dam failure.

These publications outline the following seven steps to complete an analysis for loss of life:

Step 1: Determine dam failure scenarios to evaluate

Step 2: Determine time categories for which loss of life estimates are needed

Step 3: Determine when dam failure warnings would be initiated

Step 4: Determine area flooded for each dam failure scenario

Step 5: Estimate the number of people at risk for each failure scenario and time category

Step 6: Apply empirically based equations or methods for estimating fatalities

Step 7: Evaluate uncertainty

2.6 Dam Breach Flood Routing

Routing of the breach outflow hydrograph downstream of the flood plain has a great advantage to evaluate the potential consequence of dam failure in the dam breach analysis and downstream risk area identification studies. Routing of the breach hydrograph is performed to evaluate the attenuated or reduced peak discharge at critical locations downstream of the dam. In addition to calculating the attenuation, determining the flood wave arrival time and the depth/velocity of flow at those critical locations are also very important parts of the analysis (Garcia-Martinez et al., 2009).

The propagation of flood waves in a natural channel is a gradually varied unsteady flow process, which is governed by conservation of mass and momentum equations. The solution of these equations in a distributed manner is referred to as distributed routing of flood waves. Routing by distributed system methods is called hydraulic routing and the flow is calculated as a function of both space and time. When no spatial variability is taken into account and when the channel reach or reservoir is considered as a black box, the corresponding routing procedure is referred to as lumped routing. Routing by lumped system methods is called hydrological routing. These methods calculate the flow as function of time alone (Chow, 1964)

2.6.1 Hydrologic System Routing

The hydrologic routing involved the balancing of inflow, outflow, and storage-discharge relation through use of the continuity equation in relation to time only. This application of hydrologic routing was used for reservoir routing. The reservoir component initiated by receiving upstream inflows and routed these inflows through a reservoir using Storage Routing Methods. The shape of breach, IDF, average width of breach, side slope of breach, breaching time, and other dam breach parameters were identified and predefined for generating the breaching outflow hydrographs by HEC RAS computer software.

2.6.2 Hydraulic System Routing

Dam breach outflow hydrographs were used as inputs to the river routing through the immediate downstream reaches of dam site. By the very nature, dam breach outflow hydrographs are highly unsteady flows that require a full unsteady flow routing method. In order to fully define an unsteady hydrograph, St. Venant equations should be used to analyze the routing flood wave propagation. Thus, the HEC-RAS hydraulic routing subroutine was adopted to route the dam breach outflow hydrographs through the Mhtsab Azmati Channel stream.

In the hydraulic modeling HEC-RAS helps to route the unsteady flow phenomenon either combined 1D/ 2D or fully 2D flow conditions to simulate the dynamic movement of water in the channel. By the 2D hydraulic model, how far the extent the flood inundation goes, how fast the flood wave propagates and the depth of a flood can be computed and a response can be also prepared like, evacuation maps, emergency action plans, series connections between the dam owner regulator and the downstream societies in areas of dam breach more flat terrains. Whereas, the 1D to model the steady flow condition in domains having confined corridors and step valleys to compute the water surface elevation only. The basic difference between 1D and 2D flow conditions are described as follows.

- The ability to perform combined 1D and 2D modeling within the same unsteady flow model will allow users to work on larger river systems, utilizing 1D modeling where appropriate and 2D modeling in areas that require a higher level of hydrodynamic fidelity.
- The program solves either the full 2D Saint Venant equations or the 2D Diffusion Wave equations. This is user selectable, giving modelers more flexibility.
- The 2D flow solver uses an Implicit Finite Volume algorithm. The implicit solution algorithm allows for larger computational time steps than explicit methods. The finite volume approach provides a measure of improved stability and robustness over traditional finite difference and finite element techniques.
- The 1D and 2D solution algorithms are tightly coupled on a time step by time step basis (optionally, iteration by iteration within a time step).
- The software was designed to use structured or unstructured computational meshes.
- Each cell, and cell face, of the computational mesh is pre-processed in order to develop detailed hydraulic property tables based on the underlying terrain used in the modeling process. Additionally, each face of a computational cell is pre-processed into detailed hydraulic property tables (elevation versus, wetted perimeter, area, roughness, etc...). This allows the user to use larger computational cells, without losing the detail of the underlying terrain.
- Mapping of the inundated area, as well as animations of the flooding can be accomplished using RAS Mapper.
- The 2D Flow Area computational solution has been programmed to take advantage of multi-processors on a computer (Karvazy and Webster, 2015).

2.7 Hydrological analysis for Dam breach studies

Analyzing of hydrological aspect for the construction of different hydraulic structures such as dam, levees and flood control structures has a great advantages in the evaluation of the cost and risk involved in particular design application. Dams designed with the probable maximum flood are considered as zero risk of failure for the downstream population but highly construction cost (Stedinger). Handling such type of gap, this study was highly focused on the determination the probable maximum flood using hydrological model.

In assessment of catchment hydrology, it is in practice impossible to measure everything that anyone would like to know about the hydrological system, mainly due to high catchment heterogeneity and the limitations of measurement techniques. These limitations and the need to extrapolate information from the available measurements in both space and time initiated the application of hydrological models (Pechlivanidis et al., 2011).

According to the paper Dhami and Pandey (2013) there are different recently updated hydrological models which are increasingly being utilized to analyze the quantity and quality of stream flow, flood forecasting, reservoir system operations, groundwater development and protection, surface water and groundwater conjunctive use management, water distribution system, water use, climate and land use change impact study, ecology and a range of water management activities (Wurbs 1998; Singh and Woolhiser, 2002). Some of these hydrological models are as described below.

AnnAGNPS: Annualized Agricultural Non-point Source Model (AnnAGNPS) is a watershed-scale, continuous simulation model designed to predict the impact of management on water, sediment, nutrients, and pesticides in agricultural watersheds. It provides two options for spatial representation; Grid or Cell spatial representation or hydrologic response unit spatial representation characterized by homogeneous land soil properties (Yuan et al., 2008). Due to its availability only for the agricultural land, it was not considered for this study as it includes nonagricultural lands.

GSSHA: The Gridded Surface Subsurface Hydrologic Analysis (GSSHA) model, developed by USACE ERDC, is a physics-based, distributed, hydrologic, sediment and constituent fate and transport model. It can simulate stream flow generated by infiltration-excess and saturation excess mechanism, as well as exfiltration and groundwater discharge to streams. The model employs mass-conserving solutions of partial differential equations (PDEs) and closely links the hydrologic components to assure an overall mass balance and correct feedback. Spatial heterogeneity is

considered by dividing the watershed into cells comprising a uniform finite difference grid and processes that occur before, during, and after a rainfall event are calculated for each grid cell and then the responses from individual grid cells are integrated to produce the watershed response (Downer and Ogden, 2006). As the basic input data for the GSSHA hydrological model are beyond this study, it was not recommended to use it.

HYPE: Hydrological Predictions for the Environment (HYPE) model is a hydrological model for small-scale and large-scale assessments of water resources and water quality, developed at the Swedish Meteorological and Hydrological Institute during 2005–2007. It is a recently developed process-based, semi-distributed, conceptual model to simulate different multi-basin regions, encompassing wide variations in geomorphology, soil types, land uses and topography. HYPE integrates landscape elements and hydrological compartments along the flow paths with nutrient turnover and transport (Lindström et al., 2010). As it incorporates a wide variations in geomorphology, it was not applicable for this study.

MIKE-SHE: MIKE SHE is a comprehensive deterministic, distributed, and physically based modelling system capable of simulating all major processes in the land phase of the hydrologic cycle. It includes a full suite of pre- and post-processing tools, plus a flexible mix of advanced and simple solution techniques for each of the hydrologic processes. It includes the precipitation, interception, infiltration and evapotranspiration, subsurface flow in unsaturated and saturated zones, surface flow, and flow in channels or ditches (Zhao et al., 2012). Its application is slightly different as with this study and it was ignored.

HEC-HMS: The Hydrologic Modelling System (HEC-HMS), developed by US Army Corps of Engineers Hydrologic Engineering Center, is designed for both continuous and event-based hydrologic modelling. It provides several different options to the users for modelling various components of hydrologic cycle. Initially it was developed to simulate the precipitation-runoff processes of dendritic watershed systems and it was improved to solve widest possible range of problems including large river basin water supply, flood hydrographs, and small urban or natural watershed runoff and it has the capability of integrating with other software to accommodate well organized data (Feldman, 2000).

HECHMS model has been used successfully in different parts of the world River basins for catchment modeling (Yusop et al., 2007). HEC-HMS Soil Moisture Accounting (SMA) algorithm

has been used to analyze the long term impacts of climate change on water resources availability of the Blue Nile (Yimer et al., 2009).

As discussed in the above, HEC-HMS was promising model for providing multiple options to simulate hydrologic processes for this study and it contains four components: Basin model, Meteorological model, Control specification, and Input data. All the watershed physical descriptions are accomplished through the basin component which contains elements of the basin, their connectivity, and runoff parameters generated the given catchment with the help of HEC-GeoHMS tool in the ArcGIS software.

2.8 Inundation Mapping

The primary purpose of an inundation map is to show the areas that would be flooded and travel times for wave front and flood peaks at critical locations if a dam failure occurs or there are operational releases during flooding conditions.

Inundation maps have a variety of uses including EAPs, mitigation planning, emergency response, and consequence assessment. Each use has unique information requirements and may be used in different manners (FEMA, 2013a).

2.8.1 Emergency Action Plans

An EAP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. The EAP specifies actions the dam owner, in coordination with emergency management authorities, should take to respond to incidents or emergencies related to the dam. It presents procedures and information to assist the dam owner in issuing early warning and notification messages to responsible downstream emergency management authorities.

Depending on the inundation map developed, the EAP helps to assist the dam owner and emergency management authorities with identifying critical infrastructure and population-at-risk sites that may require protective measures and warning and evacuation planning. The EAP must clearly delineate the responsibilities of all those involved in managing the incident and how those responsibilities should be coordinated.

2.8.2 Emergency Response

Emergency response embodies the actions taken in the immediate aftermath of an incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the effect on critical infrastructure and the environment. In the case of dam failures and incidents, this would

be the response by the dam owner, local community emergency management, and first responders such as fire and police departments to minimize the consequences of an imminent or actual dam failure or incident.

Given the short warning times typically encountered with dam failures and incidents, dam emergency evacuation plans should be developed before the occurrence of an incident. It is recommended that plans be based on a worst case scenario and address the following elements, including identifying the roles and responsibilities for all action items:

- Identification of critical facilities and sheltering
- Initiating emergency warning systems (who is responsible and what is the method)
- Specific evacuation procedures, including flood wave travel time considerations (for example, evacuation of special needs populations and lifting evacuation orders)
- Distance and routes to high ground
- Traffic control measures and traffic routes
- Potential effect of weather or dam releases on evacuation routes (for example, identify whether portions of the evacuation route may be flooded before the dam incident occurs)
- Vertical evacuation/sheltering-in-place
- Emergency transportation
- Safety and security measures for the dam perimeter and affected areas
- Re-entry into affected areas

Although the EAP does not need to include the actual evacuation plan, it should indicate who is responsible for an evacuation, and what plan will be followed.

2.8.3 Hazard Mitigation Planning

Mitigation is the proactive effort to reduce loss of life and property by lessening the effect of disasters. This is achieved through identifying potential hazards and the risks they pose in a given area, identifying mitigation alternatives to reduce the risk, and risk analysis of mitigation alternatives. In the case of dam failures and incidents, hazard mitigation planning involves identifying the population at risk and identifying actions to reduce their vulnerability.

Hazard mitigation planners need digital data that defines the dam breach hazard. Information needed includes the breach inundation zone boundary, depth of flooding, velocity, and timing (FEMA, 2013a).

2.8.4 Dam Breach Consequence Assessment

Dam breach consequence assessment includes identifying and quantifying the potential consequences of a dam failure or incident. While hazard mitigation planning focuses on specific projects to reduce flood risk, consequence assessment focuses on the economic and social impacts of a potential disaster and the organizational and government actions needed in the aftermath of a dam breach to respond and recover. Data compiled for a consequence assessment can also be used in risk assessments.

Consequence assessment requires the same basic data as used in hazard mitigation planning, with the addition of data related to communicating the hazard to community elected officials and the public. Advanced mapping products that allow state-of-the-art visualization is key to communicating the hazards and consequences of a potential dam failure.

Chapter 3

3. Methods and Data Collection

3.1 General description of the Study Area

3.1.1 Location of the Study Area

The dam is located in the northern part of Ethiopia in the regional state of Tigray, in Mereb Leke Woreda and Wedihazo Tabia at altitude range of 1460 m above sea level. The Mihtsab Azmati dam site is located at an aerial distance of 12 km south of Rama town having geographic coordinates of 480996E and 154969N (GPS location in UTM) and it is accessible through the Adwa - Rama gravel road.

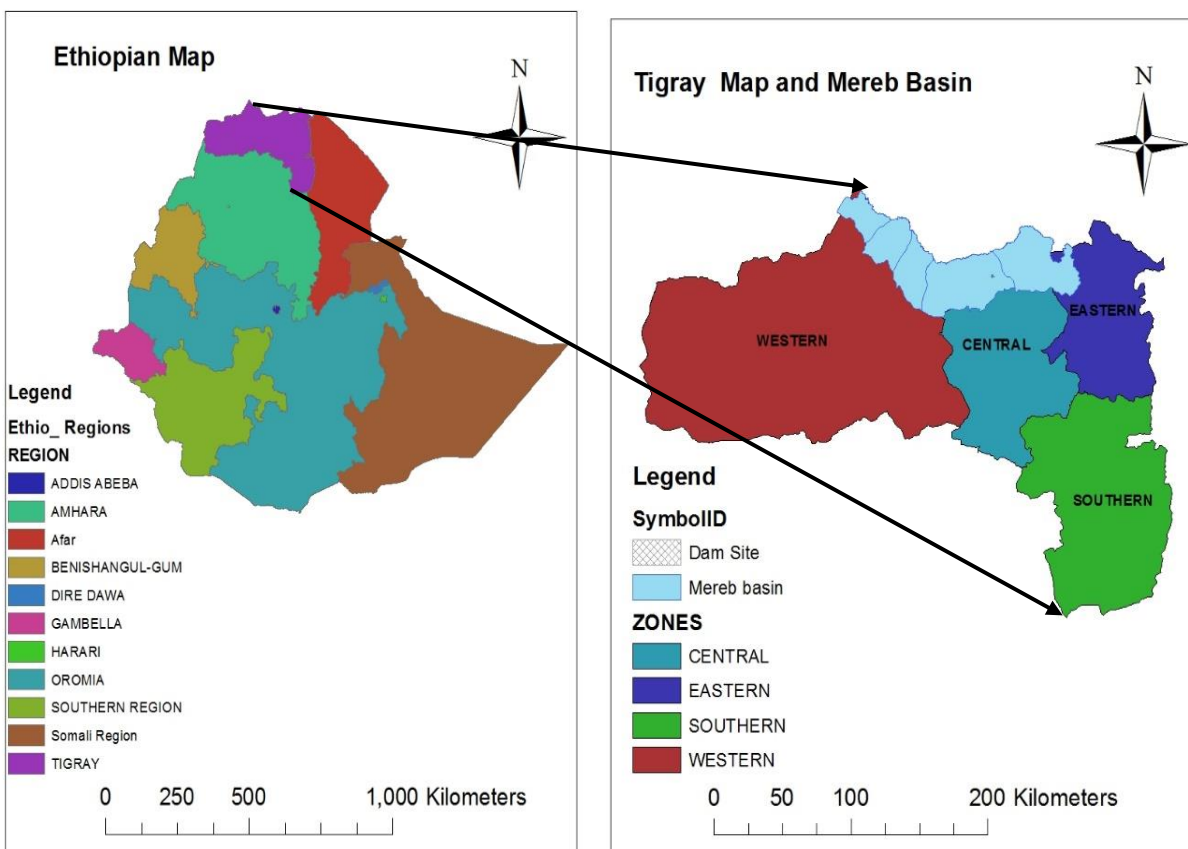


Figure 6: Location of the Study Area

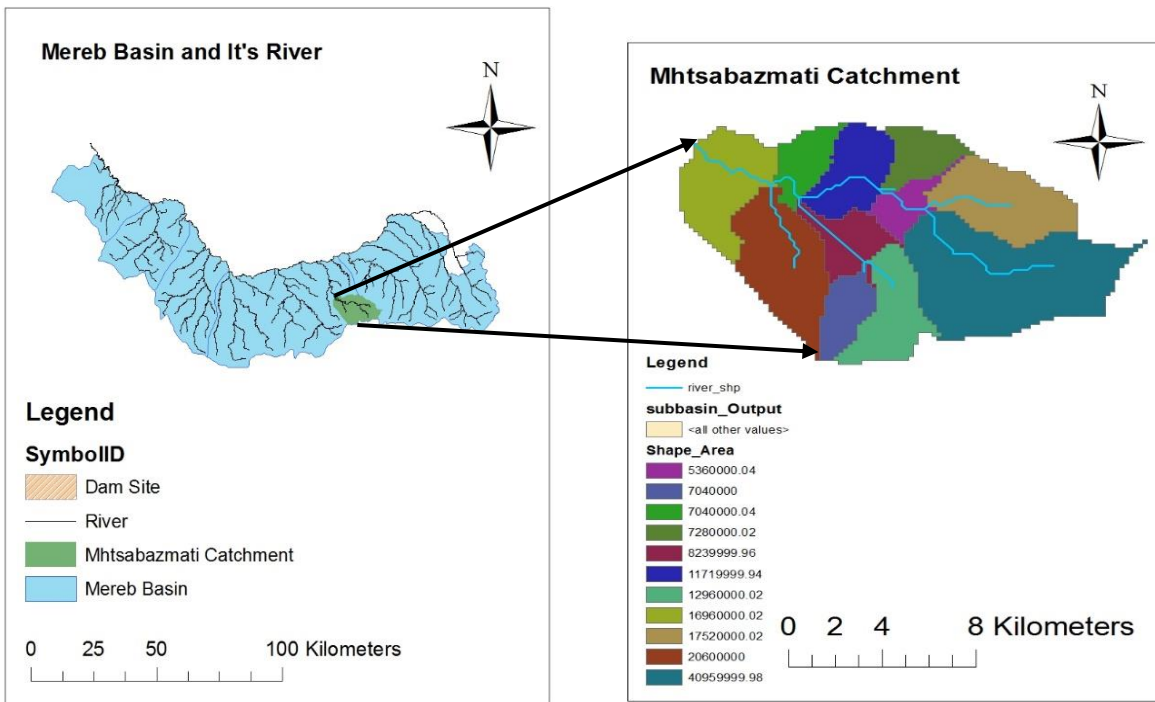


Figure 7: the catchment of the study area

The general study area and the cross section of the dam along dam axis which is extracted from 12.5m resolution size of Digital Elevation Model of the study area with the help of global mapper and looks as described in the figure below.

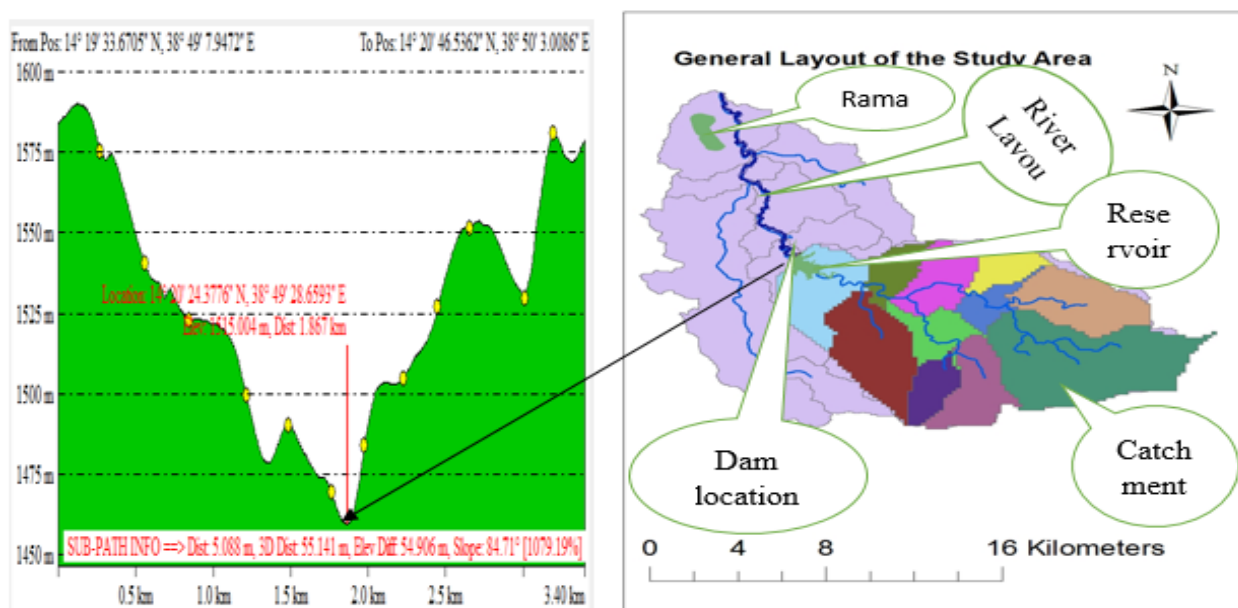


Figure 8: Cross Sectional Profile of the Dam Axis

3.1.2. Topography of the Study Area

The topography of the study area was nearly very steep sloped and undulating plane with an elevation of 1328 masl at the end of the river study and 2601 masl at the upper end of the Mhtsab Azmati catchment and about 1460 masl at the dam location. The topography had also a sloping terrains with a range of 0 - 60 degree as shown the map below. Generally the study river area, which is conducted for this thesis below the dam site, has a slope range 0 – 5 in degree or 0 – 9.8 in percent which a low land comparatively to the upper catchment of the dam.

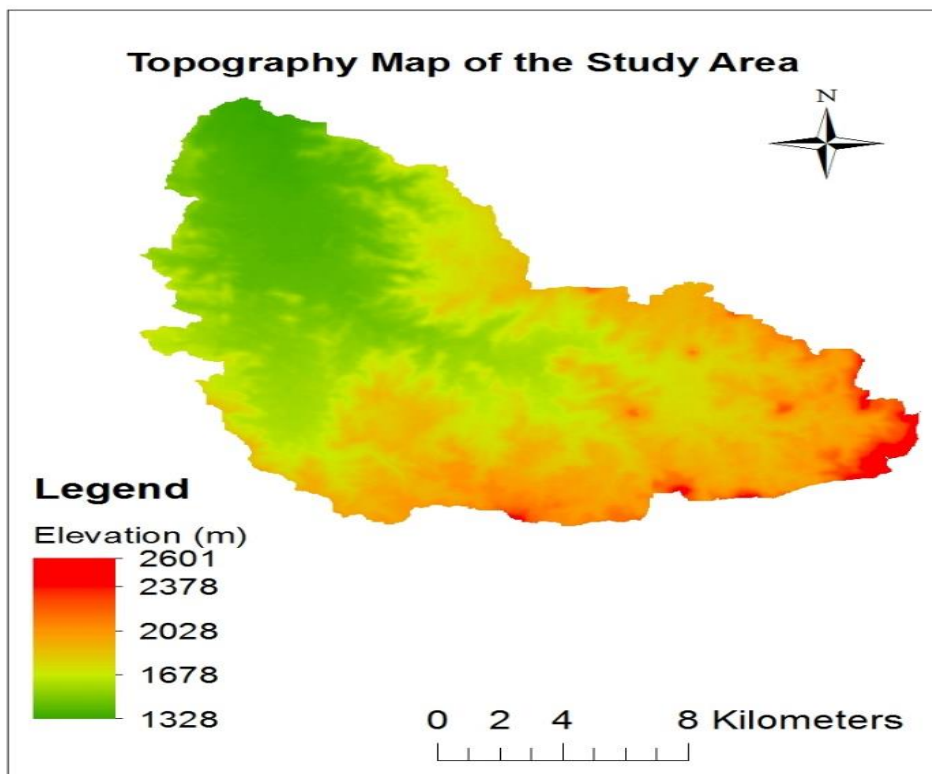


Figure 9: Topography map of the study area

3.2. Methodology of the Study

3.2.1 General

A field visit to the study area was conducted for several days in order to become familiar with the topography, reservoir area, observation of the dam characteristics like the downstream face coverage, the existed seepage condition and its location, the command area, the population settled on the downstream. Next to this, the type data to be collected and the method which help to the problem of analysis was decided. The general conceptual frame work methods the study was shortly noted as follows.

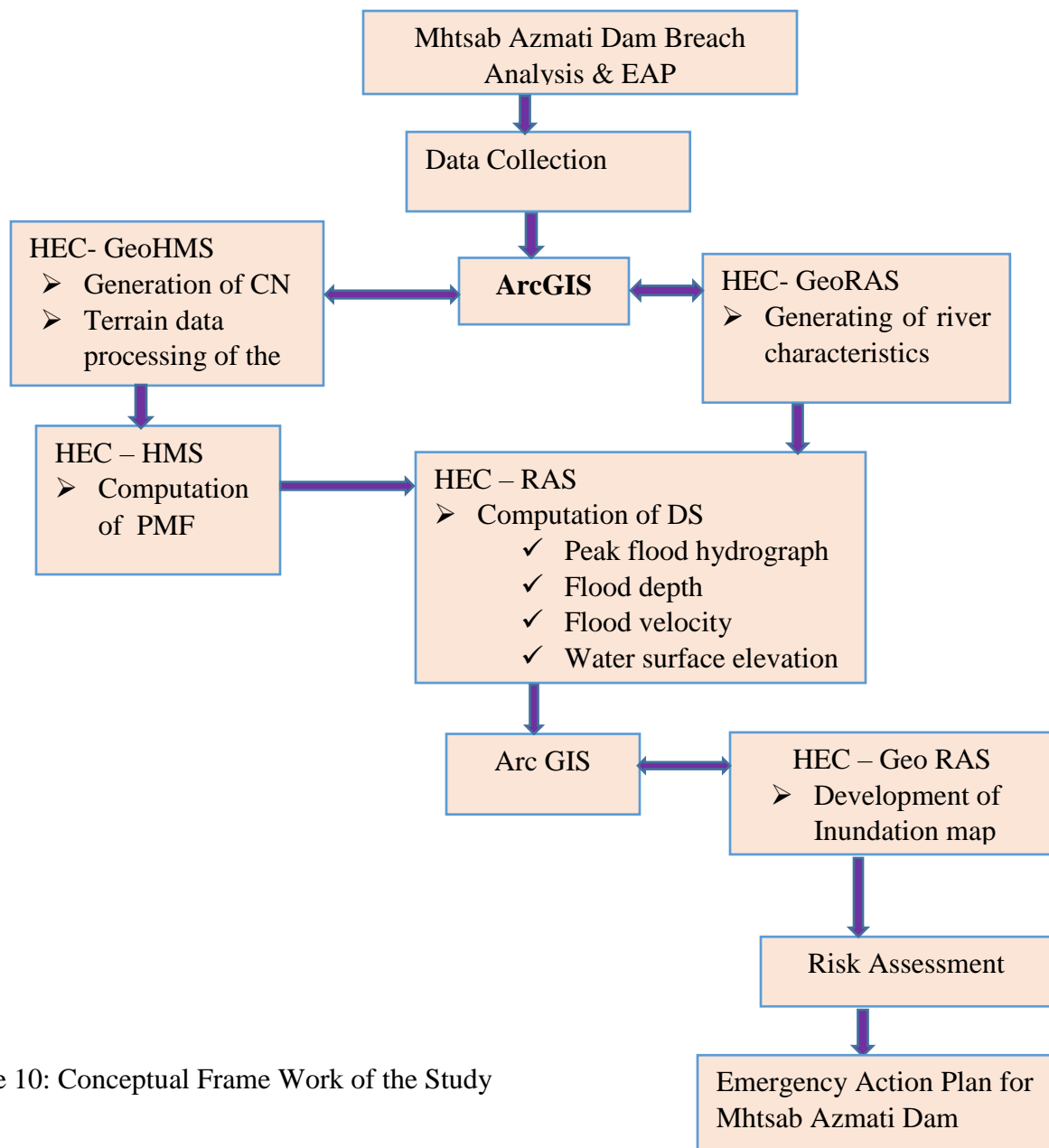


Figure 10: Conceptual Frame Work of the Study

3.2.2 Dam Breach Analysis Methods

As the research was conducted to study the dam breach analysis and the inundation map that can develop during flooding, dam breach parameter estimation methods and mathematical computation of the water surface elevation along the river are the main tasks to be analyzed before going to prepare an emergency action plan on the downstream area of Mhtsab Azmati medium dam.

3.2.2.1 Methods of Dam Breach Parameter Estimation

In dam breach analysis the estimation of the breach parameters like location, width, shape and time were crucial in order to make an accurate estimation of the flood plain level, outflow hydrograph and downstream inundation area. To compute the dam breach parameters, HEC-RAS model requires the following users input data to analyze the dam breach:

- Location: center line station of the breach in the Dam.
- Failure mode: overtopping or piping
- Shape: bottom elevation, bottom width, left and right side slope H:V
- Time: critical breach development time
- Trigger mechanism: pool elevation, pool elevation plus elevation or clock time
- Weir or pipe coefficient:
 - Weir coefficient are used to compute if the failure mode is overtopping and
 - Pipe or orifice coefficient for piping failure mode.

Keeping the above user input information, the different regression equations used for determination of dam breach parameters in HEC – RAS are listed as below (Brunner, 2010).

1. MacDonald and Langridge-Monopolis (1984)
2. Froehlich (1995)
3. Froehlich (2008)
4. Von Thun and Gillette (1990)
5. Xu and Zhang (2009)

MacDonald and Langridge-Monopolis (1984)

MacDonald and Langridge-Monopolis utilized 42 data sets (predominantly earth fill, earth fill with a clay core, and rock fill) develop a relation for the "Breach Formation factor". The Breach Formation Factor is a product of the volume of water coming out of the dam and the height of water above the dam. They used the following ranges for their regression analysis:

Height of the dam: 4.27- 92.96m

Breach outflow volume: 0.0037- 660*10⁶m³

The following is the MacDonald and Langridge-Monopolis equation for volume of material eroded and breach formation time as reported by Wahl (1998)

Earth-fill dams:

$$V_{er} = 0.0261(V_{out} * h_w)^{0.769}$$

$$T_f = 0.0179(V_{er})^{0.364}$$

Non-Earth-fill dams:

$$V_{er} = 0.00348(V_{out} * h_w)^{0.852}$$

Where: V_{eroded} = volume of material eroded from the dam embankment (m³)

V_{out} = volume of water that pass through the breach (m³) i.e storage volume at the time of breach plus volume of inflow after breach begins, minus any spillway and gate flow after breach begins.

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

t_f = breach formation time (hrs)

The actual breach dimensions are a function of the volume eroded and they considered the breach should be trapezoidal with side slopes of 0.5H: 1V and the breach size is computed by assuming the breach erodes vertically to the bottom of the dam and it erodes horizontally until the maximum amount of material has been eroded or the abutments of the dam has been reached. The base width of the breach can be computed from the dam geometry as follows.

$$W_b = \frac{V_{eroded} - h_b^2 \left(CZ_B + \frac{h_b Z_b Z_3}{3} \right)}{h_b \left(C + \frac{h_b Z_3}{2} \right)}$$

Where: W_b = bottom width of the breach (m)

h_b = height from the top of the dam to the bottom of the 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 the dam

z_2 = average slope ($z_2:1$) of the upstream face of the dam

z_b = side slope of the breach ($z_b: 1$), 0.5 for the MacDonald method.

Froehlich (1995)

Froehlich (1995) utilized 63 earthen, zoned earthen, earthen with a core wall (i.e clay) and rock fill data sets to develop a set of equations to predict average breach width, side slopes and failure time. The data that Froehlich used for his regression analysis had the following ranges.

Height of the dam: 3.66 -92.96m

Volume of water at breach time: $0.013 - 660 \cdot 10^6 \text{m}^3$

Froehlich's regression equations for average breach width and failure time are:

$$B_{ave} = 0.1803 * K_o * V_w^{0.32} * h_b^{0.19} \text{ and}$$

$$t_f = 0.00254 * V_w^{0.53} * h_b^{-0.90}$$

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

K_o = constant (1.4 for overtopping failure and 1.0 for piping)

V_w = reservoir volume at time of failure (m^3)

h_b = height of final breach (m)

t_f = breach formation time (hrs)

Froehlich states that the average side slopes should 1.4H: 1V for overtopping failure and 0.9H: 1V for piping failure.

Froehlich (2008)

Froehlich updates his breach equations based on the addition of new data as he utilized 74 earthen, zoned earthen, earthen with a core wall (i.e clay), and rock fill data sets to develop a set of equations to predict average breach width, side slopes and failure time.

The data that Froehlich used for his regression analysis had the following ranges.

Height of the dam: 3.05 -92.96m

Volume of water at breach time: $0.0139 - 660 \cdot 10^6 \text{m}^3$

Froehlich's regression equations for average breach width and failure time are:

$$B_{ave} = 0.27 * K_o * V_w^{0.32} * h_b^{0.04} \text{ and}$$

$$t_f = 63.2 * \left(\frac{V_w}{ghb^2} \right)^{0.5}$$

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

K_o = constant (1.3 for overtopping failure and 1.0 for piping)

V_w = reservoir volume at time of failure (m^3)

h_b = height of final breach (m)

g = gravitational acceleration (m/s^2)

t_f = breach formation time (second)

Froehlich states that the average side slopes should 1.0 H: 1V for overtopping failure and 0.7 H: 1V, piping failure.

Von Thun and Gillette (1990)

Von Thun and Gillette used 57 earthen dam data sets to develop a set of regression equations to predict average breach width, side slope and time of failure. The method proposes to use the breach side slope of 1H: 1V. The data that Von Thun and Gillette used for this regression analysis had the following ranges:

The equation that is developed by Von Thun and Gillette for average breach width is:

$$B_{ave} = 2.5 * h_w + C_b$$

Where: B_{ave} = average breach width (meters)

h_w = depth of water above the bottom of the breach (meters).

C_b = coefficient which is a function of the reservoir size of the dam. Its value is ranged between 6.1 and 54.9m. It is selected depending on the value that is listed in the table below.

Table 6: Coefficient as a function of reservoir size (Gary, 2014)

Reservoir Size (cubic meters)	C_b (meters)	Reservoir size (acre-feet)	C_b (feet)
$< 1.23 \times 10^6$	6.1	<1000	20
$1.23 \times 10^6 - 6.17 \times 10^6$	18.3	1000-5000	60
$6.17 \times 10^6 - 1.23 \times 10^7$	42.7	5000-10000	140
$> 1.23 \times 10^7$	54.9	>10000	180

For this study since the storage capacity is greater than $1.23 \times 10^7 m^3$, the value of C_b is 54.9m.

Von Thun and Gillette developed two different equations. These two equations also depend on the degree of erosion resistance.

The first set of equations shows the breach development time as a function of water depth for erosion resistant and easily erodible respectively are:

$$t_f = 0.02 * h_w + 0.25 \quad \text{Erosin resistance}$$

$$t_f = 0.015 * h_w \quad \text{Erodible}$$

Where: t_f = breach formation time (hrs)

h_w = depth of water above the bottom of breach (meters)

The second set of equations shows breach development time is a function of water depth and average breach width. These equations for erosion resistant and easily erodible respectively are:

$$t_f = \frac{B_{ave}}{4 * h_w} \quad \text{Erosion resistance}$$

$$t_f = \frac{B_{ave}}{4 * h_w + 61} \quad \text{Erodible}$$

Where: B_{ave} = average breach width

Generally the first regression equations formulated by Von Thun and Gillette used to predict the time failure of embankments is a function of depth of water only while the other is a function of depth of water and the computed average breach width.

These empirical regression equations were developed to predict the average breach width, breach depth, and time of failure or formation time depending actual failure of dams. MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) were among the empirical regression equations which used to calculate the dam breach parameters with the help of HEC-RAS model for this study.

3.2.3. River Hydraulic Computation Methods

Computation of the river hydraulics i.e the water depth, water surface elevation, velocity distribution of the flow helps to easily manipulating the research task during the dam breach analysis and inundation map of the study area. In such a case, the models which had a great role for the computing these necessary issues were described in here.

3.2.3.1 HEC – GeoRAS

HEC - GeoRAS is a set of ArcGIS tools specially designed to process geospatial data for use with the Hydrologic Engineering Center’s River Analysis System (HEC –RAS). The extension tool allows to develop the basic the river characteristics like stream center line, bank line layers, flow path center line layers and cross section cut line layers geospatial data using the 12.5m*12.5m resolution cell size Digital Elevation Model (DEM) within the GIS interface to create HEC – RAS import files for the specific study area . The resulting flood from dam failure and its inundation area is also analyzed with the HEC-GeoRAS extension tool at available of GIS data exported from

HEC- RAS model like maximum flow depth, water surface elevation and flow channel velocity distributions developed in the RAS Mapper tool of HEC – RAS. The main working tools of HEC-GeoRAS and the river layout of the study were described in the figure below respectively (HEC-GeoRAS, 2005).



Figure 11: HEC-GeoRAS Tool Bar Used in ArcGIS v10.3.1

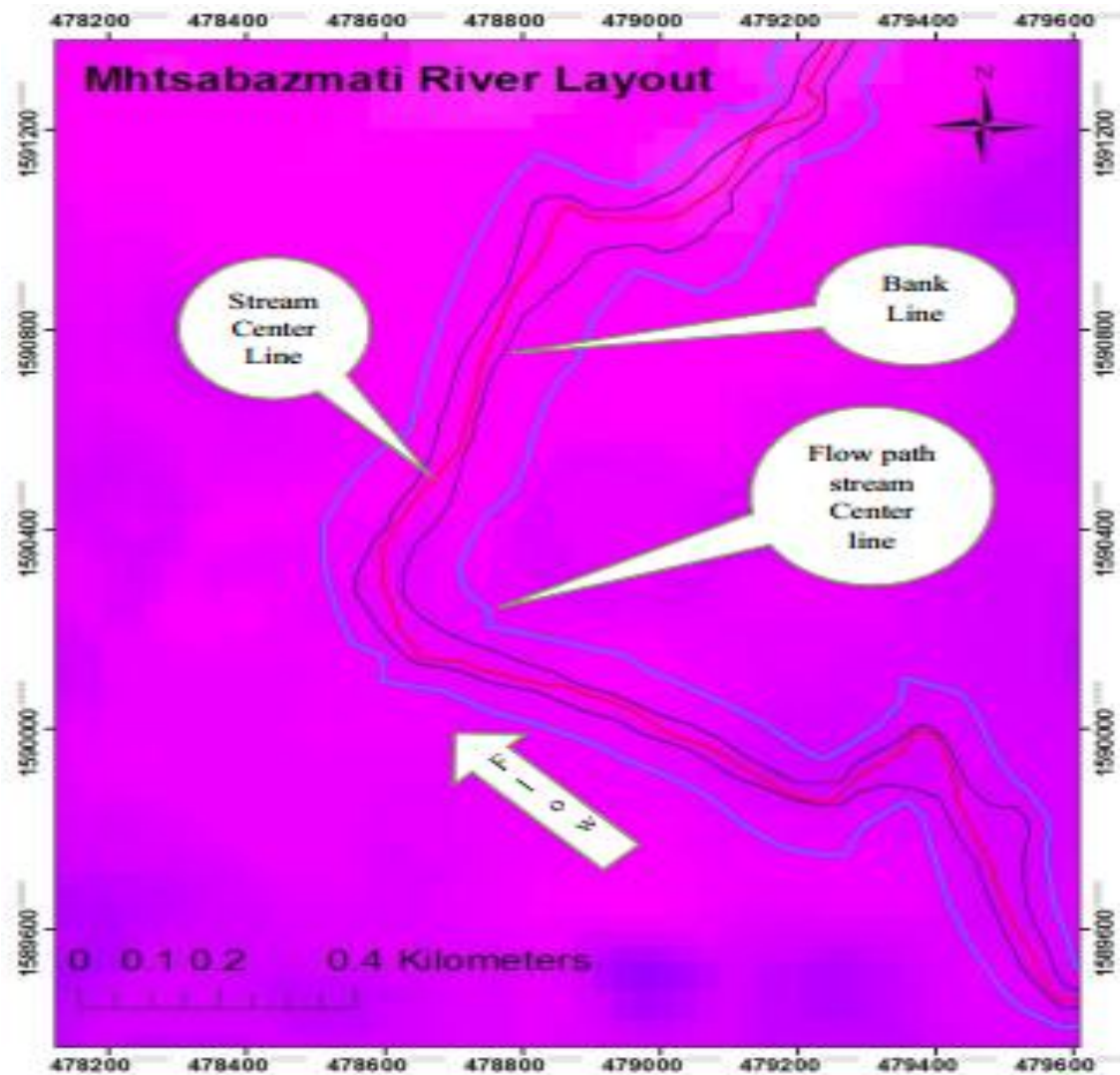


Figure 12: Study River Layout

3.2.3.2 HEC – RAS Model

HEC-RAS (Hydraulic Engineering Center's River Analysis System) is free downloadable software which was developed by Hydrologic Engineering Center of the U.S. Army Corps of Engineering. It is a computer programming that helps to model water flowing through systems of open channels i.e natural channel flow for this study. To accomplish the aim of this thesis, HEC-RAS had great application on the modeling of the Mhtsab Azmati natural river to know water flow depth, the water surface elevation and velocity distribution during the flooding time of the dam breach and exporting of the findings to the ArcGIS software to develop the inundation map of the hazard identification zones.

The different tasks included in this model while commutating the river hydraulics were editing of geometric data of the river layout and unsteady flow data as they were the basic data to formulate the dam breach analysis flooding simulations case.

3.2.3.2.1 Geometric Data

Encoding of the geospatial data of the river system to the HEC-RAS model was among the basic task in the site specific study in order to develop the river network system connection i.e river layout, river cross section and the inline structure as the dam. As discussed in the above, the geometric input data for this section were imported from the HEC-GeoRAS tool developed in the ArcGIS interface and the Manning's roughness coefficient 'n' values were adding in this section after analyzing the land use and land cover of the study area and according the standard values stated in different documents with relative to the site. The contraction and expansion constant values due to the flow cross section changes are also taken as the for the flow type of gradual transition as the river study moderately small slope. The model uses different tools while modeling the water flow system network in the geometric data editor section were as described below.

In addition to the above, fixing of the boundary conditions of the river flow system was also necessary for the computation of hydraulic parameters in the software under this portion. Accordingly, the external boundary conditions connected to the reach on the upstream river cross section and on the downstream end of the river reach were the probable maximum flood hydrograph and normal flow depth respectively.

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

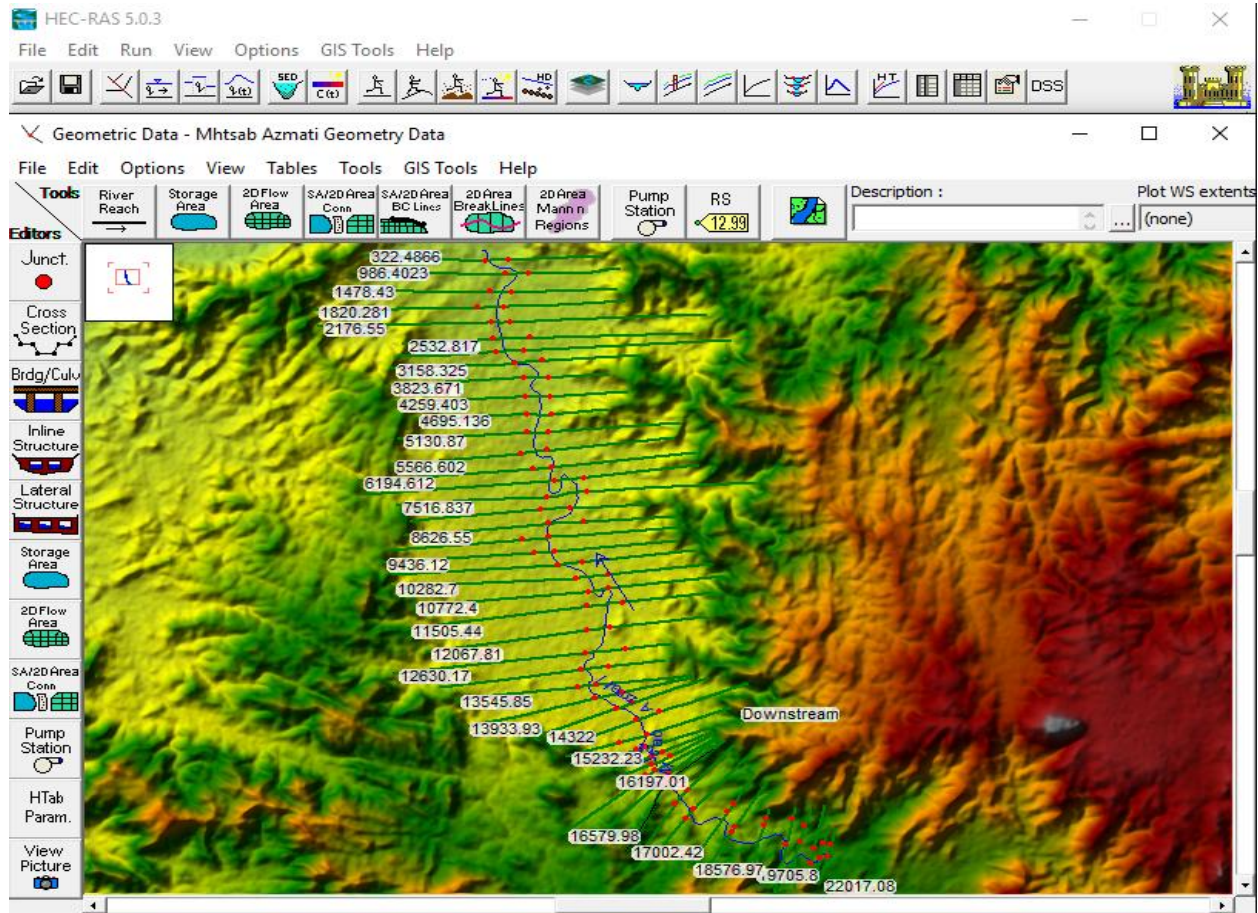


Figure 13: Geometry data editor tool bar

3.2.3.2 Unsteady Flow Analysis

Unsteady flow data analysis was also among the basic analysis that must be done during the dam breach analysis condition as needed case of determining the flood plain level to map the flood risk area. After entering all the geometry data such as boundary conditions for the farthest upstream and downstream cross sections and initial conditions into HEC-RAS, unsteady flow simulation can be initiated by adding the probable maximum flood hydrograph, which was evaluated using the HEC – HMS model at upstream and normal depth on the downstream point. The general way to enter the unsteady flow data to the model looks like as follows

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Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

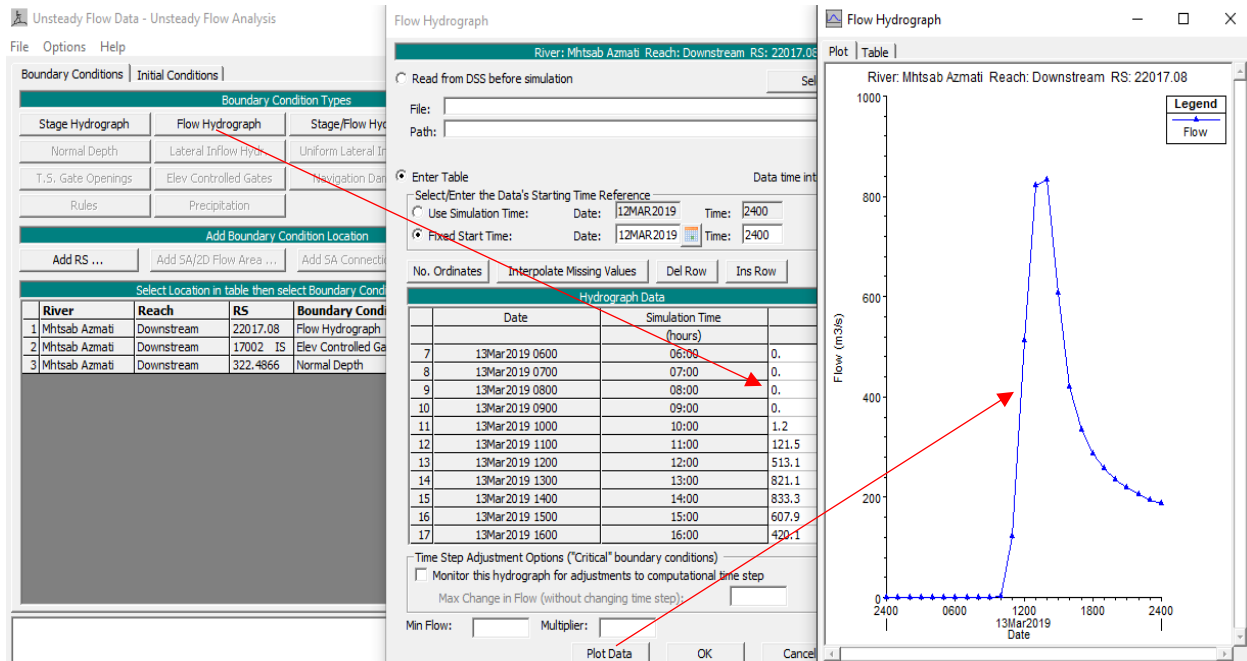


Figure 14: Unsteady flow data entry

3.2.4 Methods of Hydrological Analysis

As reviewed in the above, HEC-HMS 4.2.1 which is developed by USA Hydraulic Engineering center-Hydrologic Modelling System, was among the different hydrological models which can accommodate for the hydrological analysis and was selected of this study as elaborated in the literature review part.

HEC-HMS is a semi - distributed conceptual, continuous and event hydrological model which simulates rainfall run off. It requires daily precipitation, long term average monthly potential evapotranspiration (for calibration and validation), and geographical information such as the lag time, area of the sub basins of the catchment to get the simulated runoff at the outlet (Fleming and Neary, 2004).

HEC-HMS model setup consists of a basin model, meteorological model, control specifications, and input data (time series data).

3.2.4.1 HEC-HMS model setup

HEC-HMS Model setup consists of four main model components: basin model, meteorological model, control specifications, time series data, paired data, and gridded data. The Basin model for instance, represents the physical watershed contains the hydrologic element such as sub-basin, reach, junction, reservoir, diversion, source, and sink and their connectivity that represent the movement of water through the drainage system to compute the runoff (Ramly and Tahir, 2016). The meteorological model component helps to calculate the precipitation and evapotranspiration in point or gridded input data in the sub basin (Feldman, 2000). The spatio-temporal precipitation distribution over the basin can be accomplished by the gauge weight, specified hyetograph, gridded precipitation, frequency storm, SCS storm, inverse distance, HMR 52 storm and the standard project storm methods. Arithmetic mean, Thiessen polygon and Isohyetal are also among the techniques used to determine mean areal precipitation depth computation using the gauge weights method.

The time span of a simulation is controlled by control specifications, which include a starting date and time, ending date and time, and computation time step. The time series data component is also helps to enter the precipitation and observed flow data for the calibration and validation of long historical data.

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

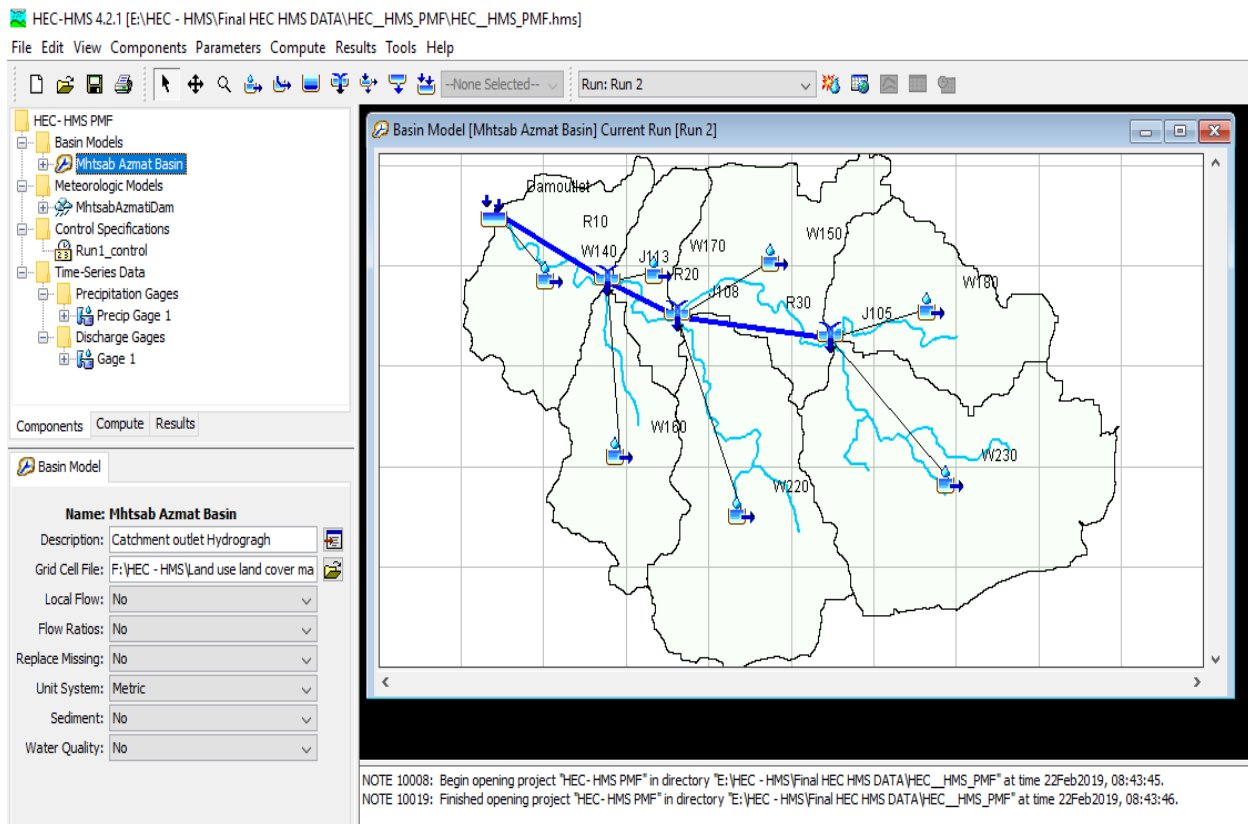


Figure 15: HEC-HMS model setup for the Mhtsab Azmati catchment

3.2.5 Methods of Inundation Area Mapping

Conducting a dam breach analysis was a critical component of any dam safety program. Knowing the possible effects of a breach allows the Licensee to understand the potential inundation areas and downstream risks that can be happened. In this study, it was intended to map the downstream of the Mhtsab Azmati dam at the time failure in order to put a basic consideration on the saving of the live of the people who live in the flat plain downstream area and the damage of constructed infrastructures like roads, schools. To handle over this task, the RAS mapper in the HEC – RAS model and the HEC-GeoRAS tool in the ArcGIS facilitates the generation of the flood plain maps depending on the results simulated in the unsteady flow simulation from the HEC – RAS river system modeling. The flood mapping for this research was analyzed by importing the water surface elevation and velocity distribution geospatial data generated from the HEC– RAS model into Arc GIS.

3.3. Data Collection and Analysis

3.3.1 General about Data Collection

In hydrological modeling system is often faced with challenges brought about by limitations in availability of good field data sets. It is impractical to collect comprehensive data on all hydrological variables at time-scales appropriate to catchment-scale processes. Yet even with these difficulties, such information is vital in order to understand catchment behavior and response to hydrological events. Data used in this research was collected from various institutions and agencies while some were downloaded from the World Wide Websites. These included hydro meteorological data (precipitation and temperature) from the Ethiopian National Meteorological Agency, soil and stream flow data from Ministry of Water, Irrigation and Electricity of Ethiopia, design documents of the dam for the case of analysis from Tgray Water Resource Bureau, digital elevation model (DEM) 12.5m x12.5m resolution size and satellite imagery (Land sat 8) from the website <http://vertex.daac.asf.alaska.edu> and visiting of the study area to visualize and analyzing it and to take some considerations on some issues like the manning roughness coefficient are among the actions taken during the data collection processes. Quality control was also done by use of graphical, statistical methods.

3.3.2 Silent Features of the Dam

The physical characteristics of the Mhtsab Azmati embankment dam had a great role on the determination of downstream inundation map area on how much it will be affected at the time of flooding and was collected from the design document of the dam owned by the Tgray Water Resource Bureau. After collected those data, it was added to the HEC – RAS model in the inline structure option as input data to perform the dam breach analysis for the specified location. The parameters of the dam are crest width 7m, maximum height above river bed 55m, Crest Length 659m, upstream slope 2.5H:1V, downstream slope 2H: 1V, maximum reservoir level (m.a.s.l) 1515m, normal reservoir level (m.a.s.l) 1512m, gross reservoir capacity 35.750Mm³, surcharge head 3m, spillway crest length 60m, design spillway capacity 304.65 m³/s, irrigation outlet level (m.a.s.l) 1480m and broad weir spillway type as shown in the figure below.

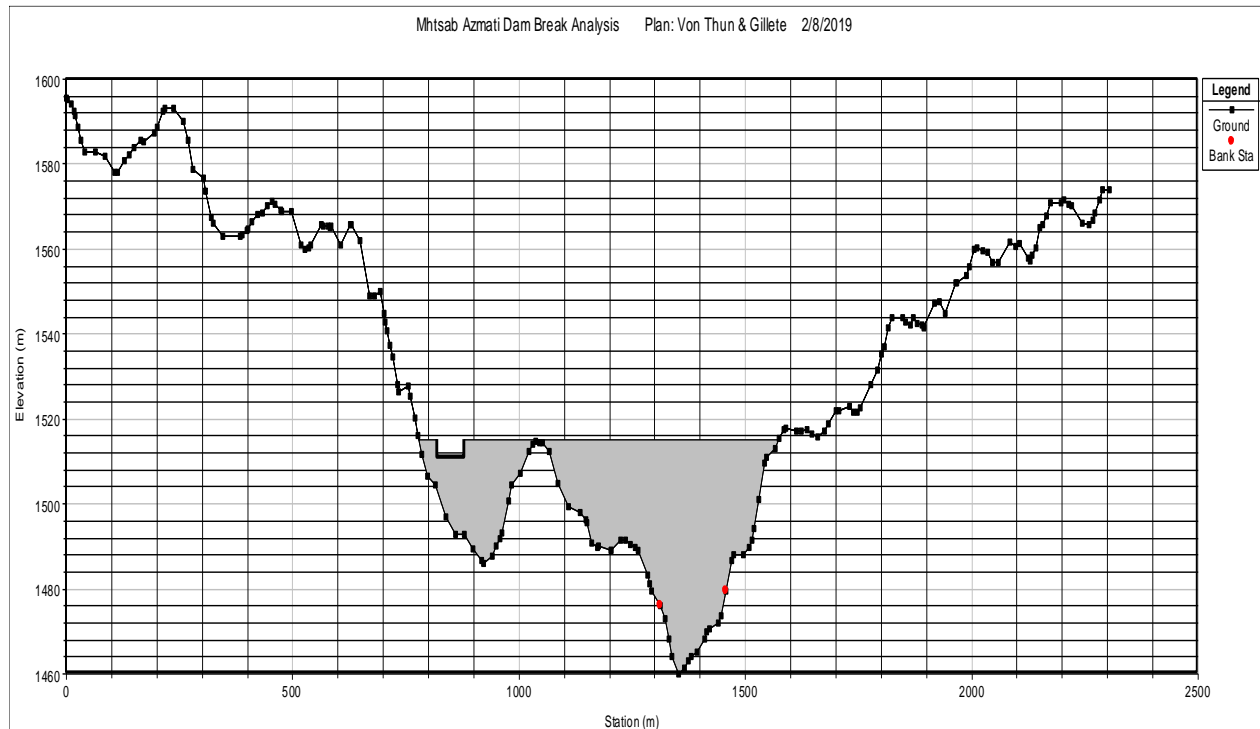


Figure 16: the dam characteristics in HEC RAS model

3.3.3 Analyzing of catchment characteristics for Mhtsab Azmati Dam

The run off generated from a given catchment was highly depend on land use and land cover, slope and soil type which covers the dendritic watershed. Stream flow can be affected by a number of aspects related to land use and can vary promptly as those factors change. Urbanization, deforestation, mining, agricultural practices and economic growth are some of the factors related to these land use changes which alter the stream flow (Venkatesh and Ramesh, 2018).

3.3.3.1 Land use and land cover of the Mhtsab Azmati catchment

Land use and land cover are the factors which highly influencing the runoff hydrograph generated from the upstream watershed of the dam site. The land use and land cover of this study was analyzed by downloading a satellite image (land sat 8) from the year of 2000 to 2018 from the website <http://earthexplorer.usgs.gov>.for the specific study area from the year of 2000 to 2018 and composing it to extract the same as catchment shape size. After the composed the image it was integrated with the signature file developed from the Google earth for the case of good representation of the land use while its classification was done using the image classification tool in Arc GIS software by the maximum likelihood classification method.

Then the main land use for the area were found as cultivated land (76.6035%), bush land (17.9549%), bare land (5.1298%) and water body (0.3118%) dominantly as described in the figure below.

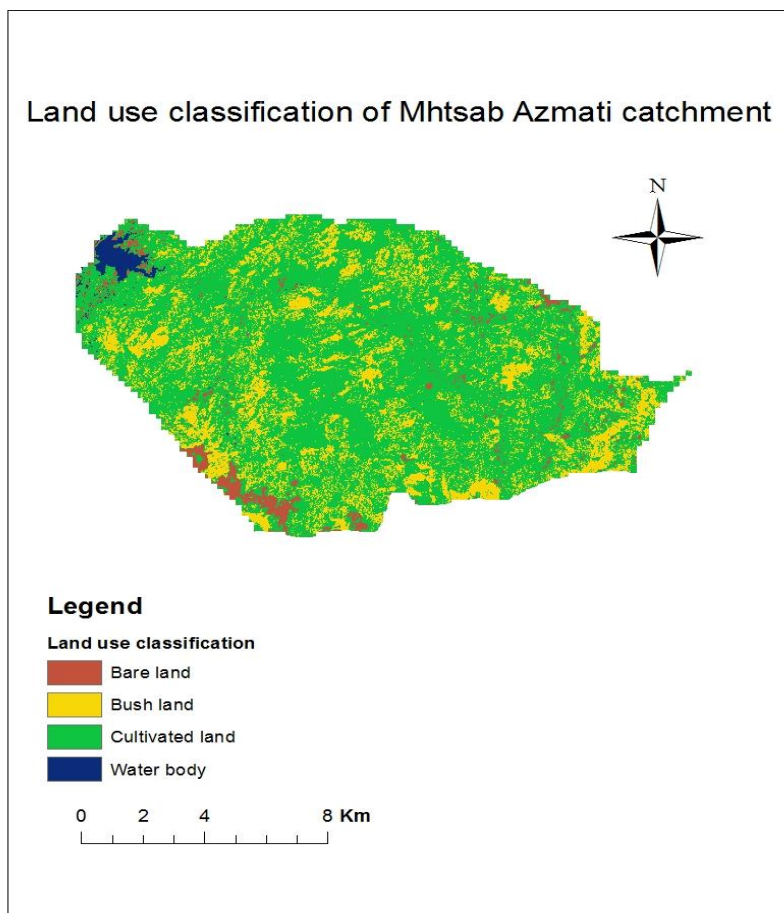


Figure 17: land use and land cover map of the Mhtsab Azmati catchment

3.3.3.2 Soil type of the Mhtsab Azmati catchment

Soil type, which collected from the Ministry of Water, Irrigation, Electricity of Ethiopia was also among the land characteristics which had an great effect on the development of river runoff hydrographs and it was assessed during this study to identify the type of soil that exist in the catchment. The type of soil, hydrological soil group and the percentage distribution of the soil on the given study catchment area was expressed as shown in the table and figure below.

Table 7: soil type and hydrological soil group (TR 55).

Soil type	Hydrological soil group	% of area coverage
Eutric Cambisols	B	53.8273
Orthic Solonchaks	B	24.3840
Leptosols	B	14.3418
calcic xerosols	B	7.4468

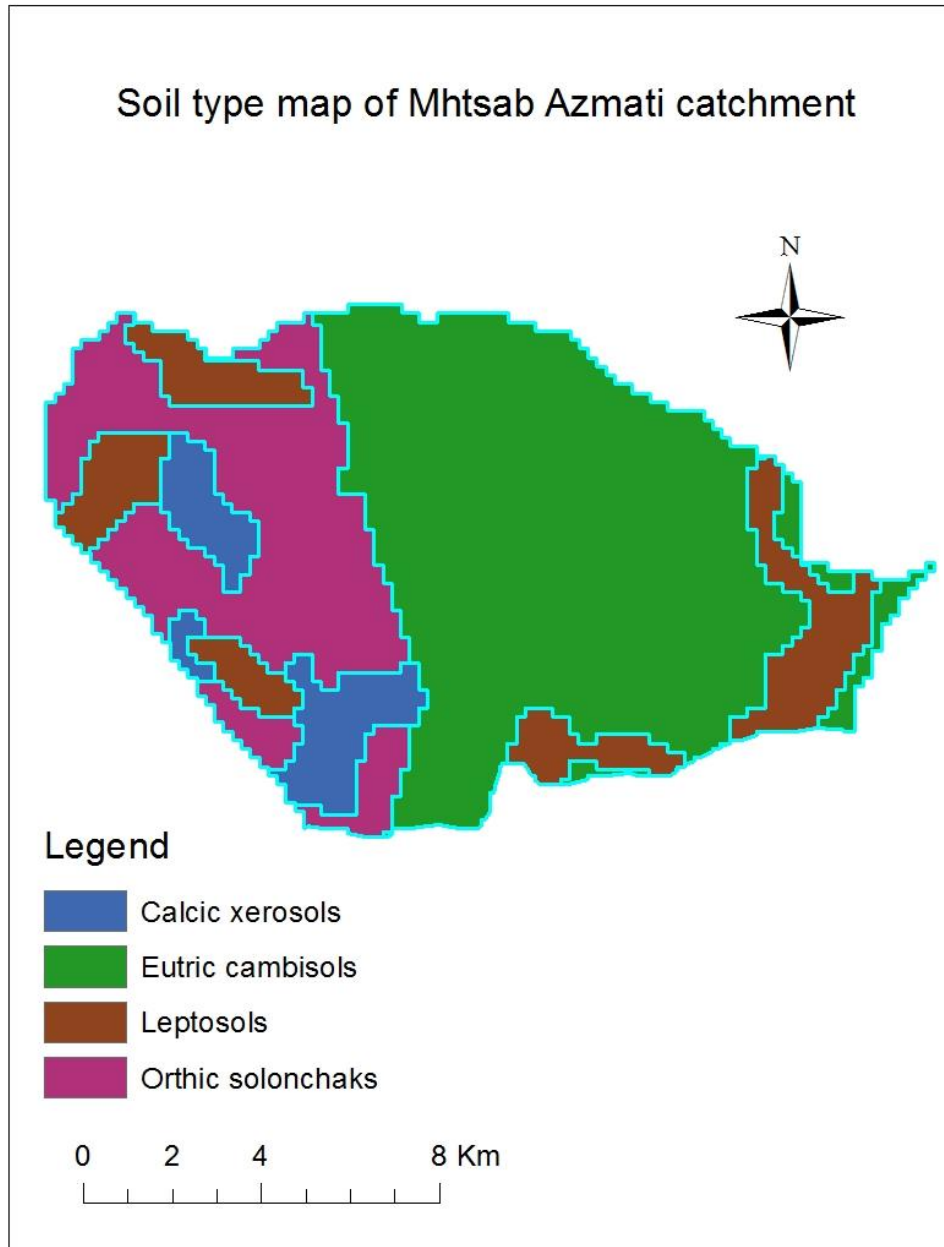


Figure 18: soil type the map of the Mhtsab Azmati catchment

3.3.3.3 The Soil Conservation Service Curve Number (SCS-CN) for the catchment

The Soil Conservation Service Curve Number (SCS-CN) method which originally developed by the US Department of Agriculture, Soil Conservation Service is widely used for estimation of direct runoff volume for a given rainfall event in a given basin (USDA, 1964). As it includes many of the factors which affecting the runoff generation like soil type, land use and treatment, surface condition, and antecedent moisture condition and incorporating them in a single CN parameter, it soon became one of the most popular techniques among the engineers and the practitioners, mainly for small catchment hydrology (Mishra and Singh, 2006).

The SCS-CN method is based on the water balance equation

$$P = Ia + F + Q \quad (3.1)$$

and on the fundamental assumption that the ratio of runoff to effective rainfall is the same as the ratio of actual retention to potential retention,

$$\frac{Q}{(P - Ia)} = \frac{F}{S} \quad (3.2)$$

Where P is the total rainfall, Ia is the initial abstraction, F is the cumulative infiltration excluding Ia, Q is the direct runoff and S is the potential maximum retention. The combination of Eqs. (3.1) and (3.2) yields the basic form of the SCSCN method

$$Q = \frac{(P - Ia)^2}{P - Ia + S} \quad (3.3)$$

Which is valid for $P \geq Ia$; otherwise $Q = 0$.

Based on a second assumption, that the amount of initial abstraction is a fraction of the potential maximum retention

$$Ia = \lambda * S \quad (3.4)$$

And Eq. (3) becomes

$$Q = \frac{(P - \lambda * S)^2}{P + (1 - \lambda) * S} \quad (3.5)$$

In Eq. (5), the initial abstraction rate is normally set to a constant value ($\lambda = 0.2$) in order for S to be the only parameter of the method. Furthermore, the potential retention S is expressed in terms of the dimensionless curve number (CN) through the relationship

$$S = \frac{25400}{CN} - 254 \quad (3.6)$$

With S, in mm, taking values from ∞ , when $S \rightarrow \infty$, to 0, when $S = 0$.

The CN for this study was computed with the help of HEC-GeoHMS tool in the Arc GIS software after the integration of the land use in a polygon form with the soil type distribution map of the specific catchment area as shown in the figure below (Soulis et al., 2009).

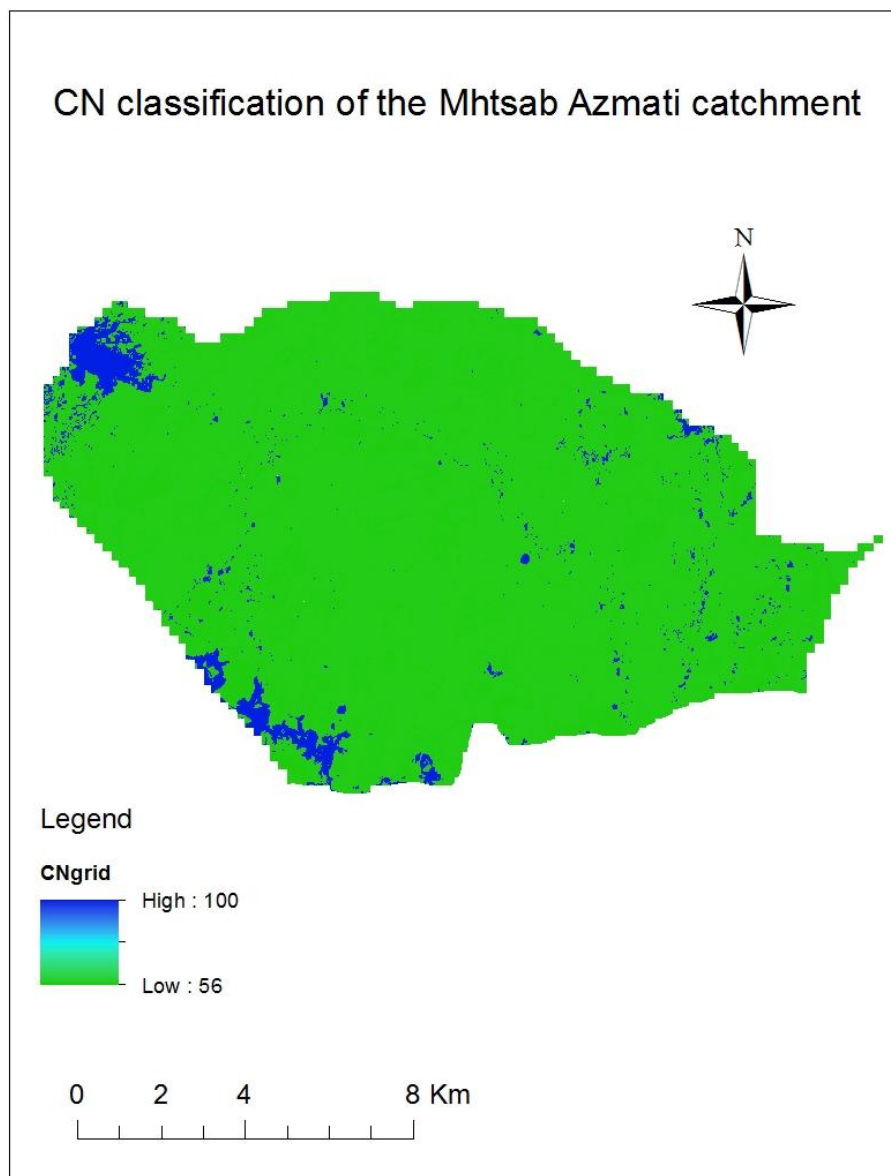


Figure 19: CN classification for the catchment

3.3.3.4 Time of concentration and lag time of the catchment (T_c)

The Time of concentration is most often defined as the time required for a particle of water to travel from the most hydrological remote point in the watershed to the point of drainage outlet whereas lag time is defined as the time difference between the center of mass of rainfall excess and the peak of the unit hydrograph. The relationship between these two parameters look:

$T_{lag} = 0.6 * T_c$ and the T_c is also computed using the relationship of:

$$T_c = \frac{227 * l^{0.8}(\lambda + 1^{0.7})}{10^5 * \sqrt{S}} \quad (3.7)$$

Where T_c = time of concentration

S = average watershed slope (in percent %)

L =length of divide (m),

λ = curve number function, which is defined as the Potential Maximum Retention and expressed as;

$$\lambda = \frac{100}{CN} - 10 \quad (3.8)$$

Whereas CN = curve number for deferent soil/land use.

The following table shows the lag time developed using the HEC- HMS model.

Table 8: Lag time for Mhtsab Azmati sub catchment

The screenshot shows a software window titled "SCS Transform[Mhtsab Azmat Basin]". It features a "Show Elements" dropdown set to "All Elements" and a "Sorting" dropdown set to "Hydrologic". Below these is a table with three columns: "Subbasin", "Graph Type", and "Lag Time (MIN)". The table lists seven subbasins (W230, W180, W220, W150, W160, W170, W140) all with a "Standard" graph type and their respective lag times in minutes. At the bottom right of the window are "Apply" and "Close" buttons.

Subbasin	Graph Type	Lag Time (MIN)
W230	Standard	13.2288
W180	Standard	10.5804
W220	Standard	14.5452
W150	Standard	11.6418
W160	Standard	10.72404
W170	Standard	6.71748
W140	Standard	8.832

3.4 Hydrological Analysis of the Mhtsab Azmati catchment

3.4.1 Estimation of Mean Areal Precipitation for the catchment

The estimation of areal rainfall is an important part of solving various hydrological problems utilizing rainfall runoff and other models (Valent and Vyleta, 2015). The mean areal precipitation for the Mhtsab Azmati catchment was detected for the calibration and validation the HEC- HMS model before the computation of the probable maximum flood for the dam breach analysis and inundation map development in the downstream area.

The meteorological stations which was collected from the Ethiopian Meteorological Agency used for this study were Adwa, Rama and Yiha. The daily precipitation and temperature of these stations were collected starting from the year of 1998 to 2016. The data consistency for the precipitation data was also checked using the double mass cure after the missing data was filled using the annual average method as shown in the Appendix A.

Thiessen polygon method is among the methods used for the estimation of the mean areal precipitation and was selected for this study as the meteorological stations which exist around the area are out of the catchment as shown in figure below. In this method the rainfall recorded at each station is given a weightage on the basis of an area closet to the station (Subramanya, 2013).

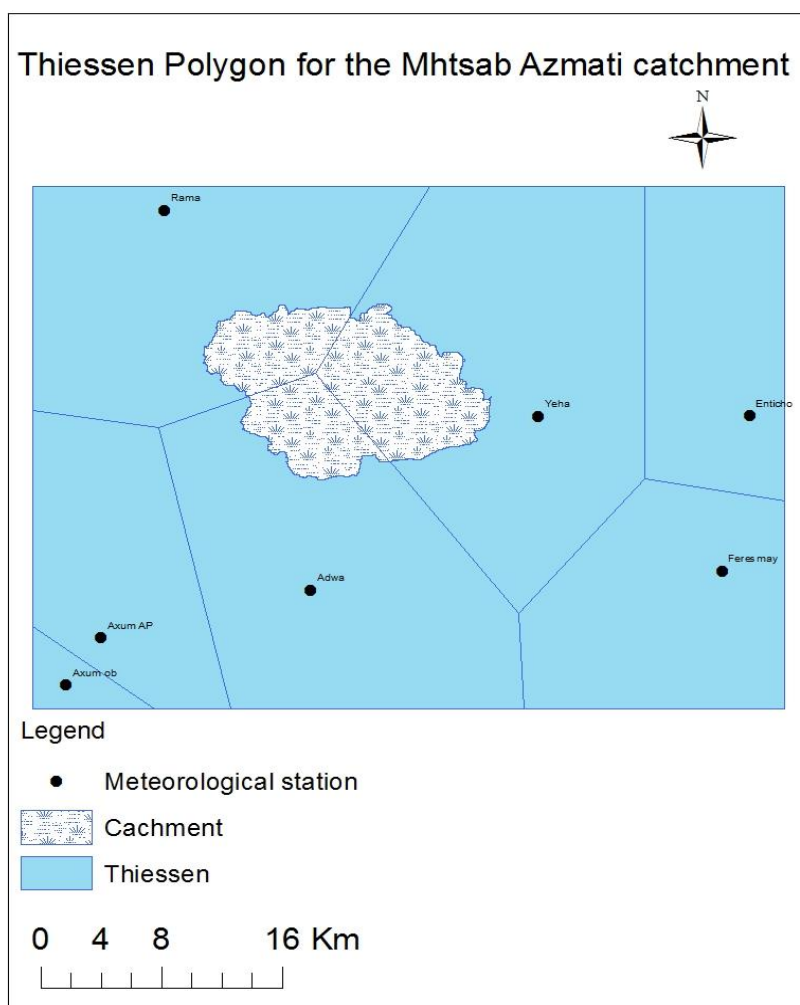


Figure 20: Thiessen polygon development for the Mhtsab Azmati catchment

As shown in the above figure, the catchment area is influenced from meteorological stations of Adwa, Yiha and Rama and their area coverage was calculated using geometry calculator in the Arc GIS and expressed as shown in table below.

Table 9: Thiessen polygon area coverage of the catchment

Name	X	Y	Z	Area (Sq.km)
Adwa	487051	1566531	1927	40.5
Rama	477363	1593081	1386	40.8
Yiha	502157	1578693	1984	75.7

The mean areal precipitation for this study was computed using the equation

$$P = \frac{A_A * P_A + A_Y * P_Y + A_R * P_R}{A_A + A_Y + A_R} \quad (3.9)$$

Where P_A = the precipitation recorded from Adwa meteorological station (mm)

P_Y = the precipitation recorded from Yiha meteorological station (mm)

P_R = the precipitation recorded from Rama meteorological station (mm)

A_A = the area contribution from Adwa meteorological station (Sq.km)

A_Y = the area contribution from Yiha meteorological station (Sq.km)

A_R = the area contribution from Rama meteorological station (Sq.km)

3.4.2 Analyzing of Stream Flow Transfer

Rainfall – runoff modelling is among the core task in hydrological sciences for using by man kindin every activities of life, but identifying behavioral parameters in ungauged catchments is still challenging (Kim et al., 2017). Prediction of stream flow at ungauged catchments requires transfer of hydrologic information (e.g., model parameters, hydrologic indices, stream flow values) from gauged to ungauged catchments. A common methods used for transfer of hydrological information from the donor to the receiver catchment are the spatial proximity and physical similarity. The spatial proximity method depends only on the spatial distance between the gauged and ungauged catchment whereas the physical similarity method depends on the physical attributes of watersheds like the forest cover type, land slope and soil type of the two catchments (Patil and Stieglitz, 2012).

The physical similarity way of stream flow transferring method was applied for this study from the Ayhda gauged stream river which is located near Axum. Depending the slope similarity, the average slope the mhtsab azamati and Ayhda catchment was found 44.5% and 33.825% respectively and both the catchments had also nearly similar in the land use and land cover. The slope, soil type and land use and land cover of the two catchments similarities were analyzed and described as in the following figures.

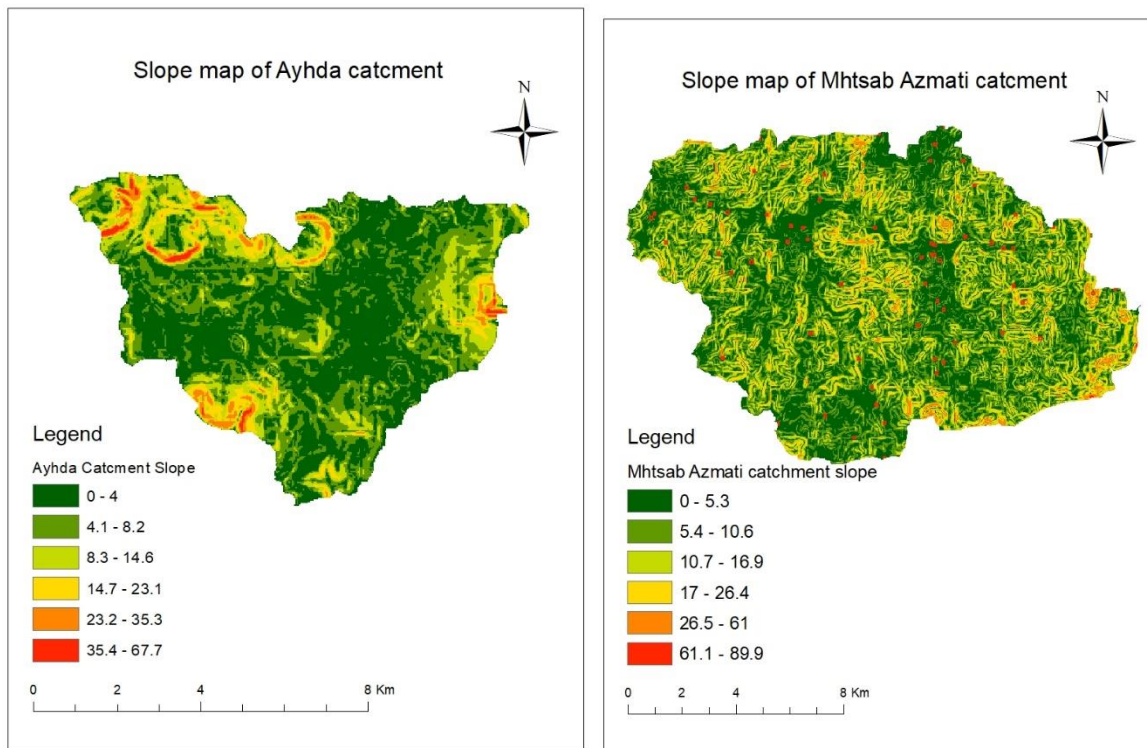


Figure 21: Slope similarity of the Mhtsab Azmati and Ayhda catchments

The soil type of the two catchments were also analyzed and had the same area coverage on their respective site as in the figure shown below.

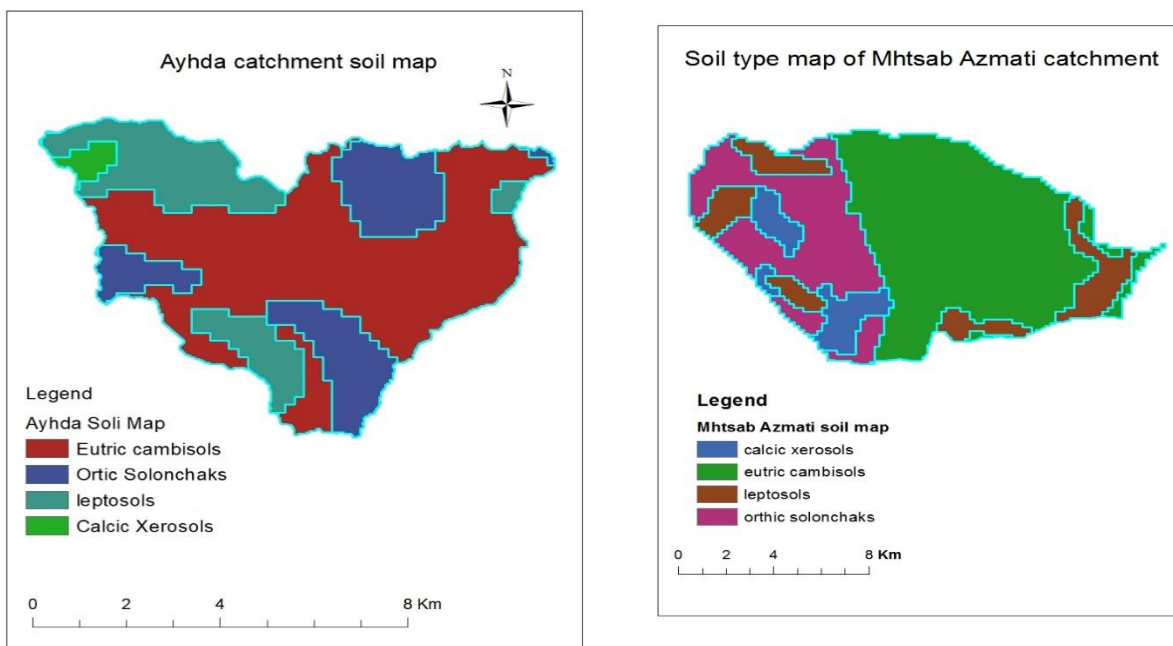


Figure 22: Soil type similarity of the Mhtsab Azmati and Ayhda catchments

According to Emerson et al., (2005) and Gianfagna, (2015) the stream flow for a site of interest (ungauged site) can be estimated by multiplying the ratio of the drainage area for the site of interest and the drainage area for a nearby stream flow-gaging station by the stream flow for the nearby stream flow-gaging station. Thus, the drainage-area ratio method is given by

$$Q_{\text{ungauged}} = \left(\frac{A_{\text{ungauged}}}{A_{\text{gauged}}} \right) * Q_{\text{gauged}} \quad (3.10)$$

Where Q_{ungauged} = the stream flow of Mhtsab Azmati ungauged catchment

Q_{gauged} = the stream flow of Ayhda gauged catchment

A_{ungauged} = the drainage area of the Mhtsab Azmati ungauged catchment

A_{gauged} = the drainage area of the Ayhda gauged catchment

The area of Mhtsab Azmati and Ayhda catchments were computed using the Arc GIS software and found 157.0 km² and 49.4 km² respectively. Then the stream flow for this study was computed using the above method for the HEC – HMS model calibration and validation cases.

3.4.3 Estimation of the potential Evapotranspiration (ET)

The term evapotranspiration (ET) is commonly used to describe two processes of water loss from land surface to atmosphere, evaporation and transpiration. Evaporation is the process where liquid water is converted to water vapor (vaporization) and removed from sources such as the soil surface, wet vegetation, pavement, water bodies, etc. Transpiration consists of the vaporization of liquid water within a plant and subsequent loss of water as vapor through leaf stomata (Zotarelli et al., 2010).

Evaporation and transpiration occur simultaneously and both processes depend on solar radiation, air temperature, relative humidity (i.e. vapor pressure deficit) and wind speed. Due to the existence of loss of water from land surface and plant portion, it was necessary to the potential evapotranspiration for the rainfall run off model calibration and validation for the long period recorded data using HEC - HMS model but ignored during the computation of the PMF as it was an event modeling.

Various derivation of the Penman equation included a bulk surface resistance, the Penman-Monteith equation was used for the computation of potential evapotranspiration for this study.

$$ET_0 = \frac{0.408 * \Delta * (R_n - G) + \gamma * \left(\frac{900}{T + 273} * u_2 (e_s - e_a) \right)}{(\Delta + \gamma * (1 + 0.34 * u_2))} \quad (3.11)$$

Where ET_0 = reference evapotranspiration rate (mm d⁻¹),

T = mean air temperature (°C)

γ = Psychrometer constant = 0.67

$$\gamma = 0.000665 * P$$

Where P = atmospheric pressure, kPa and 1000kpa at normal condtion.

Δ = Slope of saturation vapor pressure curve and was computed as

$$\Delta = \frac{4098 \left(0.6108 \exp \left(\frac{17.27 * T_{mean}}{T_{mean} + 237.3} \right) \right)}{(T_{mean} + 237.3)^2} \quad (3.12)$$

Where T_{mean} = mean daily air temperature, °C

R_n = net radiation flux (MJ m⁻² d⁻¹)

u_2 = the daily wind speed (m s⁻¹) at 2 m above the ground and was computed as

$$u_2 = u_h * \frac{4.87}{\ln(67.8 * h - 5.42)} \quad (3.13)$$

Where h = height of the measurement above the ground surface and taking 1927m for the Adwa station.

e_s = the mean saturation vapor pressure for a day and was estimated depending on the maximum and minimum daily temperature as shown below.

$$e_s = \frac{e_{(T_{max})} + e_{(T_{min})}}{2} \quad (3.14)$$

and the maximum and minimum vapor pressure respectively were computed as follows.

$$e_{(T_{max})} = 0.6108 \exp \left(\frac{17.27 * T_{max}}{T_{max} + 237.3} \right) \quad (3.15)$$

$$e_{(T_{min})} = 0.6108 \exp \left(\frac{17.27 * T_{min}}{T_{min} + 237.3} \right)$$

e_a = the actual vapor pressure that can be calculated from the relative humidity, kPa

$$e_a = \frac{e_{(T_{min})} \left(\frac{RH_{max}}{100} \right) + e_{(T_{max})} \left(\frac{RH_{min}}{100} \right)}{2} \quad (3.16)$$

Where $e_{T_{min}}$ = saturation vapor pressure at daily minimum temperature, kPa,

$e_{T_{max}}$ = saturation vapor pressure at daily maximum temperature, kPa,

RH_{\max} = maximum relative humidity, %

RH_{\min} = minimum relative humidity, %

After computing all the above parameters from the Adwa meteorological station, the daily ET_0 value was found and it was converted in to yearly value for the model calibration (1998-2008) and validation (2009 - 2011) as shown in the table below.

Table 10: The estimated evapotranspiration for the Adwa station

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1998	3.7	4.2	4.6	5.3	4.5	4.0	2.7	2.7	3.2	3.3	3.2	3.3
1999	3.3	4.5	5.0	5.0	4.5	3.9	2.9	3.0	3.4	3.0	3.4	3.1
2000	3.5	4.0	4.7	4.1	4.4	3.8	3.0	2.8	3.3	3.0	3.1	3.0
2001	3.4	3.9	4.3	4.7	4.2	3.3	2.9	2.8	3.5	3.3	3.2	3.1
2002	3.2	3.9	4.2	4.8	4.8	3.9	3.4	3.1	3.3	3.3	3.2	3.1
2003	3.4	3.8	4.2	5.0	5.1	3.7	2.9	2.9	3.3	3.5	3.1	3.0
2004	3.2	3.9	4.4	4.4	4.8	3.4	3.2	3.0	3.3	3.3	3.0	3.0
2005	3.2	4.1	4.2	4.6	4.6	3.8	2.8	3.1	3.5	3.3	3.1	3.0
2006	3.7	4.5	4.7	5.3	5.2	5.0	3.3	3.0	3.5	4.0	3.1	3.3
2007	3.6	4.3	5.1	5.3	5.2	4.4	3.2	3.2	3.6	4.0	3.8	3.4
2008	3.6	4.3	5.0	6.1	4.8	4.4	3.3	3.3	3.6	4.1	2.2	3.2
Average	3.4	4.1	4.6	5.0	4.7	4.0	3.1	3.0	3.4	3.5	3.1	3.1
2009	3.6	4.3	5.0	5.6	5.8	4.7	3.1	3.2	3.9	4.2	3.7	3.3
2010	3.4	4.1	4.8	5.2	4.6	3.5	3.1	3.3	3.5	3.5	3.2	2.9
2011	3.2	4.3	4.3	5.4	4.6	4.5	3.5	2.9	3.5	3.9	3.4	3.3
Average	3.4	4.2	4.7	5.4	5.0	4.2	3.3	3.1	3.6	3.9	3.4	3.2

3.4.4 Estimation of Probable Maximum Precipitation (PMP)

Probable Maximum Precipitation is theoretically defined as the greatest depth of precipitation for a given duration which is physically possible over a given size storm area at a particular geographical location and at a particular time of year. Estimation of PMP for the catchment of the dam helps to calculate the maximum probable flood (PMF) that can generated to dam and analyzing for the breach during the rainy season for the development of flood map in the failure time.

Number of methods such as statistical, empirical and dynamic are generally used for estimation of PMP for non-orographic or plain area regions. Statistical method utilizes time series data of

long period rain gauge stations. This method is particularly useful for making quick estimates for basins having size less than 1000 km². As the basin size of this study was within this range, it was decided to use for the computation of the PMP (Vivekanandan, 2015).

The mathematical representation Hershfield method for the one day of Mhtsab Azmati Catchment was determined as follows.

$$PMP = P_{ave} + K_m * \delta_n \quad (3.17)$$

Where PMP = the one day probable maximum precipitation

P_{ave} = mean of Annual 1-day maximum rainfall

δ_n = standard deviation of annual 1-day maximum rainfall maximum rainfall

K_m = the frequency factor which depends upon the number of observations and obtained as follows.

$$K_m = \frac{P_{max} - P_{ave(n-1)}}{\delta_{(n-1)}} \quad (3.18)$$

Where P_{max} = the highest value in the series of annual daily maximum rainfall

$P_{ave (n-1)}$ = the mean of annual 1- day maximum rainfall of n-1 observations excluding the highest rainfall.

$\delta_{(n-1)}$ = standard deviation of annual 1-day maximum rainfall maximum rainfall of n-1 observations excluding the highest rainfall.

The historical time series rainfall data collected for this study was taken from 1998 to 2016 and the above parameters were computed as shown in the Appendix B and in the table below.

Table 11: Computation of the PMP Parameters

Parameter	Stations		
	Adwa	Yiha	Rama
P_{max}	150.5	105.5	126.3
P_{ave}	60.7	51.8	57.0
δ_n	27.9	26.7	23.8
$P_{ave (n-1)}$	55.7	48.8	53.2
$\delta_{(n-1)}$	17.9	24.0	17.4
km	5.3	2.4	4.2
1Day PMP	208.2	114.8	157.0

Depending the above table, the maximum one day PMP among the three stations was 208.2 mm and converted to areal using reduction factor and was taken for the computation of the PMF using HEC –HMS hydrological model after disintegrated into 24hours.

The point rainfall of 208.1928 mm was also converted into areal precipitation in order to represent throughout the given catchment using the following relationship (Allen and DeGaetano, 2005).

$$ARF = 1 - e^{-1.1t^{0.25}} + e^{(-1.1t^{0.25}-0.259*A)} \quad (3.19)$$

Where ARF = the areal reduction dimensionless factor

t = duration time in hr

A = Area of the catchment in km²

After substituting all the parameters the in above equation, the ARF becomes 0.9962 and the areal precipitation was calculated as

$$\begin{aligned} \text{Areal Precipitation} &= 1 \text{ Day PMP} * \text{ARF} \\ &= 208.2\text{mm} * 0.9962 = 207.4\text{mm} \end{aligned} \quad (3.20)$$

The above one day areal precipitation was generally assumed to represent for the given Mhtab Azmati basin and was also disintegrated into 24 hour using the equation below before using it as input data in to the HEC HMS model.

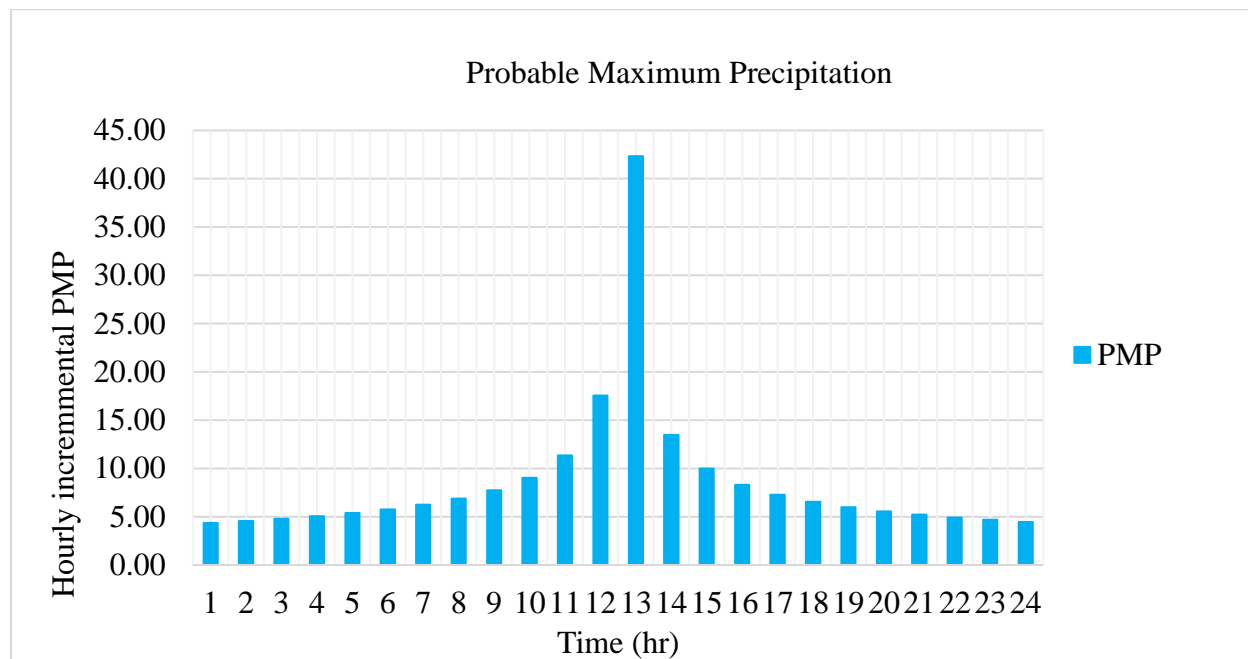


Figure 23: One day PMP of Mhtsab Azmati catchment

3.4.5 Estimation of Probable Maximum Flood

As described in the above PMP is used for the calculation of Probable Maximum Flood (PMF) which is then used for the design of hydraulic structures, such as large dams and spillways, flood control works, levees, and to size the hydraulic structures such that the risk of their failure is minimized (Micovic et al., 2015).

Accordingly, Computation of the probable maximum flood for this research helps to determine the peak flood that can occurred during the time of PMP falling in the catchment in repose of the different land parameters at dam site as catchment outlet which used as input data for the dam breach case study in the downstream in order to assess flood depth and map. The PMP entered in the selected hydrological model of HEC HMS was configured as shown below.

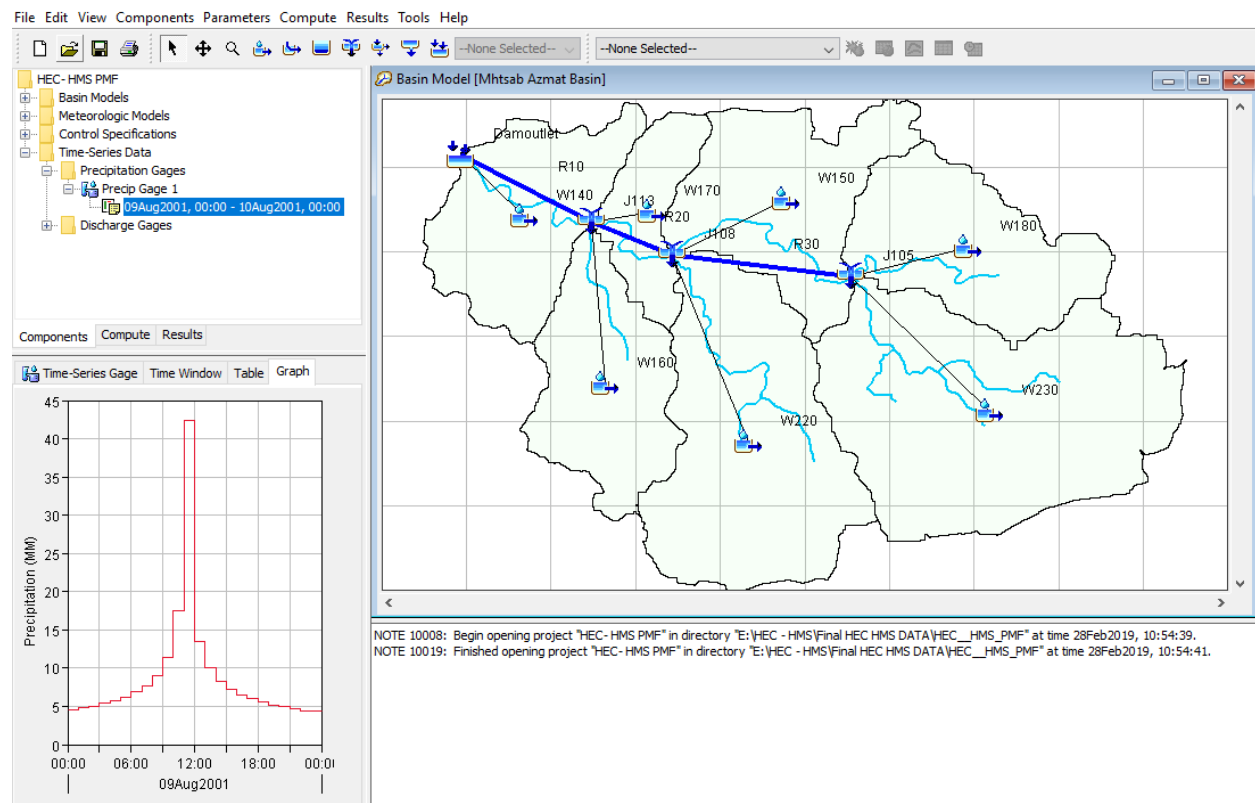


Figure 24: PMP at the dam outlet in the HEC – HMS model

3.5 Topographic Data

Topographic data was one input data during the dam breach studies and downstream inundation area. So knowing of the elevation of water surface profile outflow from the dam breach the study area helps to prepare an appropriate flooding level and inundation map while analysis the area that

can affect. Then, the primary topographic (elevation) data for river channel, left and right banks, left and right flow path center lines and each cross-section used for the analysis of Mhtsab Azmati medium dam breach was extracted using a raster cell data, Digital Elevation Model (DEM) with resolution cell size of 12.5m* 12.5m as in form of Triangular Irregular Network (TIN) in Arc GIS 10.3.1 with the help of HEC- GeoRAS extension tool along the river of 18km downstream of the dam and finally imported in the HEC- RAS model.

In addition to the above, the land cover and land use of the study area is also classified using the land cover and land use raster shape file of Ethiopia which helps on the identification of the nature of the area coverage and the impacts on the out flow hydrographs along the river due to roughness.

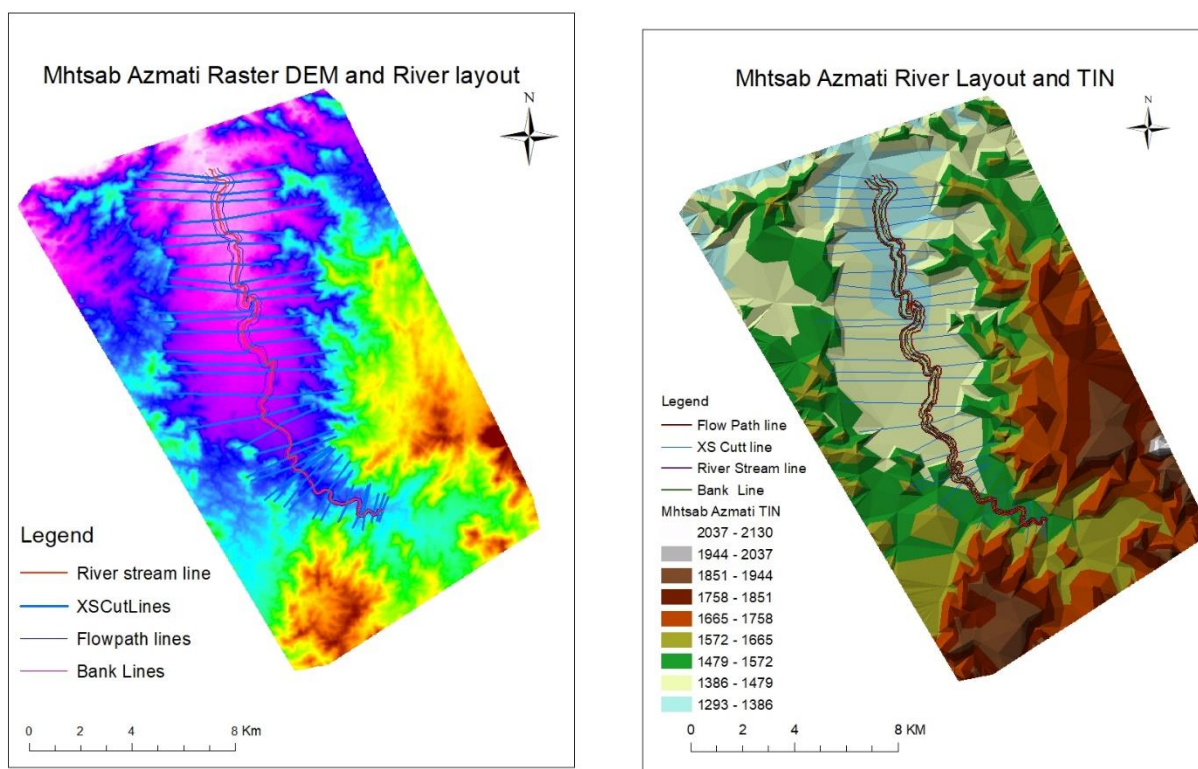


Figure 25: Mhtsab Azmati raster, TIN and river layout

3.5.1 River Geometry Data

As early stated in the above computation of the river water surface elevation during flooding had a great role while analyzing and preparing the inundation map and an emergency action plan to downstream location areas i.e. population settled in the Rama town and around the command area, the irrigation infrastructures, roads (main Adwa to Rama) etc. The elements or categories of the river geometry were discussed as below.

The river system schematic was the representative figure of the natural river which was drawn with help of Google Earth software and extracted in the "text".kml format and converted in a layer data in Arc GIS with the help of Arc Toolbox. The new developed layer was also familiarized in to a raster data (DEM) by creating a new shape file. After, all the river characteristics i.e. stream center line, cross section cut line layer, bank lines layer, flow path center line layers and land use layer are developed with the help of HEC – GeoRAS tool of Arc GIS and finally imported in HEC – RAS model.

Cross section data represents the geometric boundary of the study stream. An accurate representation of the stream channel and the overbank area (flood plain) were absolutely necessary to create an accurate hydraulic model. For this study the cross sections were located at relatively short distance intervals along the stream to characterize the flow carrying capacity of the stream channel and its adjacent flood plain. The cross section data for this study are developed at nearly constant reach length of the stations with help of HEC – GeoRAS tool in Arc GIS interface and looks like as follow.

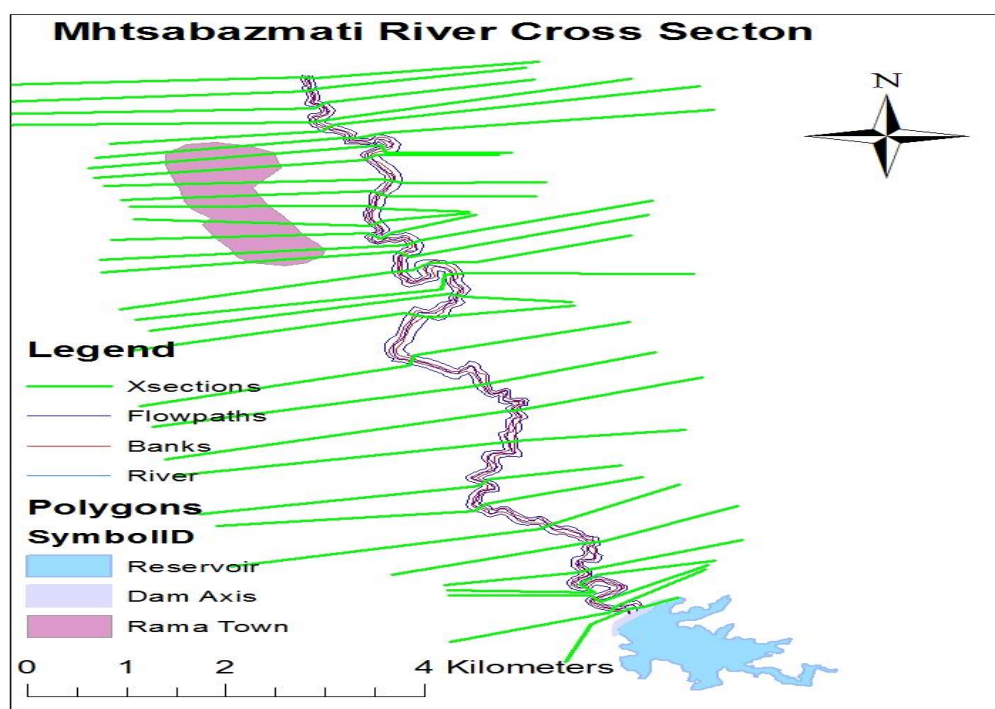


Figure 26: Cross Section Geometry of the Study River Stream

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

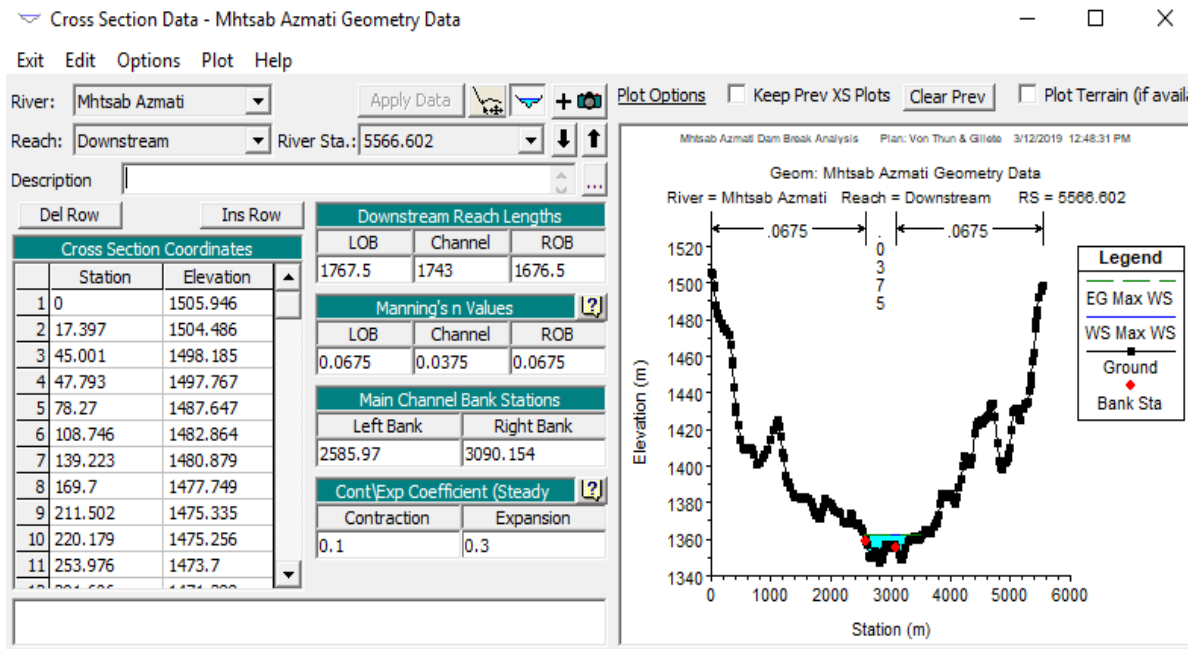


Figure 27: Cross Sectional Profile of Sample Station (St. No 5566.602)

3.5.2 Manning' Roughness (n)

Selection of an appropriate value of Manning's Roughness (n) was very significant to the accuracy of the computed water surface elevations of the natural river. The value of Manning' n was highly variable and depends on a number of factors including: surface roughness, vegetation cover, channel irregularities, channel alignment, scour and deposition, obstruction, size and shape of the channel, stage and discharge, seasonal changes, temperature, suspended and bed load materials (USACE, 2016).

The flood plain of the study area is mostly characterized by the coverage of intensively cultivated around the river study downstream of the dam, moderately cultivated, open bush land and open grass land on the upstream of the dam site. Depending on these characteristics of flood plain, the manning's n values for the cultivated stream channel downstream of the Mhtsab Azmati dam ranges from 0.025 to 0.045 and 0.035 to 0.1 on the left and right bank of river with an irregular and nearly wide top width to reflect the dynamic and extreme nature of a dam breach flood wave as well as different material types within the channel and bank sides respectively as shown in the figure below. Manning's n-values were based on published values for similar conditions (Yen, 1992), (CHOW, 1959).

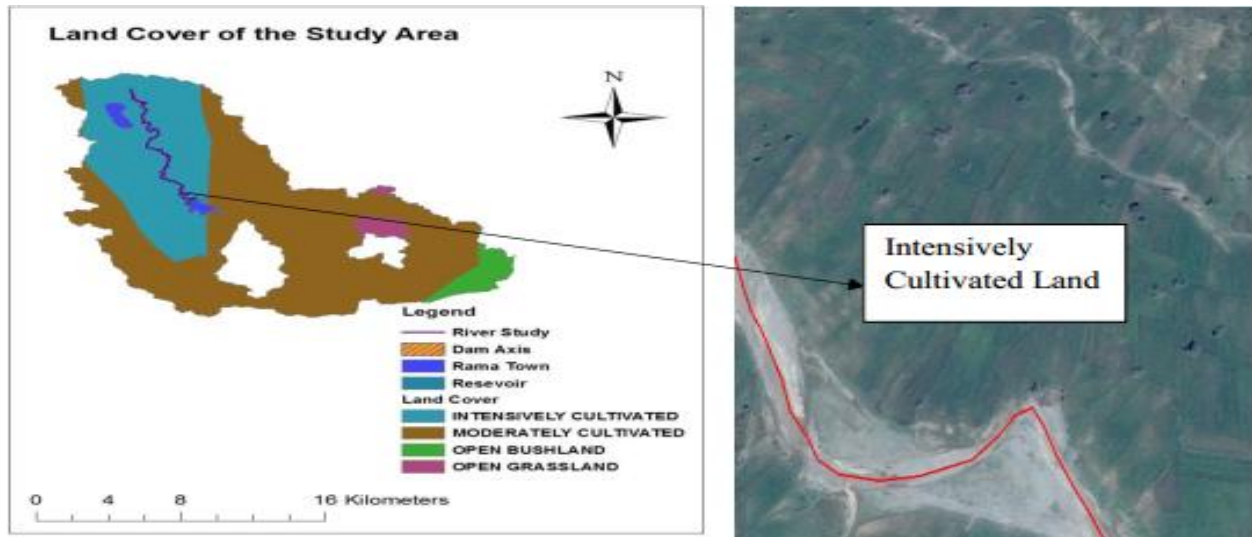


Figure 28: Land cover of the study area

The river channel manning roughness coefficient (n) for the river study was related with calibrated n values developed by (Barnes, 1967) on the left side figure and taken as 0.035 for specific site.



Figure 29: River channel bed material selection

3.6 Boundary conditions

Upstream and downstream boundary conditions were among the basic issues in order to compute the unsteady flow analysis of the river system. The unsteady component of the HEC-RAS can perform subcritical, supercritical or mixed flow regime computation using different external boundary conditions i.e flow hydrograph, stage hydrograph, normal depth, rating curve and stage/flow hydrograph. In this study mixed flow regime simulation is selected in the HEC-RAS model.

3.6.1 Upstream boundary condition

The reservoir area upstream of the dam can either be modeled with cross-sections or by using storage area. If cross-sections were used, then HEC-RAS will perform full unsteady flow routing through the reservoir pool and downstream of the dam. If storage area was used, HEC-RAS uses level pool routing through the reservoir and unsteady flow routing through downstream of the dam. For this study the upstream end river cross section is considered as external upstream boundary condition for dam breach simulation.

Among the different possibilities which could be an upstream boundary conditions, the flow hydrograph at the river cross section of 22017.1 was selected for this study and the probable maximum flood that generate from the catchment was used for the dam breach analysis and its hydrograph was as shown below.

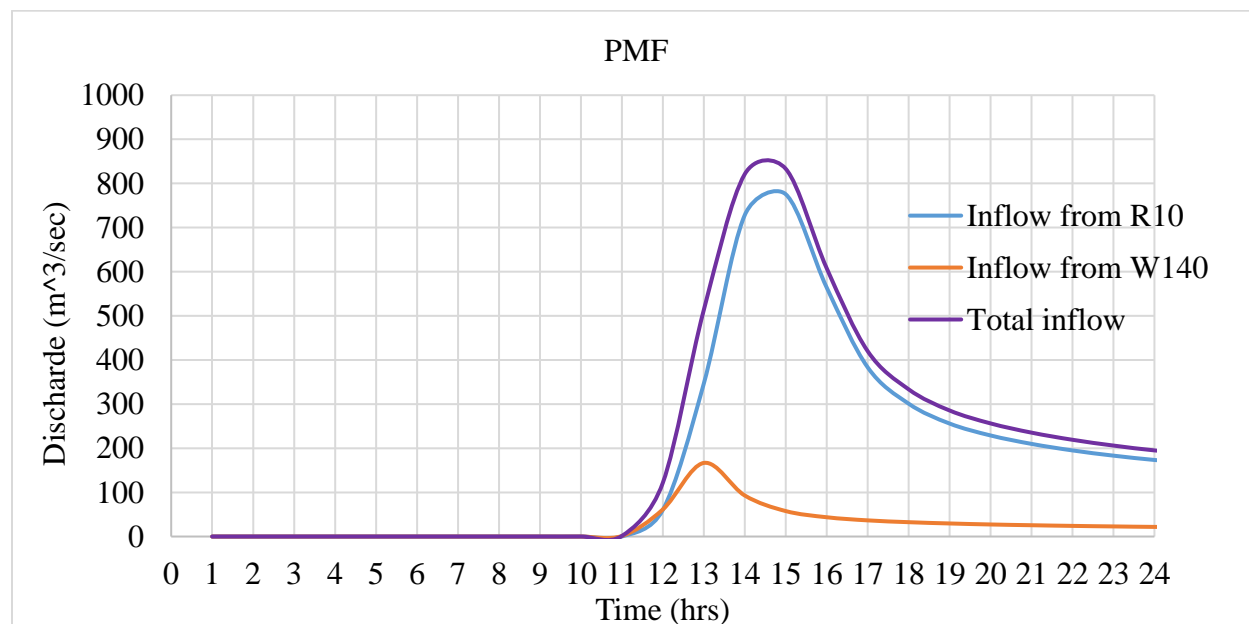


Figure 30: Inflow Hydrograph (1 day PMF)

3.6.2 Downstream boundary conditions

For the study of Mhtsab Azmati river system, fixing of the downstream boundary condition had a great role on the unsteady flow analysis computations. Among the different possibilities of boundary conditions normal depth was considered as a downstream boundary condition for this study case. The normal depth value 0.01315, was calculated as the average slope of the two cross – sections of the stream center line the same as the frictional slope.

3.6.3 Initial boundary condition of unsteady flow

In the case of initial conditions the actual existed parameter values of the river can affect the peak flood hydrograph which developed after breach and should be entered as initial condition in the HEC-RAS model as an input data at the beginning of the unsteady flow simulation. Among these input data, initial flow condition of the river system was considered as there was an initial flow which designed as excess flow over the spillway and the allowable seepage analyzed by the dam owner. For the Mhtsab Azmati Dam breach analysis on the two failure mode cases, the initial flow and elevation data were considered. The initial flow for the over topping was considered as the 10 % of the design capacity of the spillway as $Q_d = 304.65 \text{ m}^3/\text{s} * 10\% = 41.7 \text{ m}^3/\text{s}$ (Brunner, 2010). The dam breach was analyzed for the worst case flood and allowable seepage flow $Q_s = 264,920 \text{ m}^3/\text{year}$ ($0.0084 \text{ m}^3/\text{sec}$) for the piping case was also considered.

Chapter 4

4. Results and Discussion

4.1 Hydrological results of the Study

The hydrological assessment of the Mhtsab Azmati was conducted during the study for the computation of the probable maximum flood which helps for the dam breach analysis and preparation of an emergency action plan for the downstream inundation area. The one day probable maximum flood

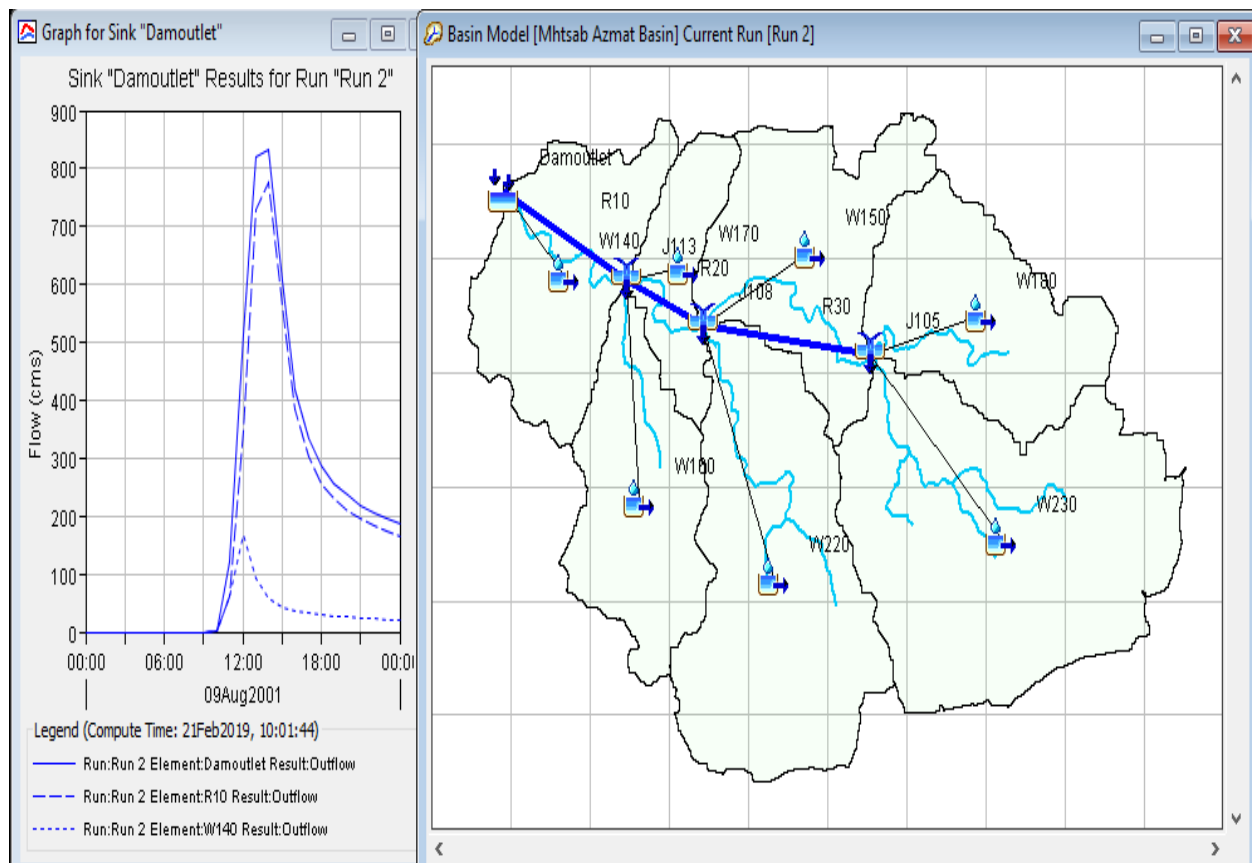


Figure 31: the sub- basins of the Mhtsab Azmati catchment

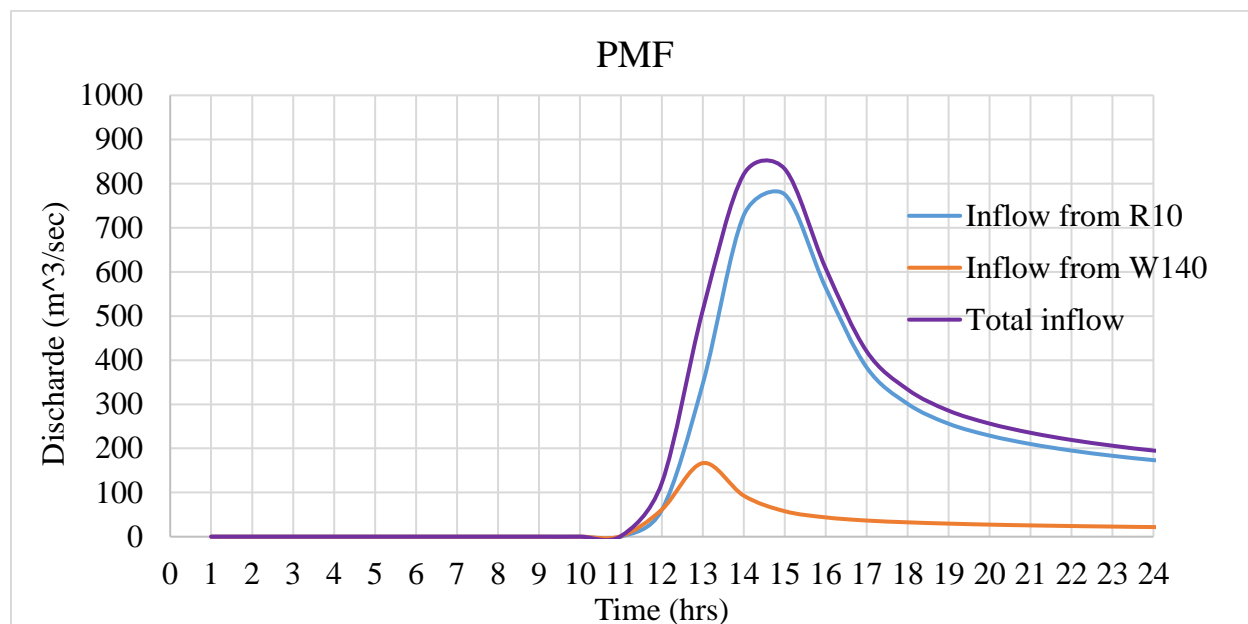


Figure 32: PMF generated from the catchment using HEC – HMS

4.1.1 Calibration and Validation of HEC HMS Model

Model calibration is a systematic process of adjusting model parameter values until model results match acceptably the observed data. In the precipitation-runoff models, objective function measures the degree of variation between computed and observed hydrographs and the calibration process finds the optimal parameter values that minimize the objective function. Calibration can either be manual or automated (optimization). Manual calibration relies on user’s knowledge of basin physical properties and expertise in hydrologic modeling. In the automated calibration model parameters are iteratively adjusted until the value of the selected objective function is minimized (Cunderlik and Simonovic, 2004). The model calibration for this study was done using the optimization parameter from the year of 1998 to 2008 and its acceptability was checked using the Nash shuffle coefficient.

Model validation is a process of testing model ability to simulate observed data other than those used for the calibration process with acceptable range of accuracy. Keeping the calibrated model parameters constant, the model was simulated to obtain the observed and it was in the acceptable range and was also checked by the Nash shuffle coefficient with its value of 0.784.

According, (Ibarra et al., 2016) using Nash shuffle efficiency (NSE) very good simulation for $0.75 < NSE < 1.0$, good for $0.65 < NSE < 0.75$, satisfactory for $0.50 < NSE < 0.65$ and unsatisfactory

for $NSE < 0.50$ and as the NSE of this study was found 0.784, it is very good between the runoff simulation and observed value.

The calibration and validation of model helps weather the parameters can represent nearly the actual

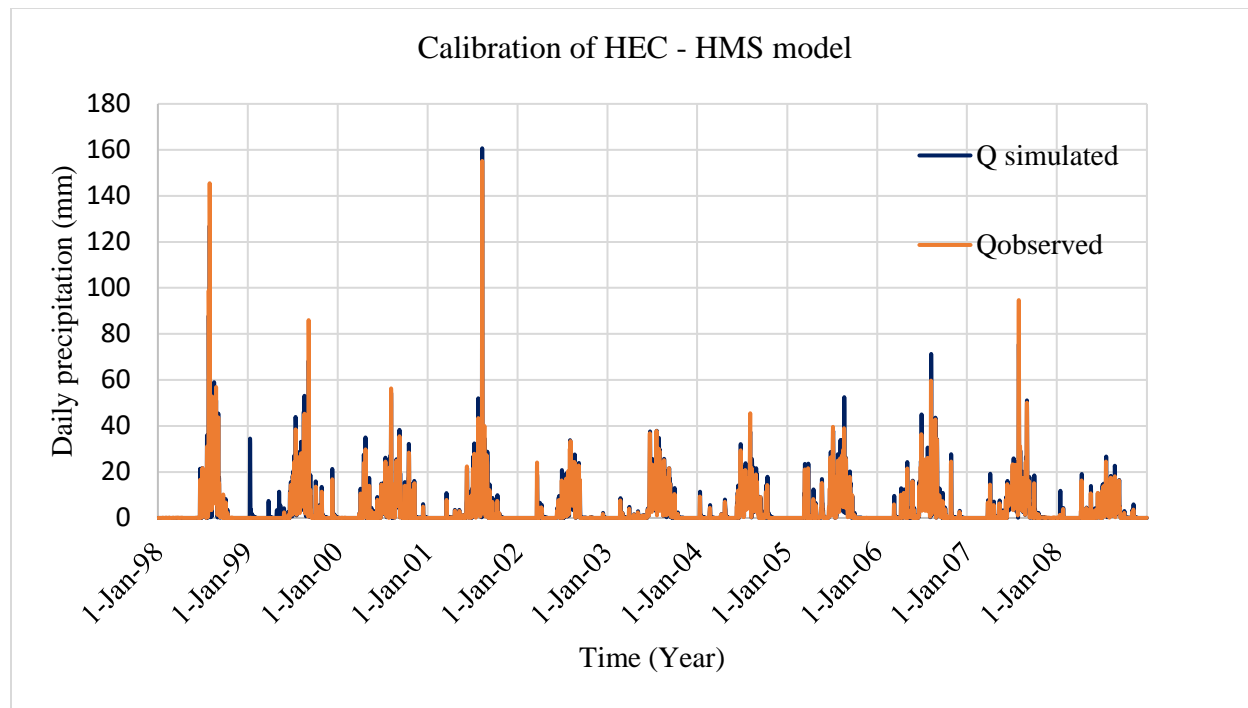


Figure 33: Calibration of HEC – HMS (1998 - 2008)

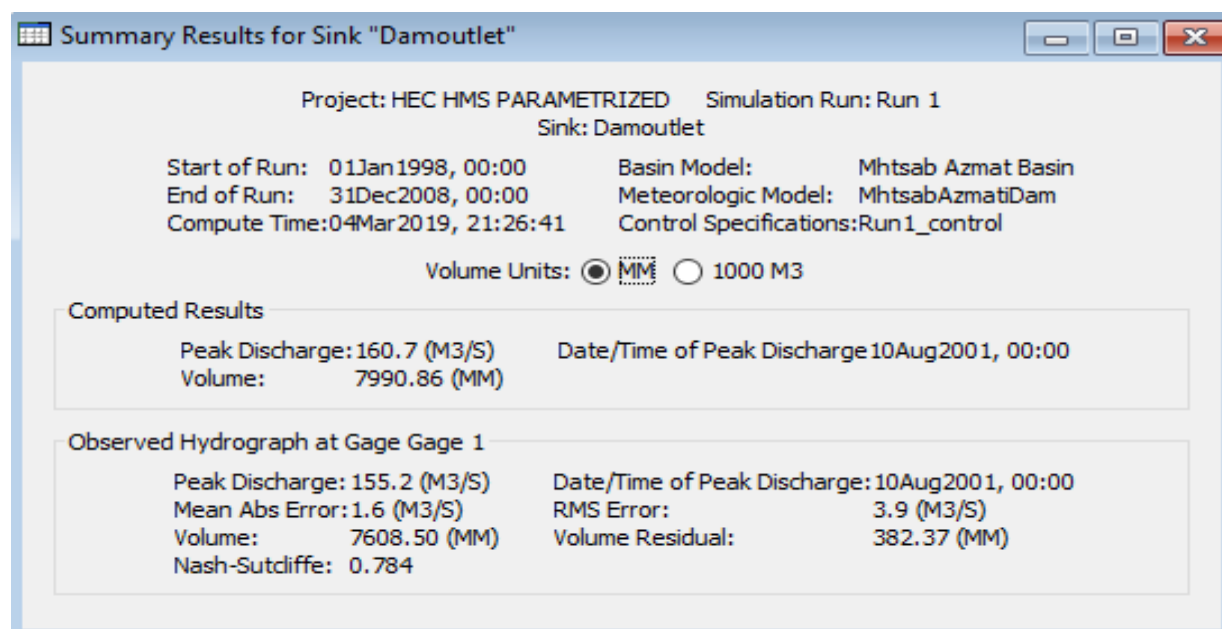


Figure 34: Calibration summary output of HEC – HMS model (1998 - 2008)

4.1.2 Sensitivity Analysis of the HEC – HMS Parameters

Sensitivity analysis is generally performed to understand how the model results react to changes in model parameters (Holberg, 2015). While using hydrological models for the purpose of rainfall runoff simulation, establishment of sensitivity analysis is needed to accelerate and optimize the calibration process, in the estimation of parameters and to understand the behavior of the model itself to the possible variation of the parameters more representative, which have an intrinsic error in its determination and define the dependencies of these parameters in the model solution (Ibarra et al., 2016)

During the computation of the probable maximum flood for this study, sensitivity analysis was done to identify which parameter could be highly affect for the peak discharge generated from the catchment. By considering -20%, -15%, -10%, -5%, 0%, 5%, 10%, 15% and 20% variations of the input parameters, the peak discharge was varying as in the figure shown below and among the input parameters, a change in the curve number value can highly affect the output of the simulation and the Muskingum k next to it.

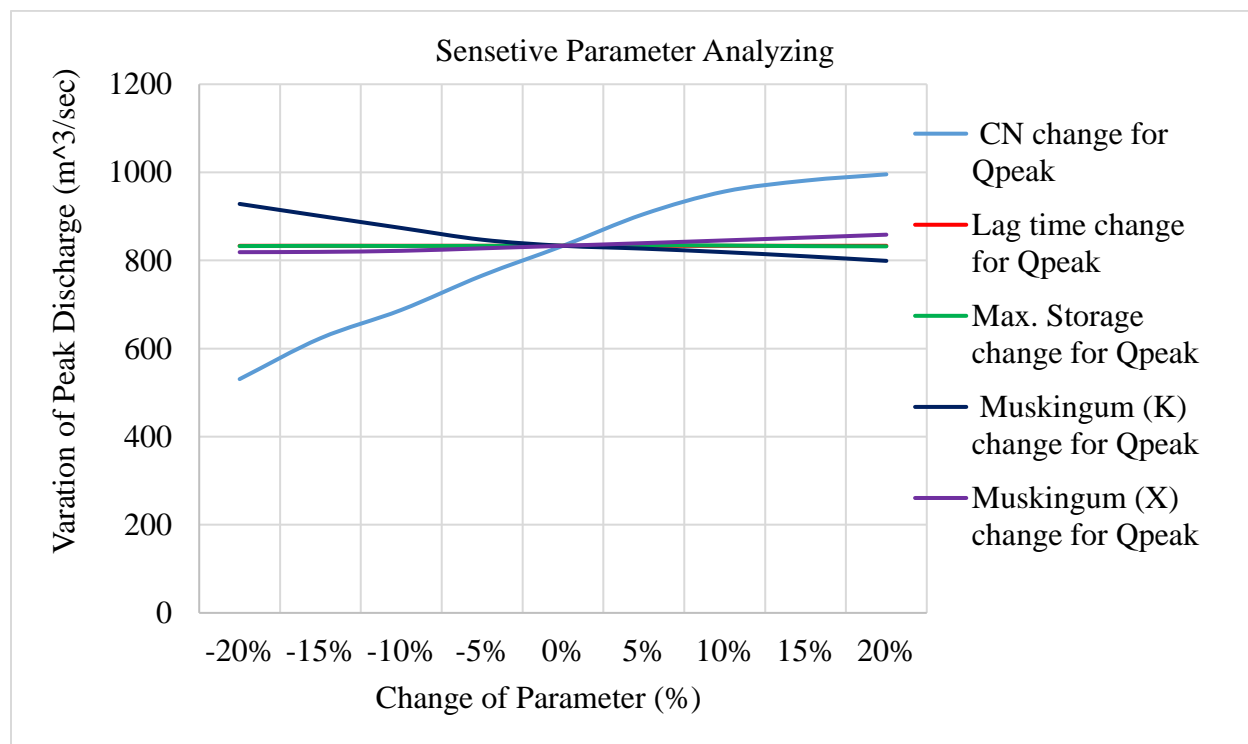


Figure 35: sensitive parameter analyzing

4.2 Dam breach results of the study

4.2.1 Dam breach Dimension and Development of Time

As described in the previous chapters, computing of the dam breach dimension had a direct relationship with the amount of discharge that release at the time of dam breaching either by overtopping and/or piping. MacDonald and Langridge -Monopolis (1984), Froehlich (1995) Froehlich (2008) and Von Thun and Gillette (1990) were among the regression equations used as an input for HEC-RAS to compute the dam breach dimension and development of time at specified elevation of the dam. For this case study for the overtopping mode of failure, the dam crest i.e 1515m is considered as the upper elevation and 1477m as the final bottom breach elevation. During the site visting time, there was a highly observable seepage around and above the irrigation outlet at an elevation of 1480m and this point is considered as the piping starting and develops down to final bottom breach 1477m for the full reservoir and dry weather condition. The dam breach dimensions for both overtopping and piping mode of failures are as described below.

Table 12: Dam breach dimensions for the overtopping mode of failure

Method	Breach Bottom Width (m)	Side Slopes (H:V)	Breach Development Time (hrs)	
MacDonald et al	15	0.5	1.18	Select
Froehlich (1995)	46	1.4	0.61	Select
Froehlich (2008)	42	1	0.57	Select
Von Thun & Gillete	131	0.5	1.01	Select
Xu & Zhang	60	0.85	1.54 *	Select

Table 13: Dam breach dimension for the piping mode of failure

Method	Breach Bottom Width (m)	Side Slopes (H:V)	Breach Development Time (hrs)	
MacDonald et al	15	0.5	1.18	Select
Froehlich (1995)	37	0.9	0.61	Select
Froehlich (2008)	35	0.7	0.57	Select
Von Thun & Gillete	131	0.5	1.01	Select
Xu & Zhang	35	0.50	1.49 *	Select

4.2.2 Unsteady Flow Simulation Results

The unsteady flow simulation of the Mhtsab Azmati embankment dam was done after all the necessary data were collected and analyzed as in the previous chapter and the dam breach dimensions were used as input to determine the amount of discharge the can generate from the dam breach for the overtopping and piping cases. After the unsteady flow simulation was done, the maximum breaching size of the dam and the maximum overtopping depth was found as shown in the figure below.

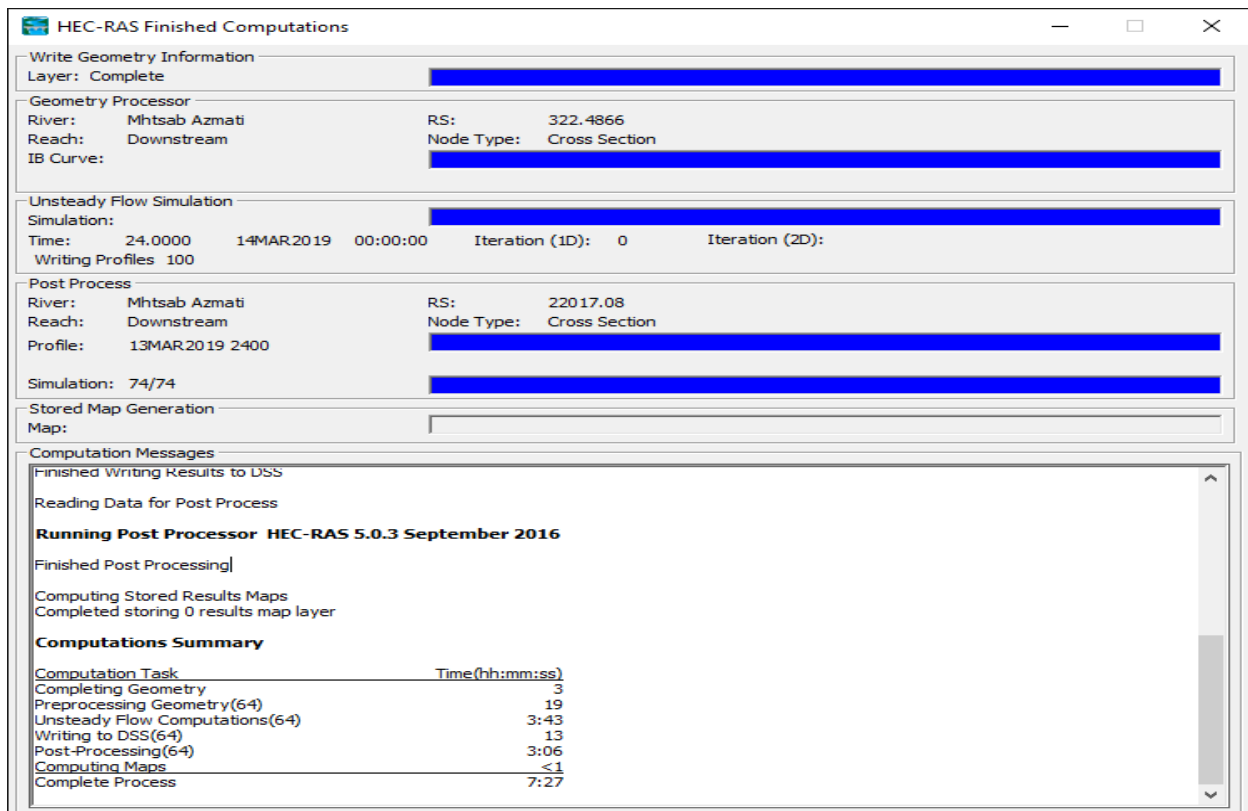


Figure 36: Unsteady flow simulation result

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

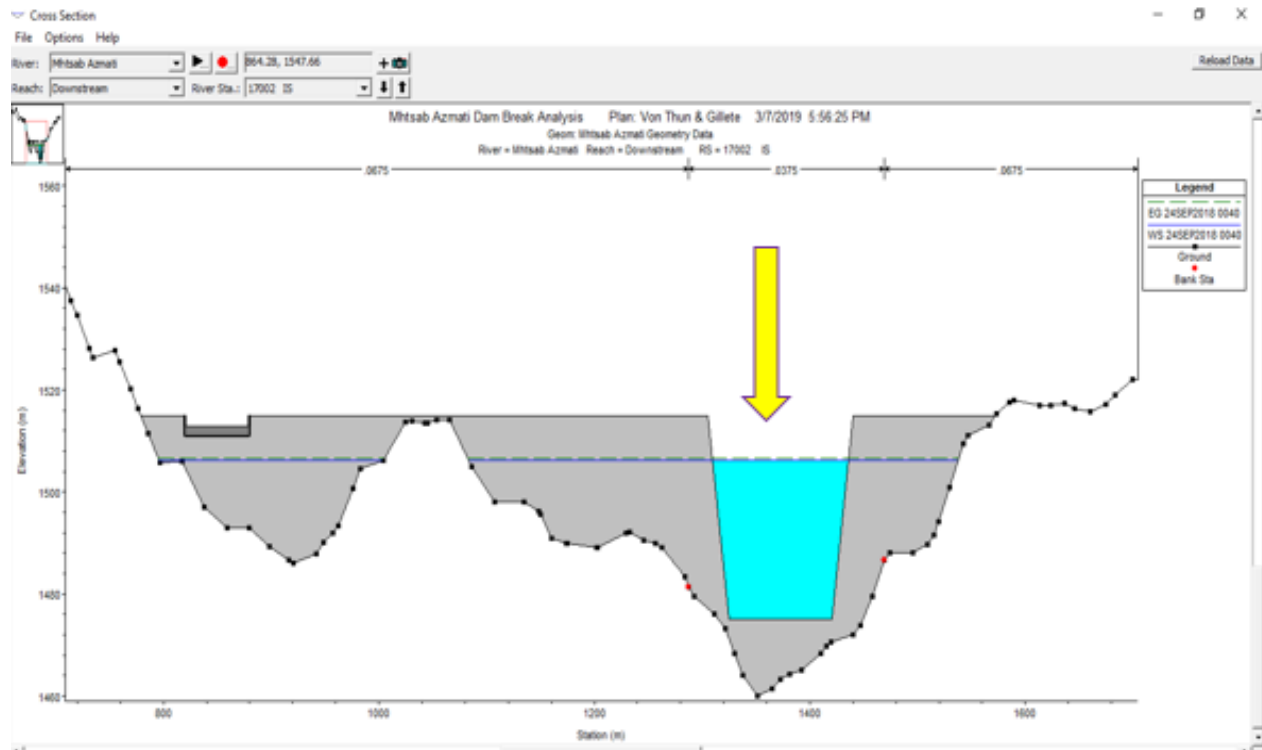


Figure 37: Dam breaching size by overtopping

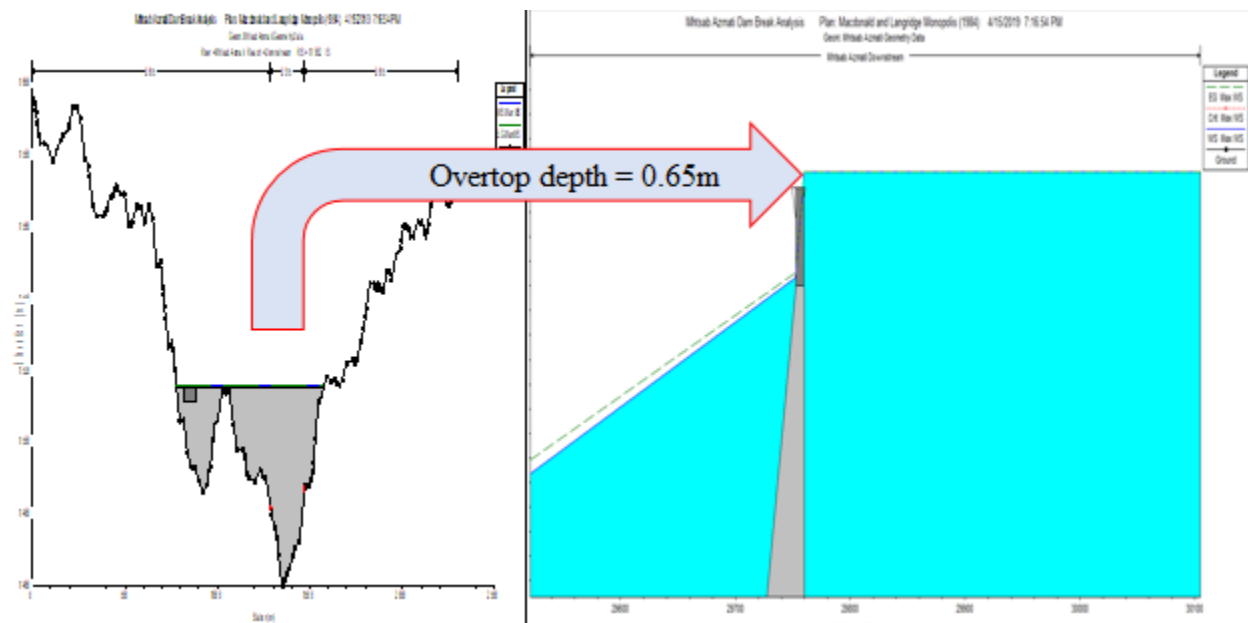


Figure 38: Overtopping depth

4.2.3 Routing of flood hydrograph at different downstream Stations by overtopping

During the conducting of this study, the failure location for overtopping mode failure was considered to be at main channel centerline of the dam (at an elevation of 1477 m.a.s.) which is

below the dead storage of the reservoir. The flood Routing hydrograph is also analyzed at different chainage points, at the dam site (inline structure), 3.46 Km, 7.92 Km and 11.44 Km downstream of the dam.

Depending the dam breach dimensions parameters computed as in the above table, different flood hydrographs were obtained at four critical stations Using Macdonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) as shown in the Appendix D.

After analyzing the different flood hydrographs at the critical stations using the four regression equations for the dam breach dimension, their comparison was also done as in figure below.

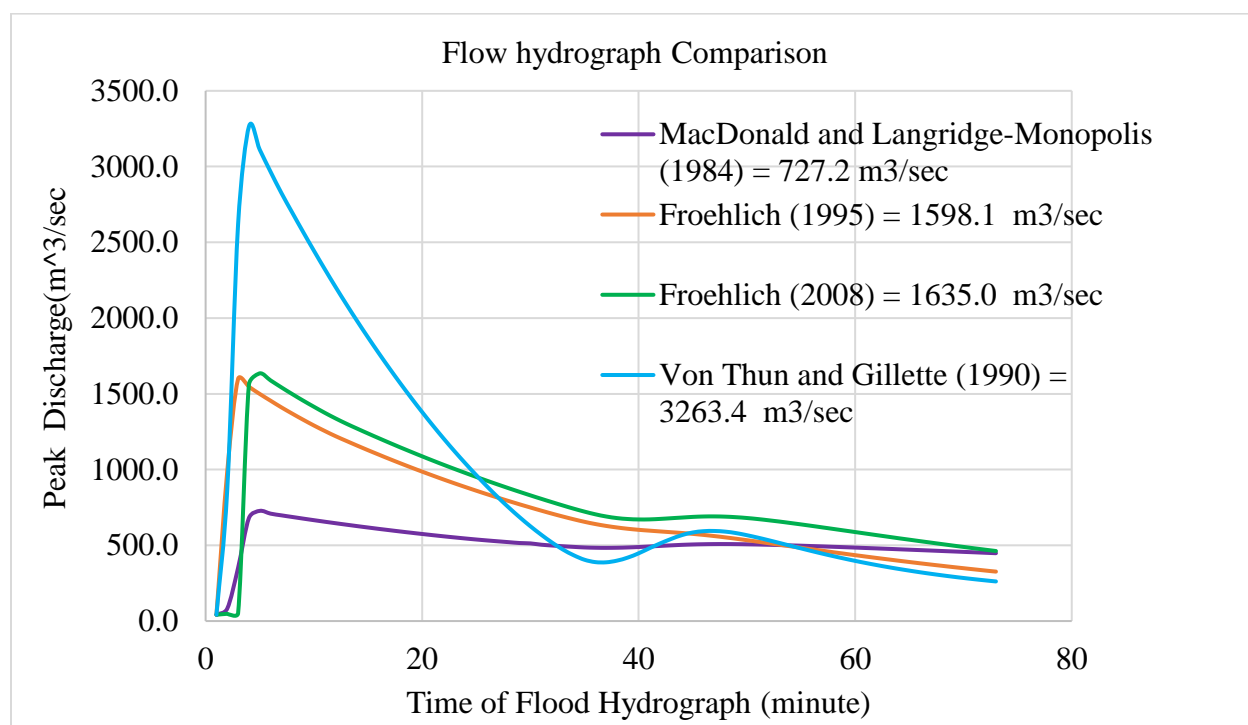


Figure 39: Downstream flood hydrograph comparison for overtopping failure mode

As shown in the above flood hydrographs, the values of 3263.4 m³/sec, 1598.1 m³/sec, 1635 m³/sec and 727.2 m³/sec are the peak flood hydrographs developed using the method of Von Thun and Gillette (1990), Froehlich (1995a), Froehlich (2008) and Macdonald and Langridge Monopolis (1984) respectively were taken at the inline structure. Except the result obtained using Macdonald and Langridge Monopolis (1984), the downstream flood hydrographs developed using the rest regression equation were beyond the reservoir capacity and the maximum run off that can develop from the probable maximum precipitation fall in the catchment. So these results were ignored for the downstream flood analysis. The downstream flood hydrograph developed using the

MacDonald and Langridge-Monopolis (1984) was also analyzed in the four critical stations as shown below.

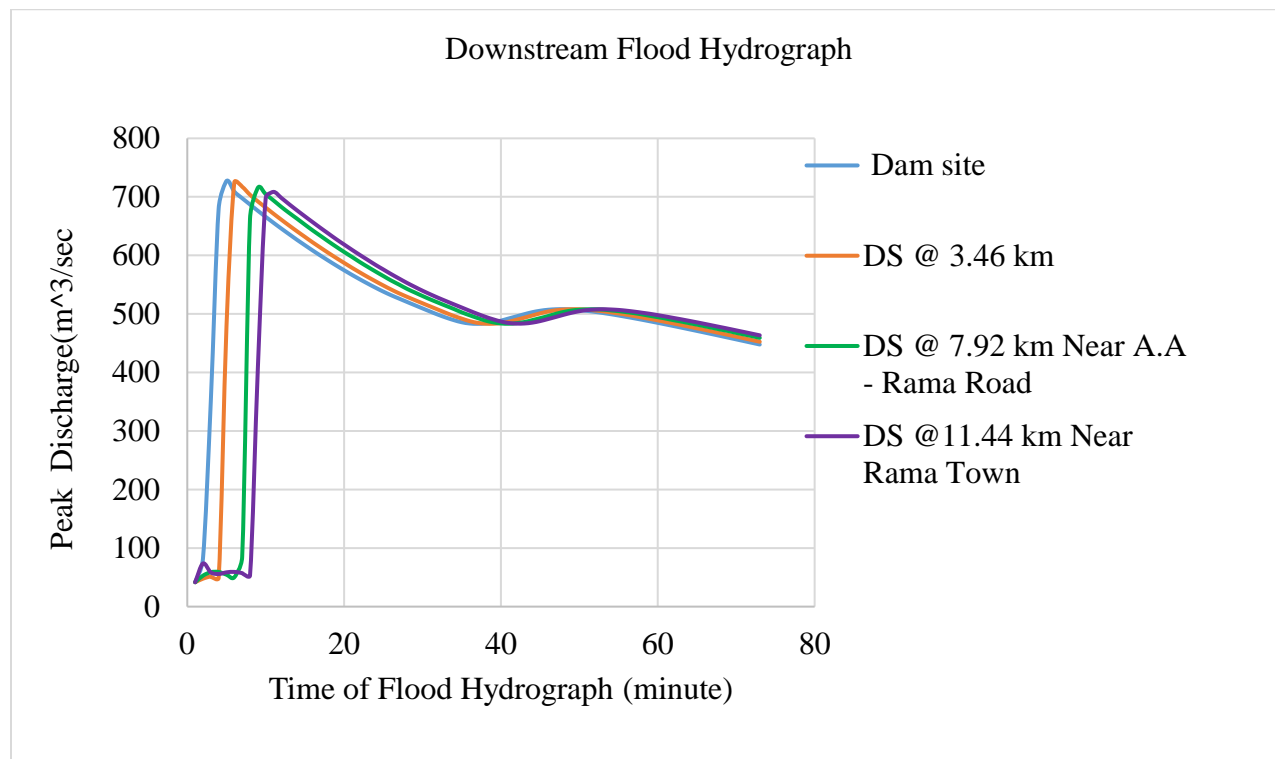


Figure 40: Downstream flood hydrograph using MacDonald and Langridge-Monopolis (1984)

4.2.4 Routing of Flood Hydrograph at Different Downstream Stations by Piping

Depending the dam breach dimensions analyzed in the above table, the downstream flood hydrographs that can develop at the four critical stations using the four regression equations as in the overtopping their result was computed as shown in Appendix E and their comparison as described below.

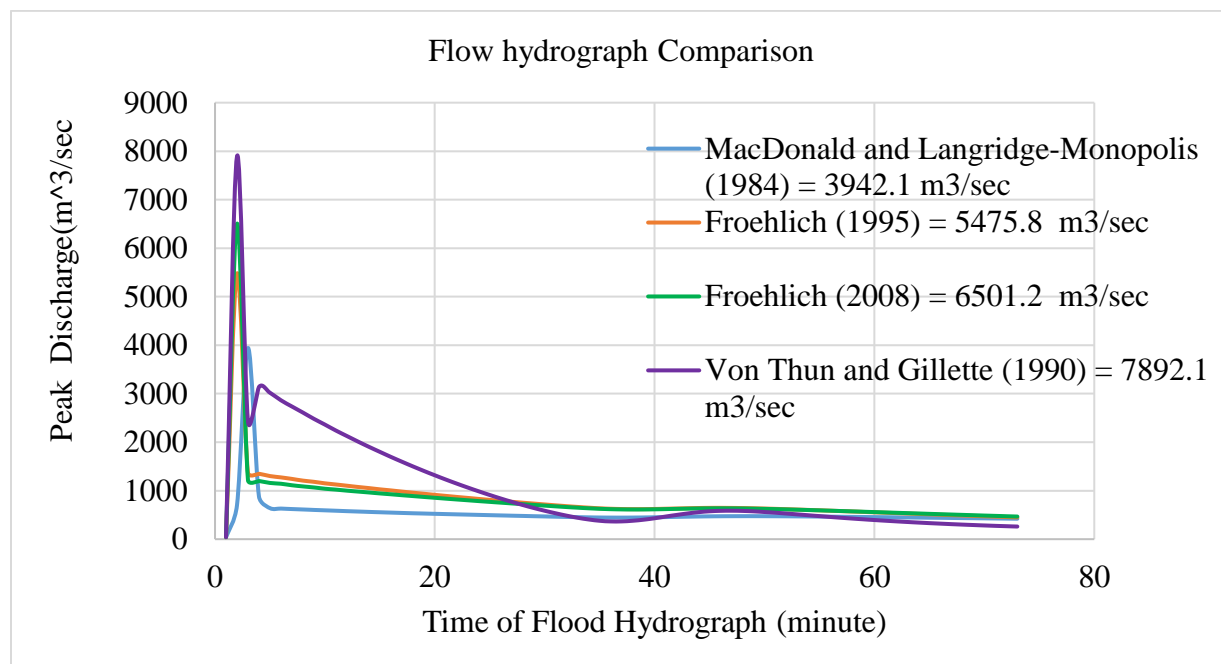


Figure 41: Downstream flood hydrograph comparison for piping failure mode

The amount of peak discharge resulted due to the dam breach through piping with the regression methods of Von Thun and Gillette (1990), Froehlich (2008), Froehlich (1995) and Macdonald and Langridge Monopolis (1984) were 7892.1 m³/sec, 6501.2 m³/sec, 5475.8 m³/sec and 3942.1 m³/sec respectively. The minimum downstream flood hydrograph developed from the dam breach through piping mode of failure was the hydrograph obtained using the Macdonald and Langridge Monopolis (1984) and was also analyzed at the four critical downstream river stations as described below.

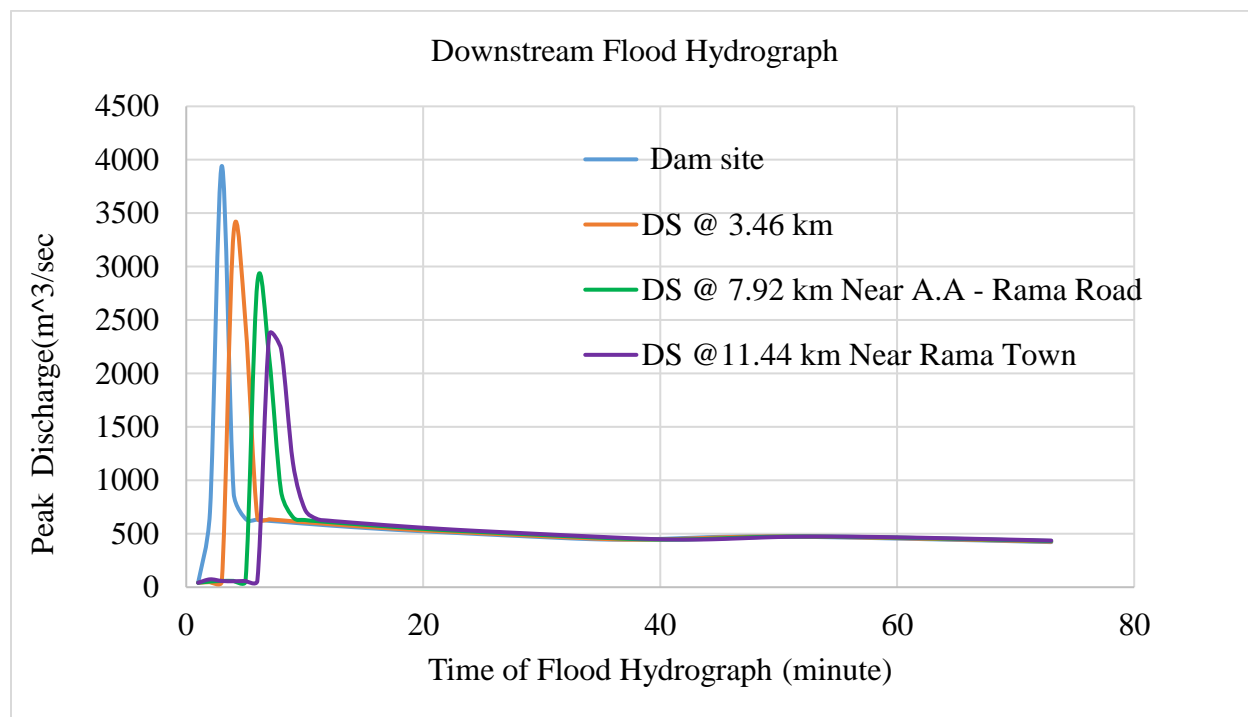


Figure 42: Downstream Flood Hydrograph using MacDonald and Langridge-Monopolis (1984) due to piping

The minimum flood hydrograph obtained on the downstream due to dam breach by the piping mode of failure was also beyond the maximum reservoir capacity and the maximum probable maximum flood that could be generated from the upstream catchment. Due to this upper limitation, the downstream flood hydrographs developed by this mode of failure were ignored from the downstream flood prone area studies.

4.2.5 Comparing the peak flood discharge with the experienced envelope curves

As the above result shows, the downstream flood hydrograph developed by the regression equation of MacDonald and Langridge -Monopolis (1984) in the overtopping mode of failure was taken for the downstream river flood analysis. This peak discharge 727.2 m³/sec which attained after 00:40:00 hrs and was also checked using the experienced outflow envelope curves for the reasonableness and reality of the research result as it is developed from different dam breach regression equations for the case of comparing by several researchers (Gray, 2014).

The peak flood discharge of the study is located below the peak envelope curve as shown in the figure below and was taken for the further analysis.

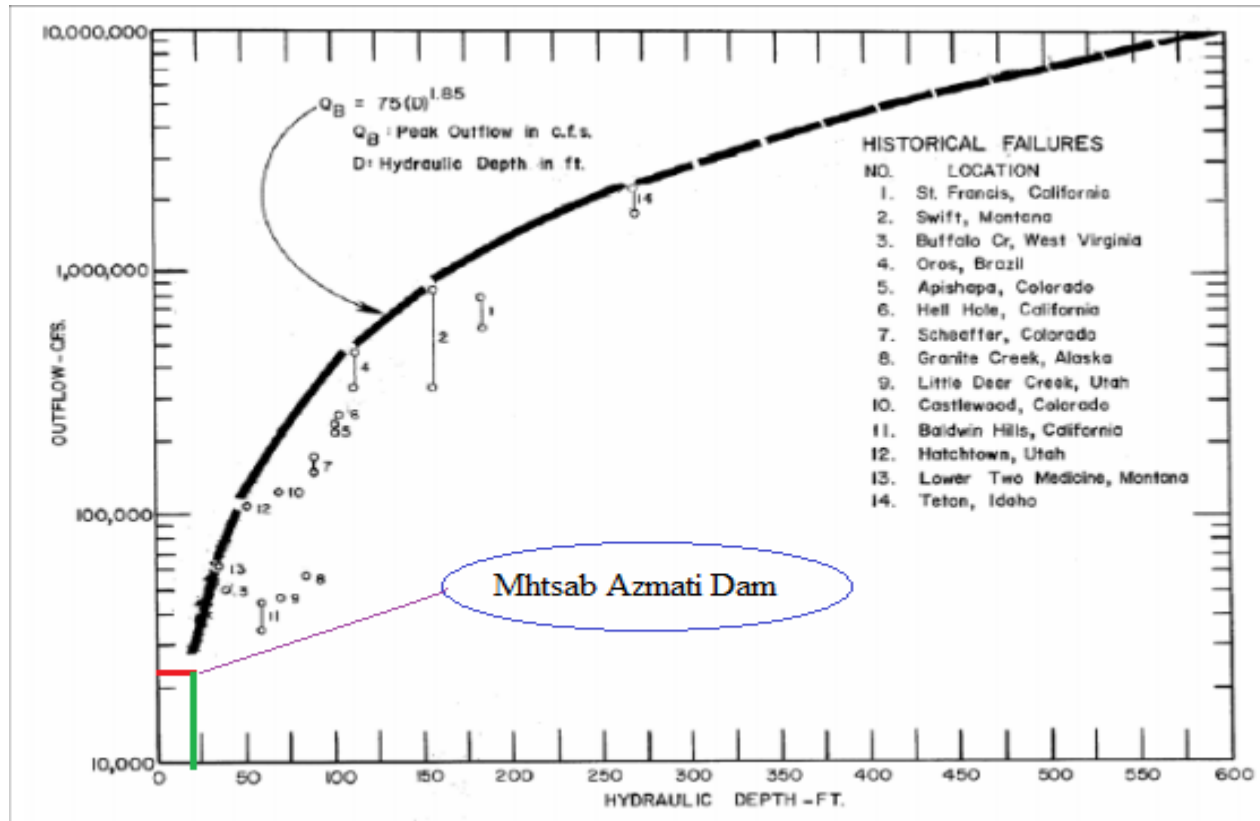


Figure 43: Location of Mhtsab Azmati dam on envelope of experienced outflow rates from overtopping

4.2.6 Longitudinal and Cross sectional profile of the study River

The longitudinal profile of the Mhtsab Azmati River contains the energy grade elevation, the water surface elevation, the base elevation and the main channel distance from the upstream boundary condition to the downstream boundary condition. At the inline structure the, the maximum flood profile of the study has an maximum channel depth of 55.65m, energy grade line 1515.65m, maximum water surface elevation of 1515.65m, top width of 799.79m and a minimum channel elevation of 1460m as shown below.

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

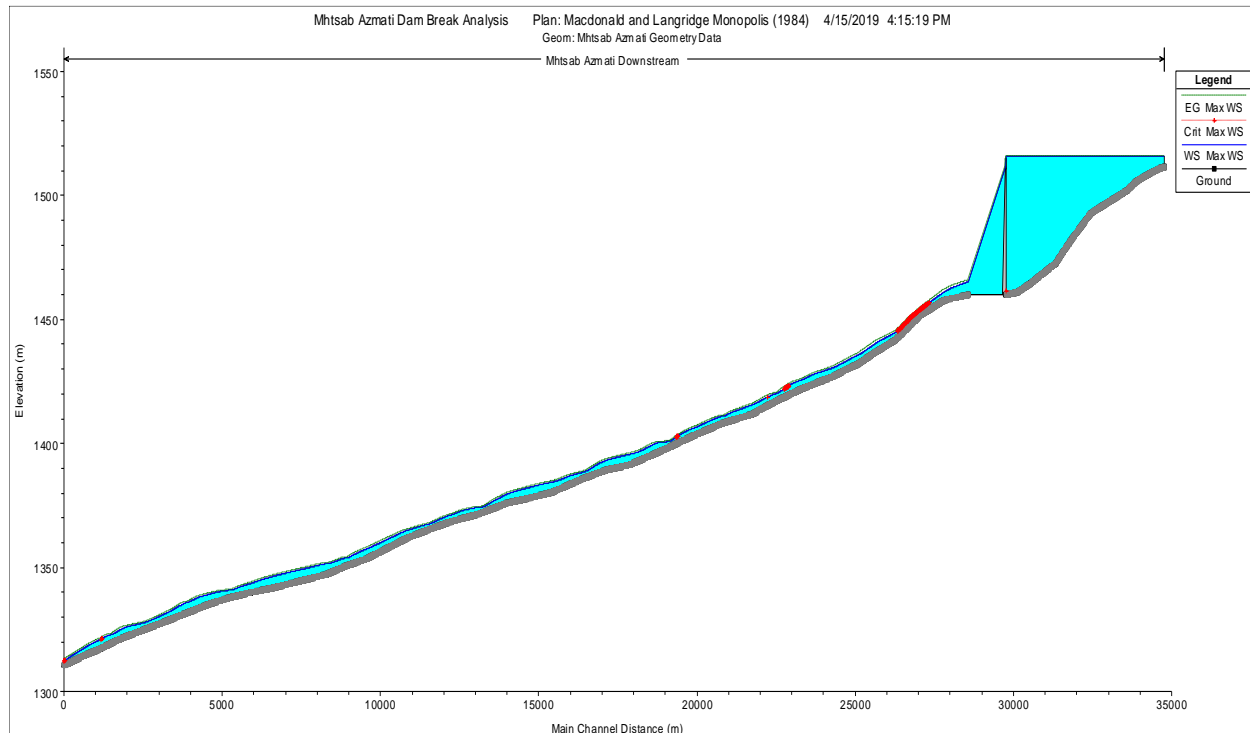


Figure 44: Longitudinal bed profile of Mhtsab Azmati River

The river cross sections top width and depth at the critical locations i.e at the inline structure, 3.46 Km, 7.92 Km and 11.44 Km downstream of the dam were also analyzed. The top width of the flood at those location were also found that 799.79m, 281.81m, 469.25m and 265.15m respectively to the downstream. The maximum depth of the flood at the locations were found 55.65m, 3.43m, 4.5 m and 4.28m respectively to the downstream as shown in the figure below.

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

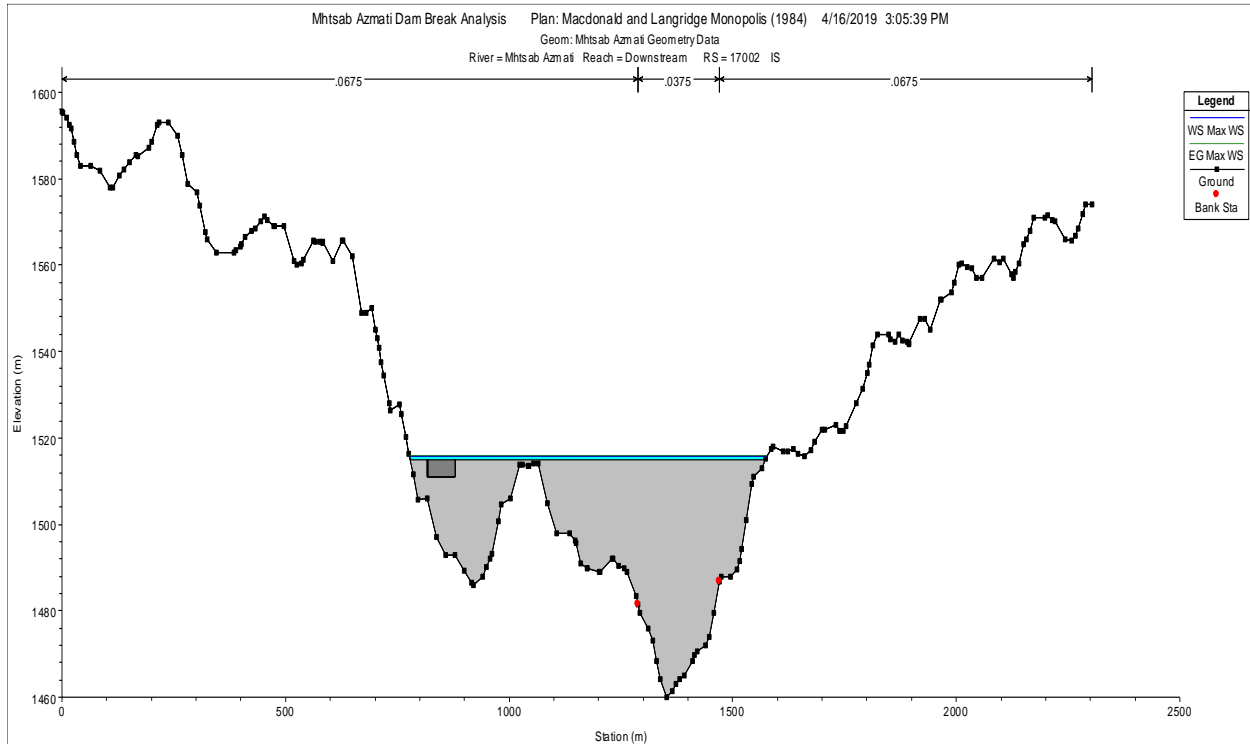


Figure 45: River cross section at the inline structure

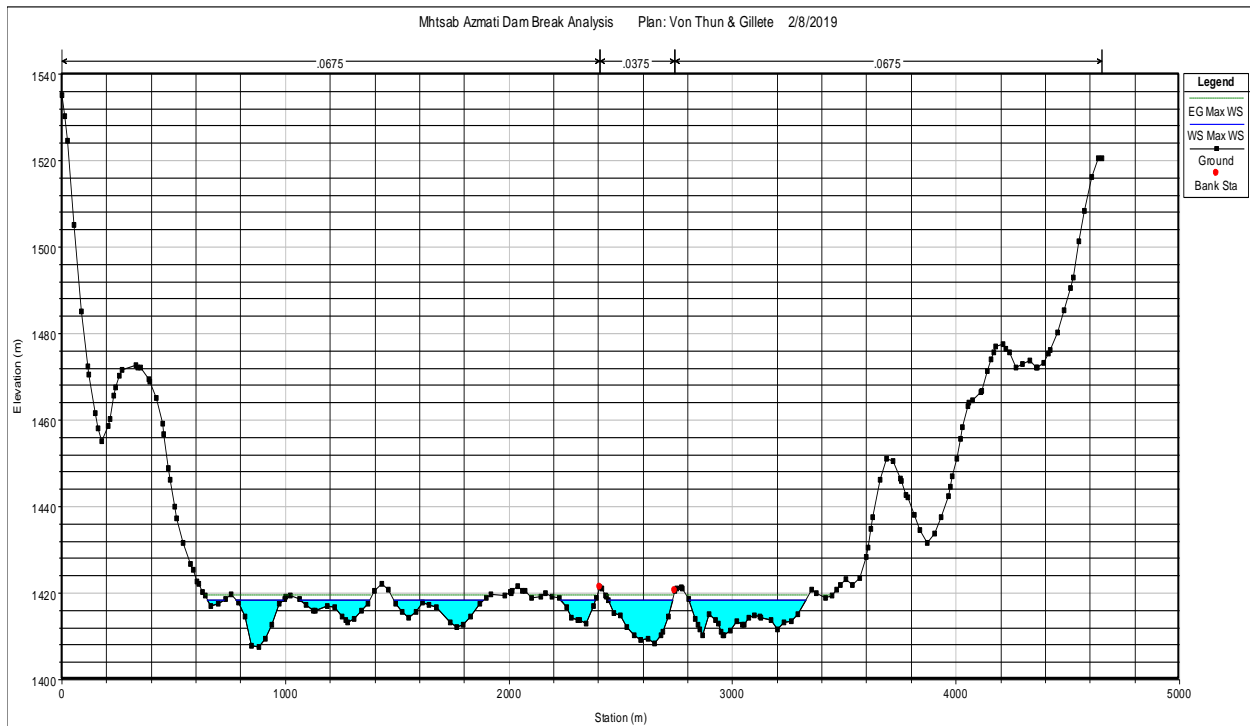


Figure 46: River cross section at 3.46km

Dam Breach Analysis and EAP: Case study Mhtsab Azmati Embankment Dam

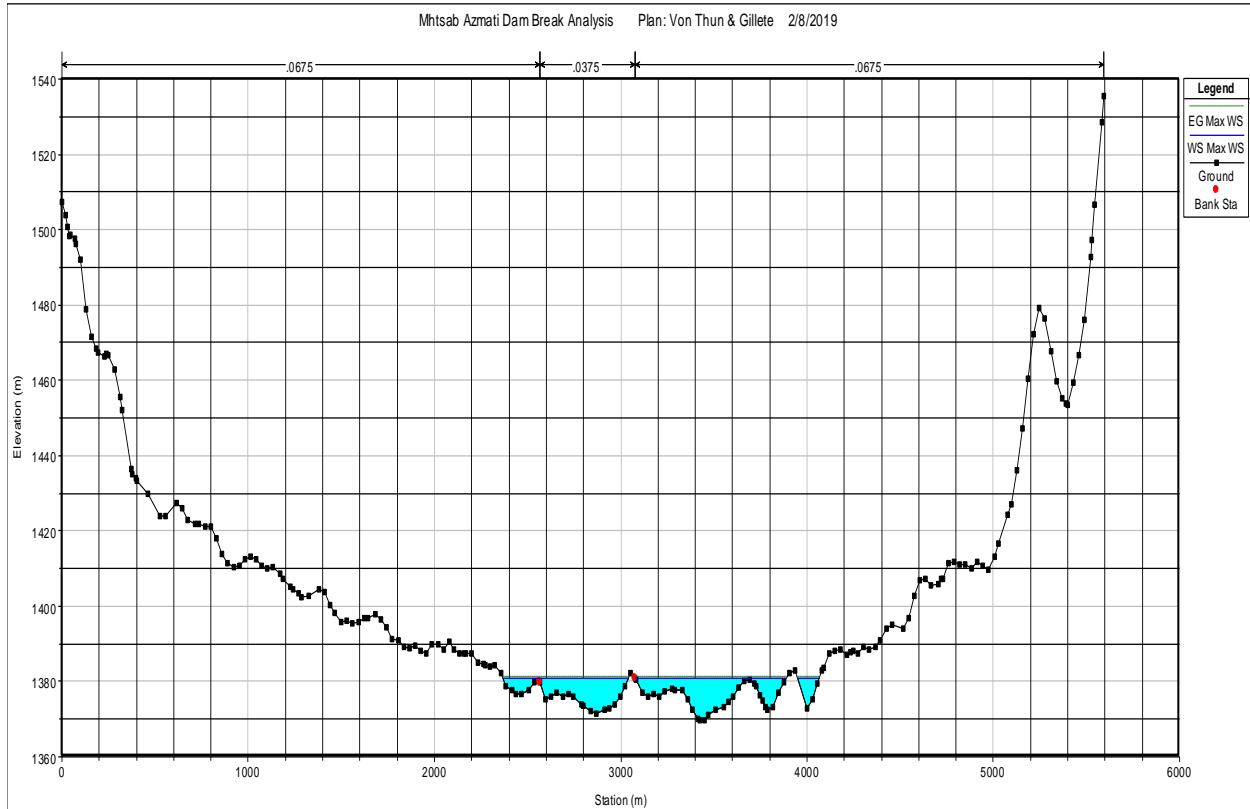


Figure 47: River cross section at 7.92 Km

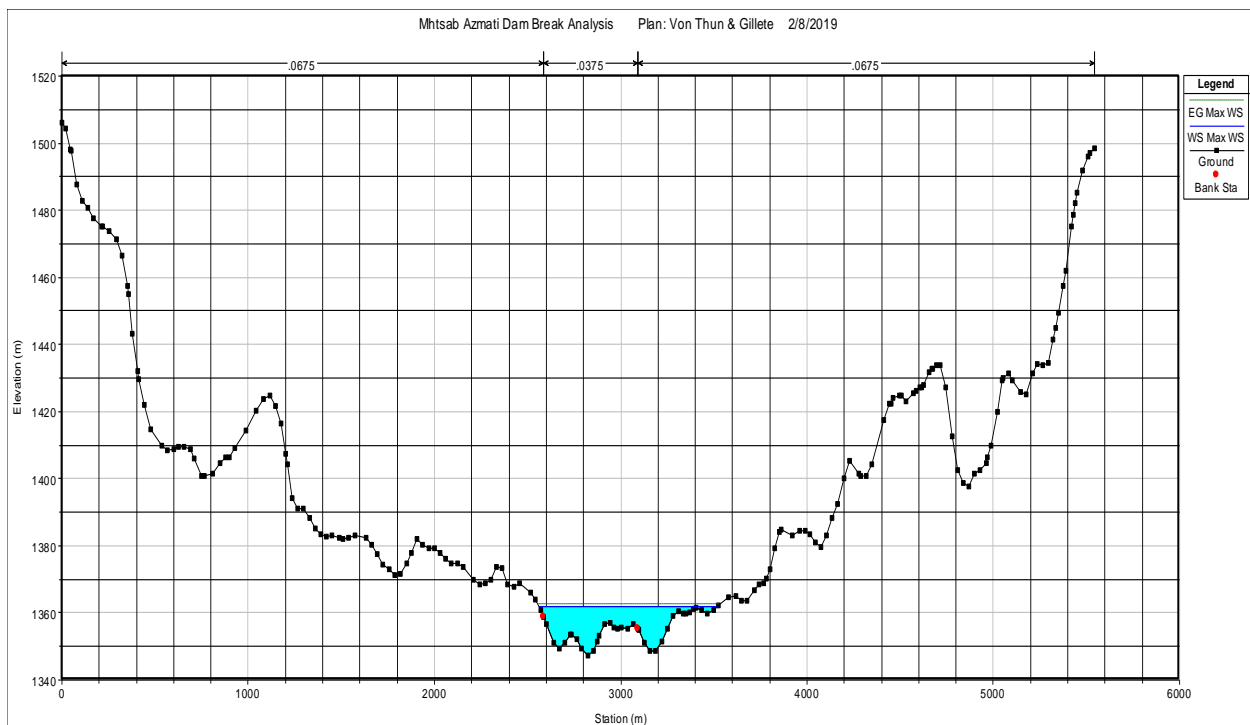


Figure 48: River cross section at 11.44 km

4.3 Inundation Map Development

Inundation map is a map delineating the area that would be flooded by a particular flood event. It includes the ground surfaces downstream of a dam showing the probable encroachment by water released because of failure of a dam or from abnormal flood flows released through a dam's spillway and/or other appurtenant works. Development of an inundation map resulted from the computation of top width, flow depth and arrival of time has a great role in the analyzing of downstream risk assessment and their consequence classification (Duessa and Jubir, 2018).

After the unsteady flow simulation was done in the HEC-RAS software, the maximum flow depth, water surface elevation and velocity were exported in to Arc GIS and integrated with the areal map in order to visualize the inundation map extent developed from the dam breach through overtopping failure case as shown below.

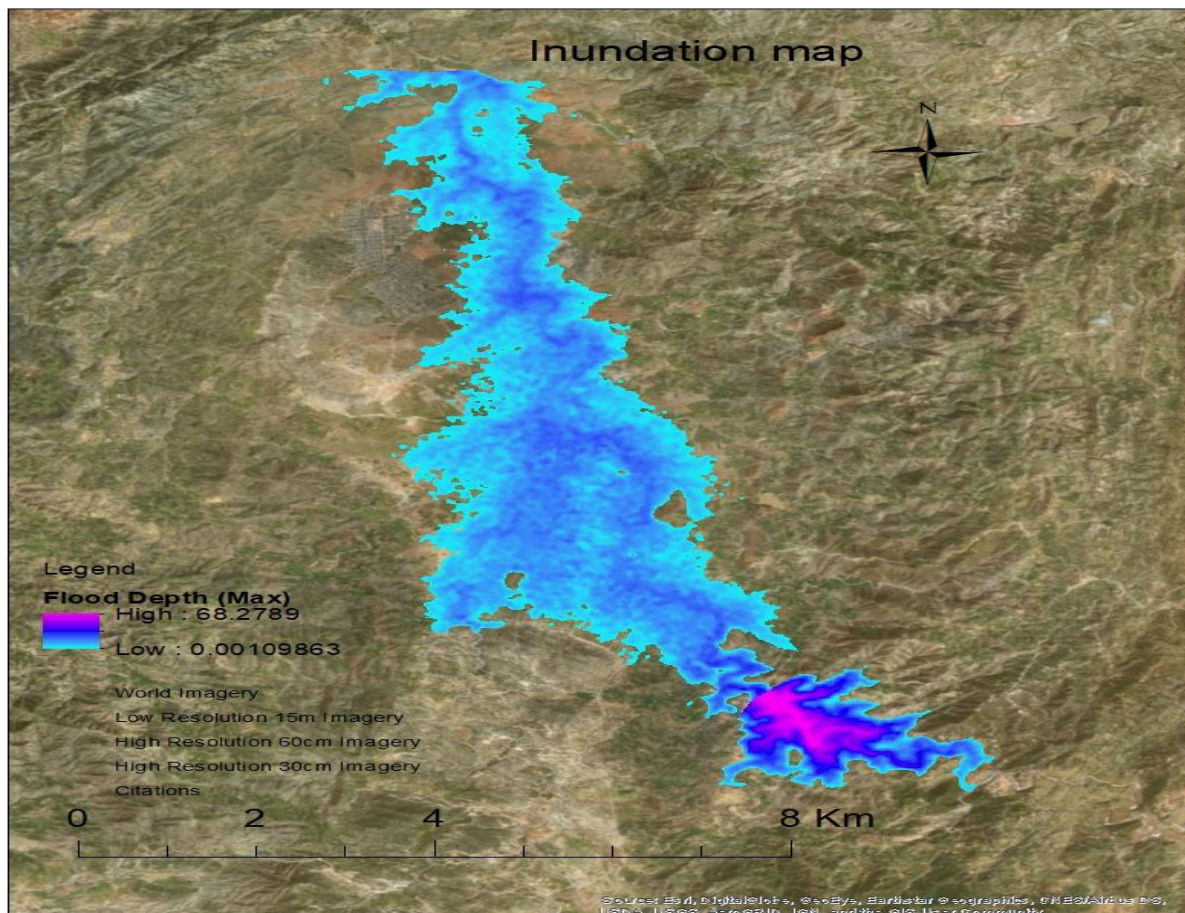


Figure 49: Inundation map

4.4 Risk Assessment of the Downstream Flooding

Risk assessment is a term used to describe the overall process of identifying hazards and risk factors that have the potential to cause harm and analyze and evaluate the risk associated with that hazard (https://www.ccohs.ca/oshanswers/hsprograms/risk_assessment.html). Dam classification can be done according the hazard potential impact resulted from the dam failure as Low, Significant and High-hazard categories based on the potential consequences to the downstream life and properties (Agency., 1998).

Development of an inundation map helps to assess the downstream impact of the flooded area whether population, infrastructures and economic activities were exist in the flood plain. According this study, the population settled in the Rama town and rural area for some extent, external link road (Ethiopia - Eritrea through Rama) and local road (Addis Ababa – Rama) were among the highly affected from the output of Mhtsab Azmati dam breach failure analysis using the probable maximum flood estimated from the upstream catchment. As such type results were found, the dam was categorized as high hazard potential classification due to the highly loss of lives and properties damages. In addition to the loss of human lives and the properties i.e roads, greater than 2000 hr irrigable area and irrigation scheme structures were also highly flooded and causes an accountable economic losses of the society. The following maps shows the inundation map developed from the maximum flow depth and velocity with respect to the population distribution in the area and the roads respectively.

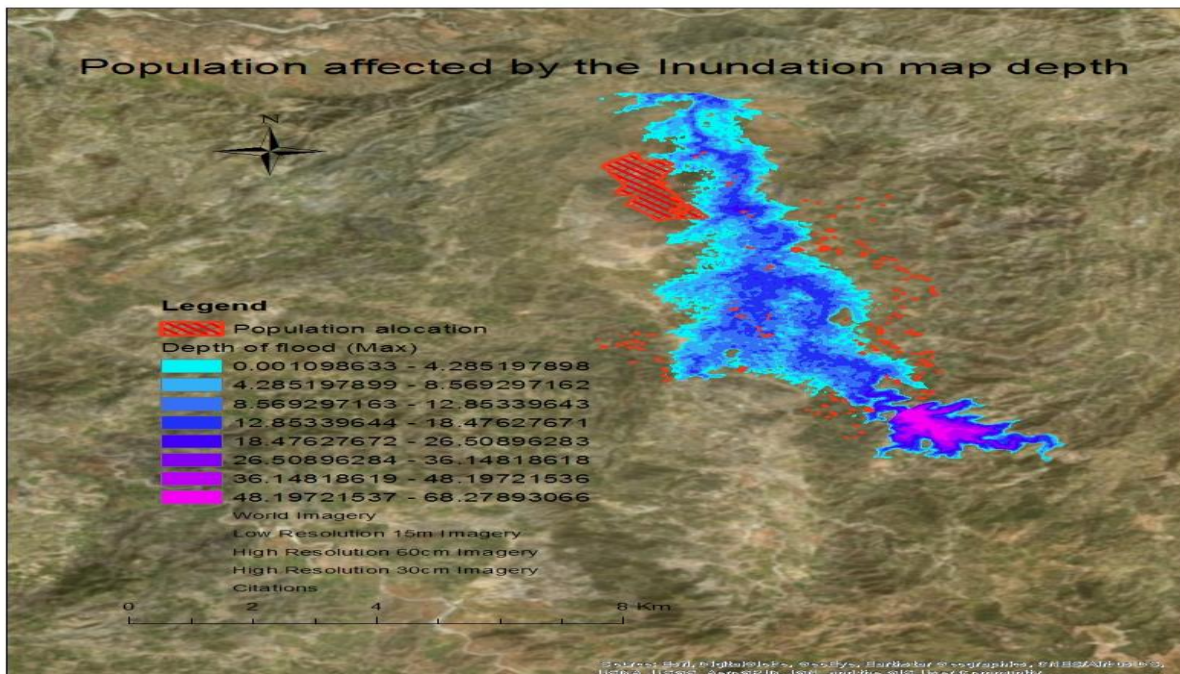


Figure 50: Inundation map due to flood depth

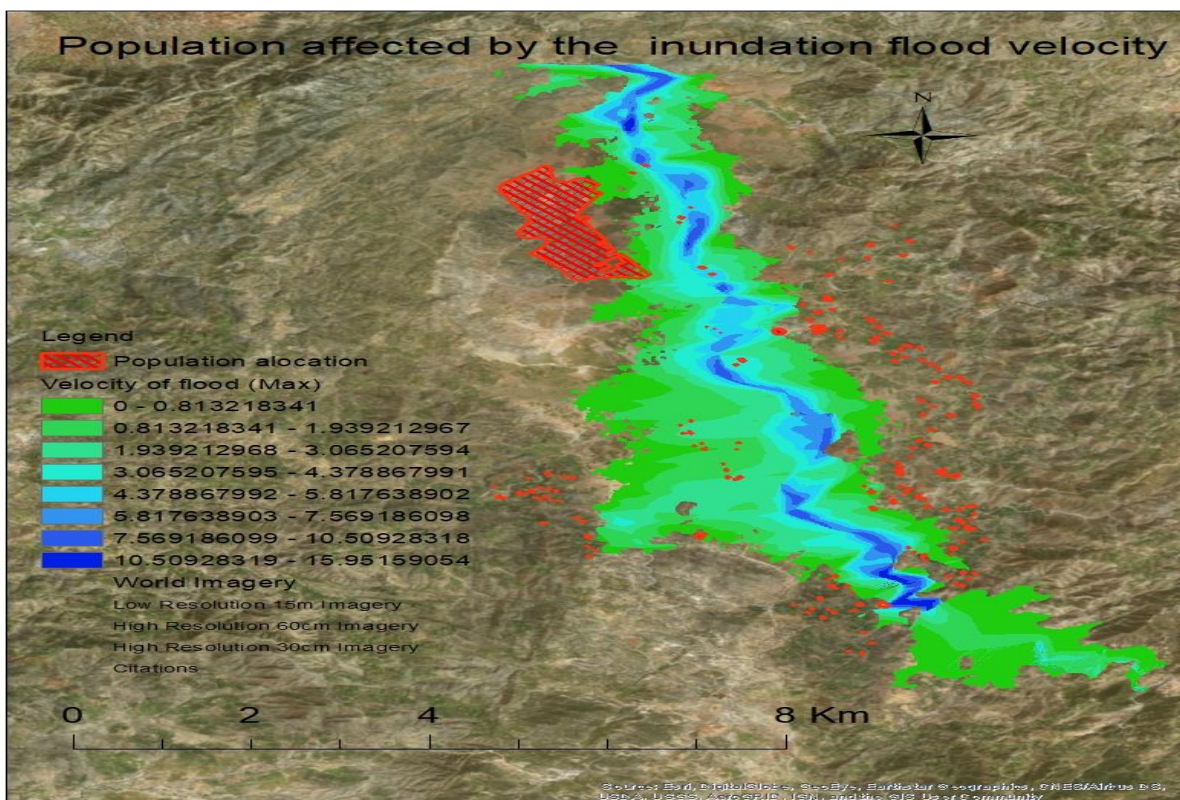


Figure 51: Inundation map due to flood velocity

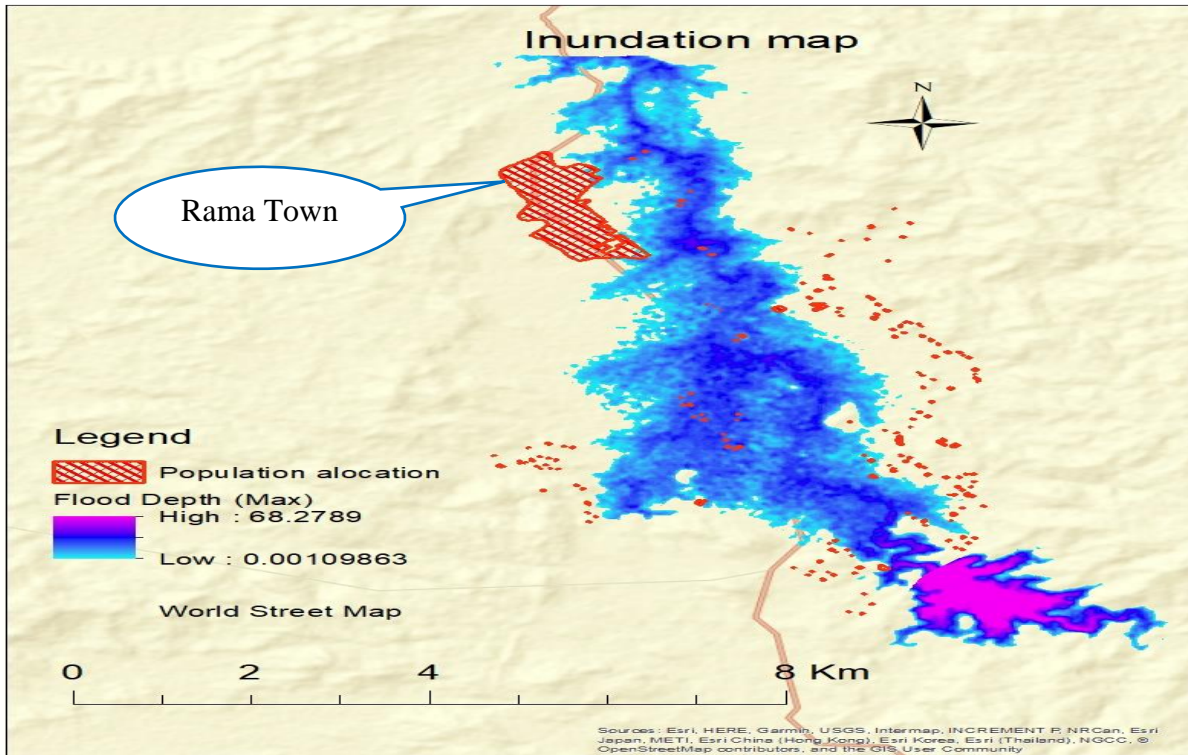


Figure 52: Effect of inundation flood depth for the Rama town

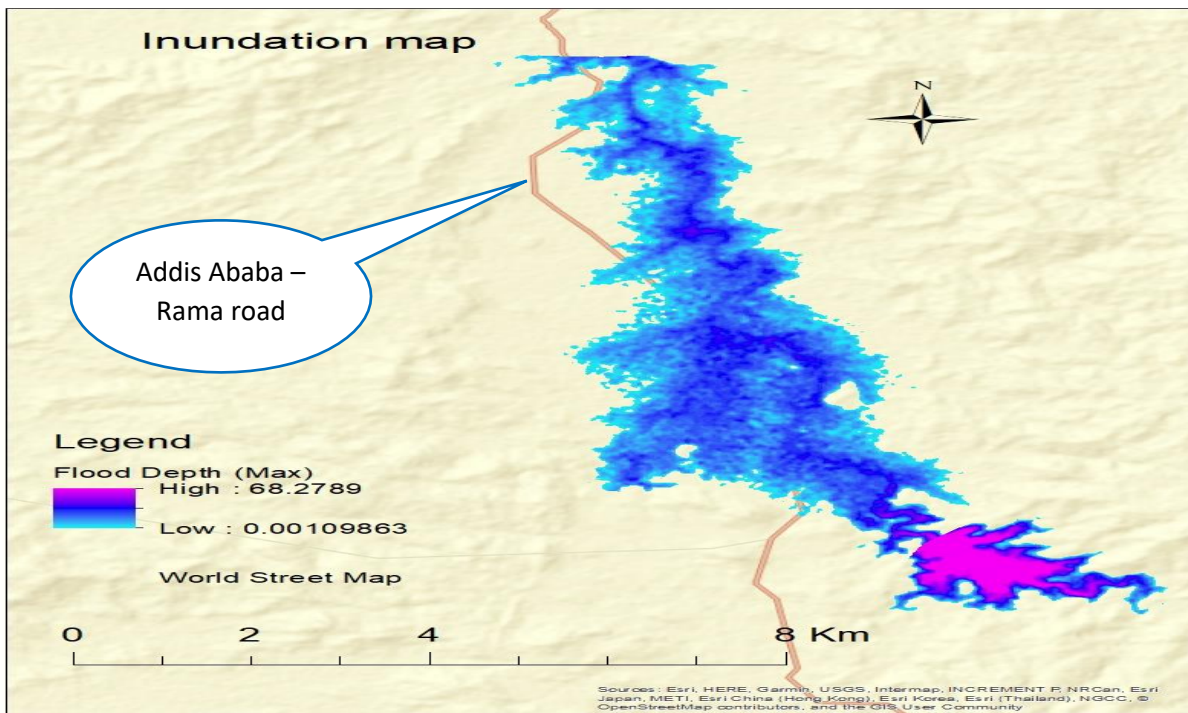


Figure 53: Effect of the flood map on the Addis Ababa – Rama road

4.5 Emergency Action Plan (EAP) Preparation

An Emergency Action Plan is a formal document that identifies incidents that can lead to potential emergency conditions at a dam that identifies the areas that can be affected by the dam the failure and specifies pre-planned actions to be followed to minimize potential loss of life, property damage, potential loss of infrastructure and water resource. Even though most dam owners have a high level of confidence in the structures and dams will not fail, history has shown that on occasion dams do fail and that often these events cause extensive property damage and sometimes death (GAZIOĞLU et al.). Due to this reason, analyzing of dam failure for the extreme floods that can probability be happened and preparation of an emergency action plan was among the main task in this study.

The Mhtsab Azmati dam emergency action plan was prepared by incorporating the dam owners, local emergency response managers and public awareness using the inundation map developed as a reference as it was done by the maximum flood that could be existed.

I. Dam Owner Responsibilities:

Tigray Water Resource Bureau was the owner of the Mhtsab Azmati dam starting from the design to the construction of all infrastructures of the irrigation schemes. But there is no an emergency action plan developed for the dam failure cases even though a number of population are settled in the downstream of the dam. The dam owner's should have the responsibilities to ensure timely and effective action to save the lives and properties from damaging. The dam owner has the responsibilities of identification of emergency at dam, initial notifications, implementation of repairs and security and technical assistance on site. Standard Operating Plan document must be outlined on how to operate dam system to manage the safety guide lines developed by the dam owner and state of dam safety. The following check list must be followed by the dam owner to control the emergency faced as in the inundation map.

II. Local Emergency Responders

Local emergency responders are primary users of emergency action plans developed by the dam owners and state of dam safety. As shown in the inundation map, some of the rural residential and some portion of the Rama Town was among the highly affected area from dam failure. As a result a local emergency responder community planner was highly needed to have a two way communication between the people who affected from the flood and the other responsible bodies to check whether the dam operation system is normal, non-emergency conditions.

III. Public Awareness

Public awareness is a critical component of emergency planning. Development of a public awareness helps to safeguard the individual lives of the downstream community by spreading of qualified information from the responsible body via a well-developed alarm, phone cells and other loud speakers like microphone and uncommon blowing light which provided by the local emergency responders group. These activities must clearly specify by the community and responsible body and should the well-organized to be the indicators of the emergency and should not common to the society not to be familiar and to make the consciously. The following flow chart shows the summary and the two way communication between the responsible bodies.

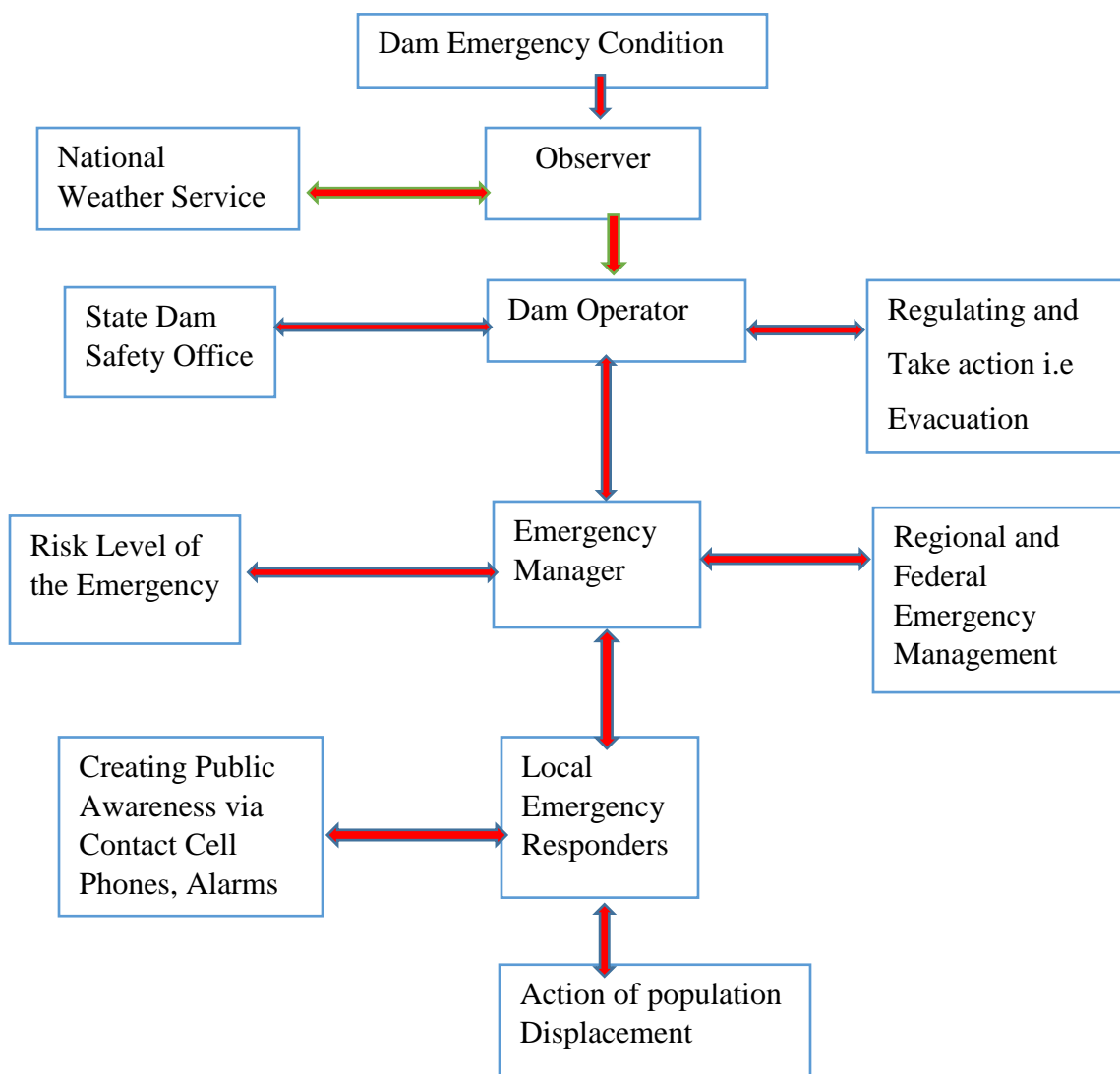


Figure 54: Flow chart of an EAP

Chapter 5

5. Conclusion and Recommendation

5.1 Conclusion

This study was focused on the Dam breach analysis and emergency action plan for the Mhtsab Azmati embankment dam using the probable maximum flood that generated from the its upstream catchment. The probable maximum daily precipitation 208.1928mm, which evaluated from the Adwa, Yiha and Rama meteorological stations and was converted in to areal precipitation of 207.407 mm was used as input data for the generation of probable maximum flood with the help of HEC – HMS model. The catchment characteristics like the curve number value (56 -100), the lag time for each sub basin, catchment basin and river layout were also evaluated using HEC – GeoHMS. The land use and land cover of the catchment were also analyzed as cultivated land (76.6035%), bush land (17.9549%), bare land (5.1298%) and water body (0.3118%).

The dam breach dimensions were computed using the regression equation methods of MacDonald and Langridge -Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) for both modes of failure in the inline structure part of the HEC – RAS model and the unsteady flow simulation was conducted for the analyzing of the peak flood hydrographs of the downstream river. From these regression methods, MacDonald and Langridge - Monopolis (1984) was selected and for the overtopping mode of failures case the breach bottom width, side slopes and the breach development time were found 15m, (0.5H: 1V) 1.18hrs respectively. The maximum overtopping size was also found as 65cm depth. The downstream peak flood hydrographs were also computed using these methods and analyzed using the envelope curves. The peak flood of 727.2 m³/sec which found using MacDonald and Langridge -Monopolis (1984) regression method was selected for the downstream flood impact assessment. The top width and the maximum flood depth of 799.79m, 281.81m, 469.25m and 265.15m and 55.65m, 3.43m, 4.5 m and 4.28m were also found at the respective distance of the inline structure, 3.46 Km, 7.92 Km and 11.44 Km of the downstream river.

An inundation map for the downstream area the Mhtsab Azmati dam was developed as the result of the dam breach. According to this map, the population settled at the south east of the Rama town and around the irrigable area were highly affected by the flood. The Addis Ababa - Ram route road currently works as the external link and local service and was highly affected by this flooding resulted from the dam breach. An appropriate emergency action plan EAP was also developed.

5.2 Recommendation

As it was observed according to the different literatures and histories of dam failure in the world, there are a lots of dam failed by different modes of failure. But among the modes of failure two of them i.e. overtopping and piping were considered for this study and a further dam breach analysis for the Mhtsab Azmati dam using the other mode of failure i.e. the non-weather or non-hydrological like slope stability should be done for their supporting to save the downstream population lives as any failure of the dam has the probability of great impact on them.

Development of an inundation map is among the necessary tasks which have a great importance for the risk assessment of downstream flood prone areas to safeguard lives and property damages but it is highly cost for the collection the actual elevation of the area. The dam owner should have fund budget to collect the downstream survey elevation data which were beyond this study to develop a further investigation due to this analysis was done using 12.5m*12.5m cell resolution size of DEM which has a slightly deviated from the actual one. So further studies should be conducted by the other researchers or especially by the dam owner as it was not taken in to action still now.

The system of hydrological data collection around the study area even in the Tigay regional state is very poor and an amendment should be taken on the collection system as hydrological data is the key factor for the evaluation any hydrological related studies. The Ministry of Water, Irrigation and Electricity of Ethiopia should be develop a modern and updated system of stream data collection to feed a qualified data for economical design and evaluation any hydraulic and hydrological studies.

Around 200ha of irrigable area, the external link and local road, the population settled in the south east of the Rama Town and the rural residential around the irrigable area were strongly affected by the flood. So to reduce or if possible to eliminate the risk, the local population, the dam owner, the contractor, the consultant, National Meteorological Agency, regional or local police should have highly participation and follow in the two of communication according the emergency action plan provided.

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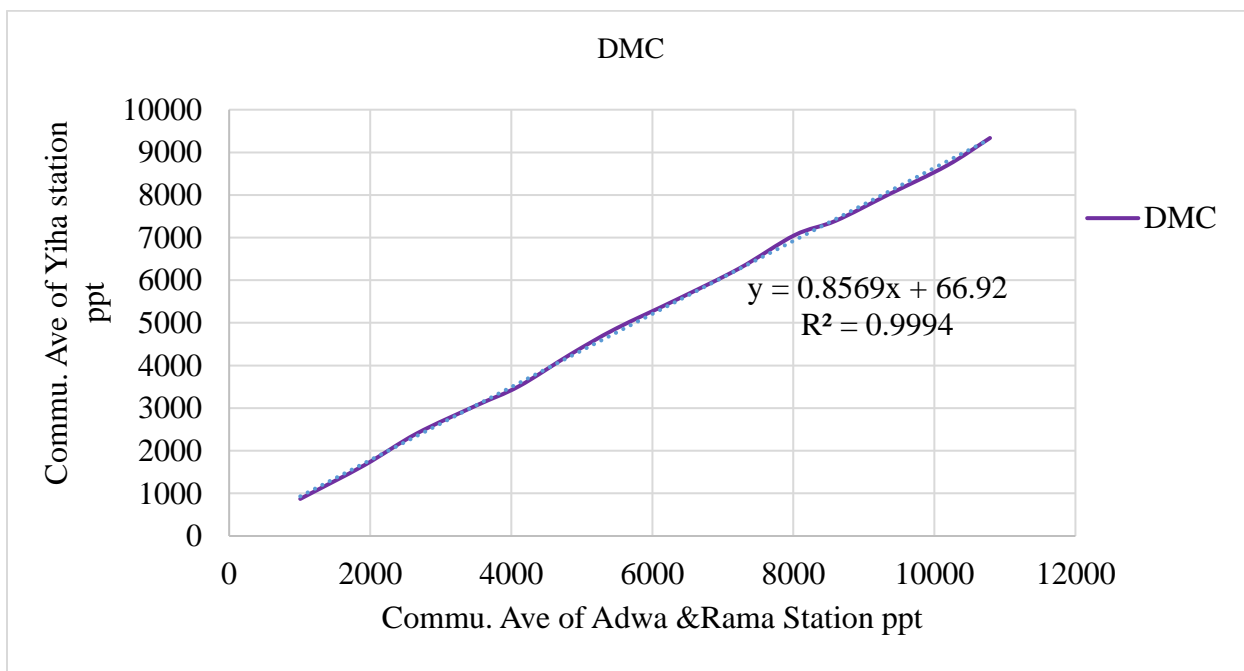
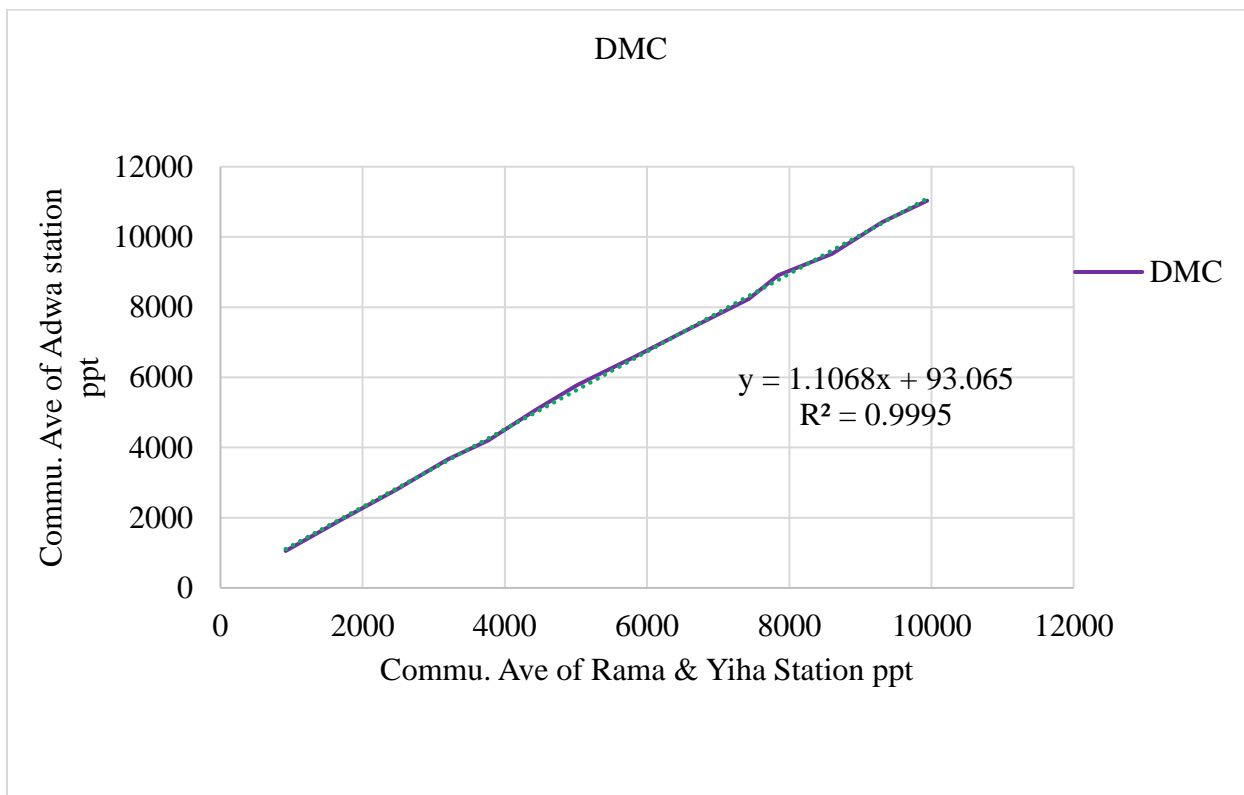
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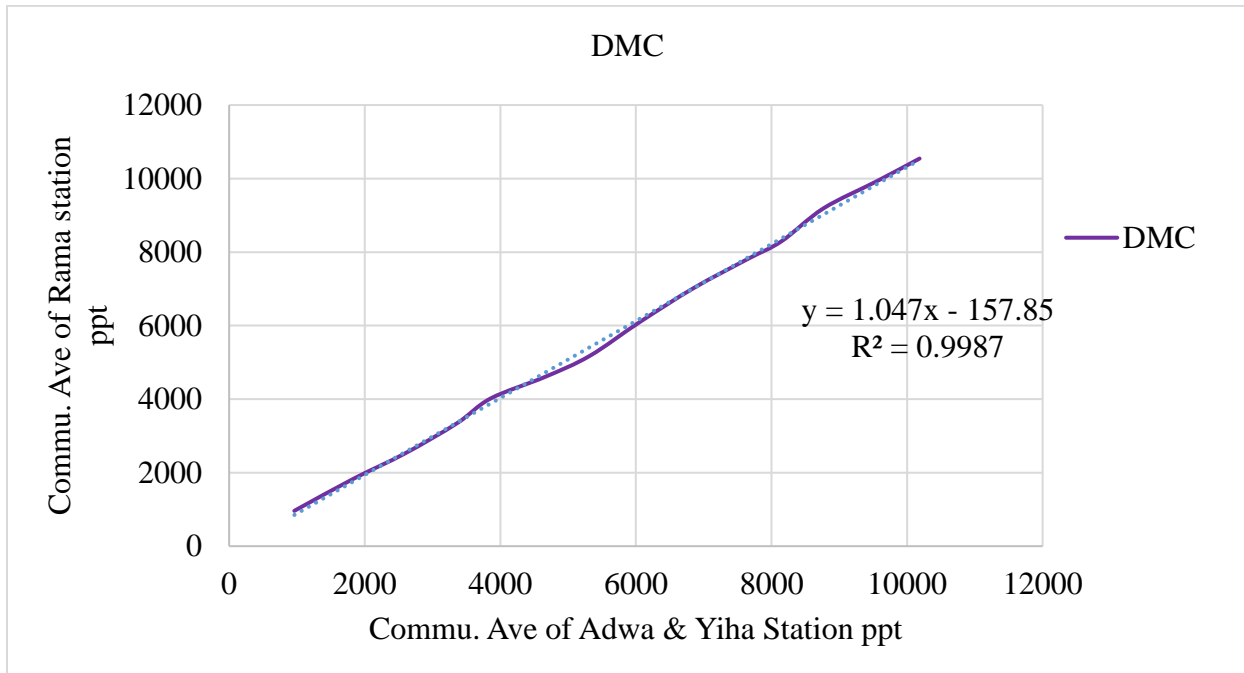
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Appendix

Appendix A: Double Mass Curve for the three stations





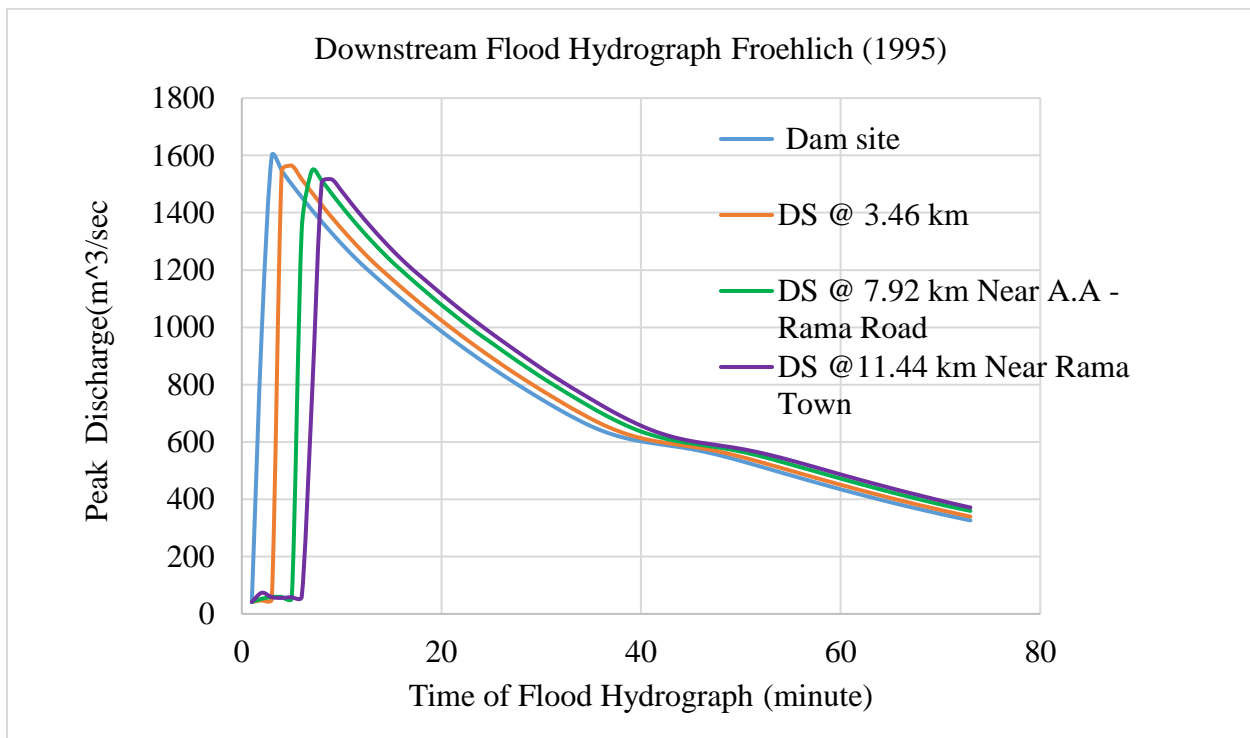
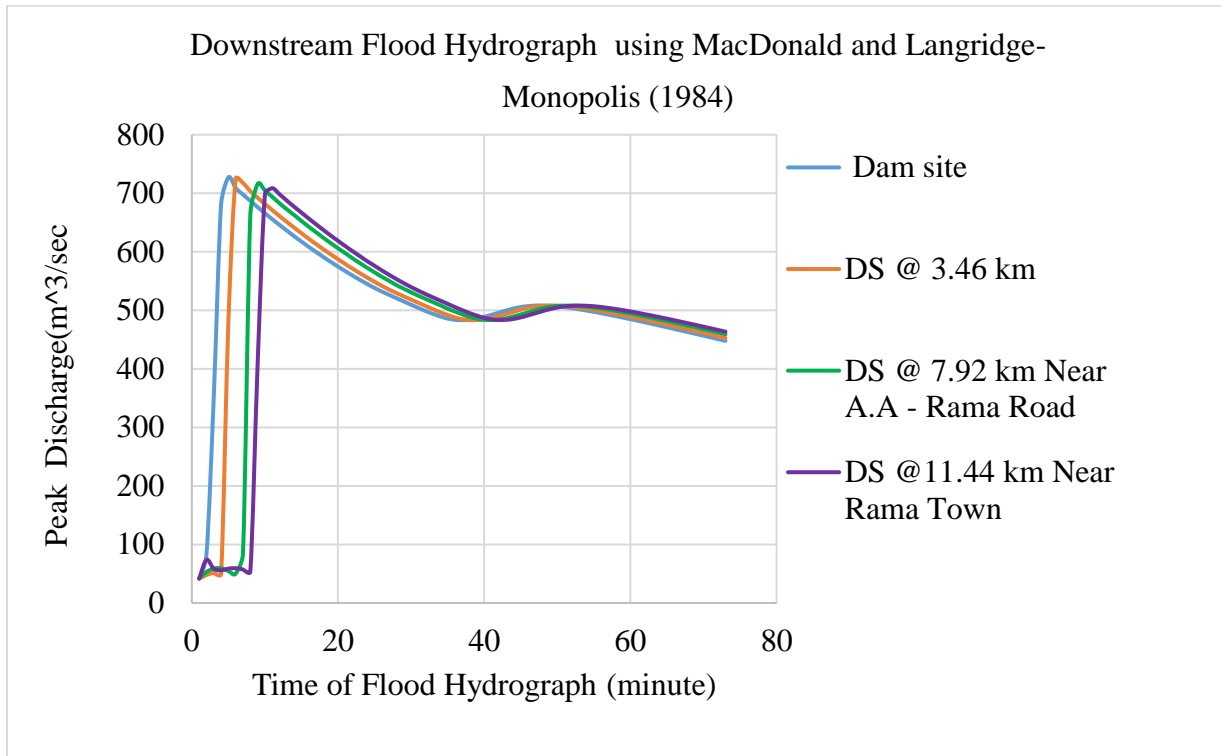
Appendix B: Yearly one day maximum precipitation of the meteorological stations.

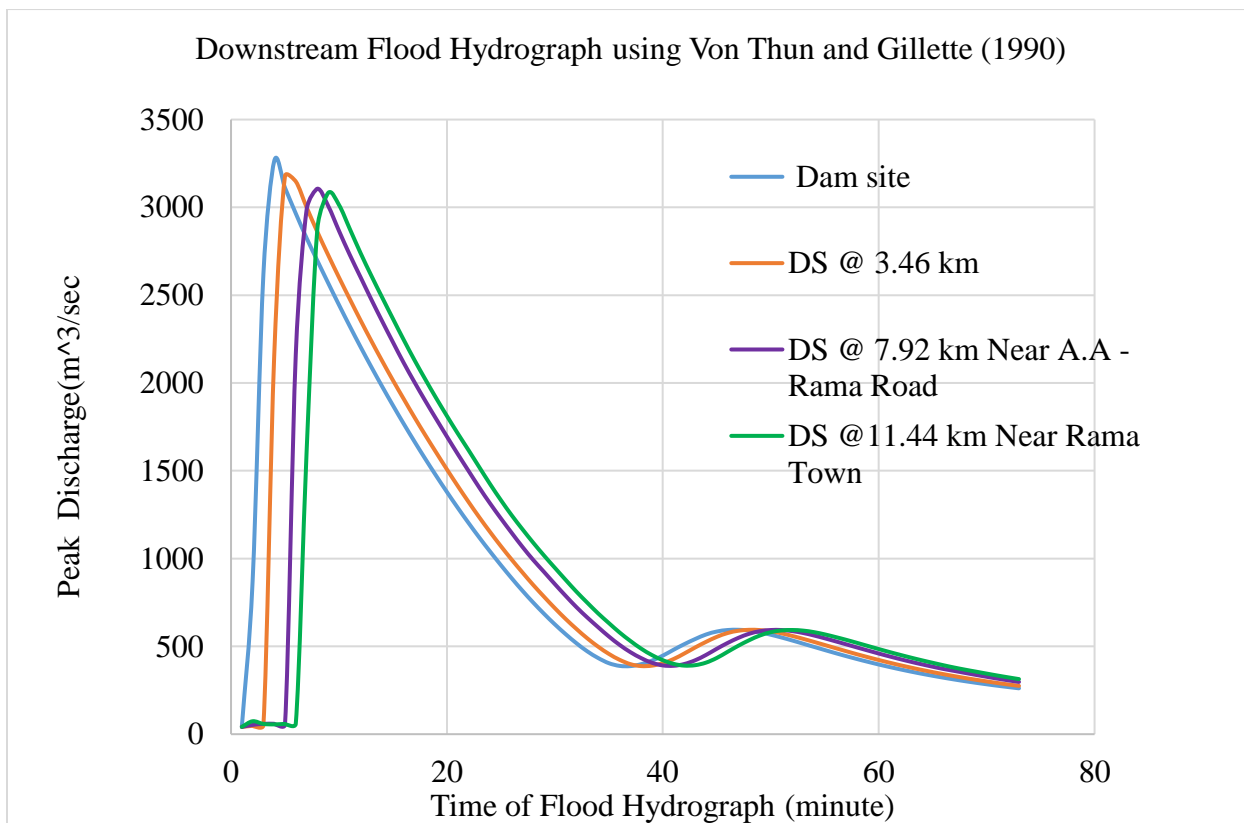
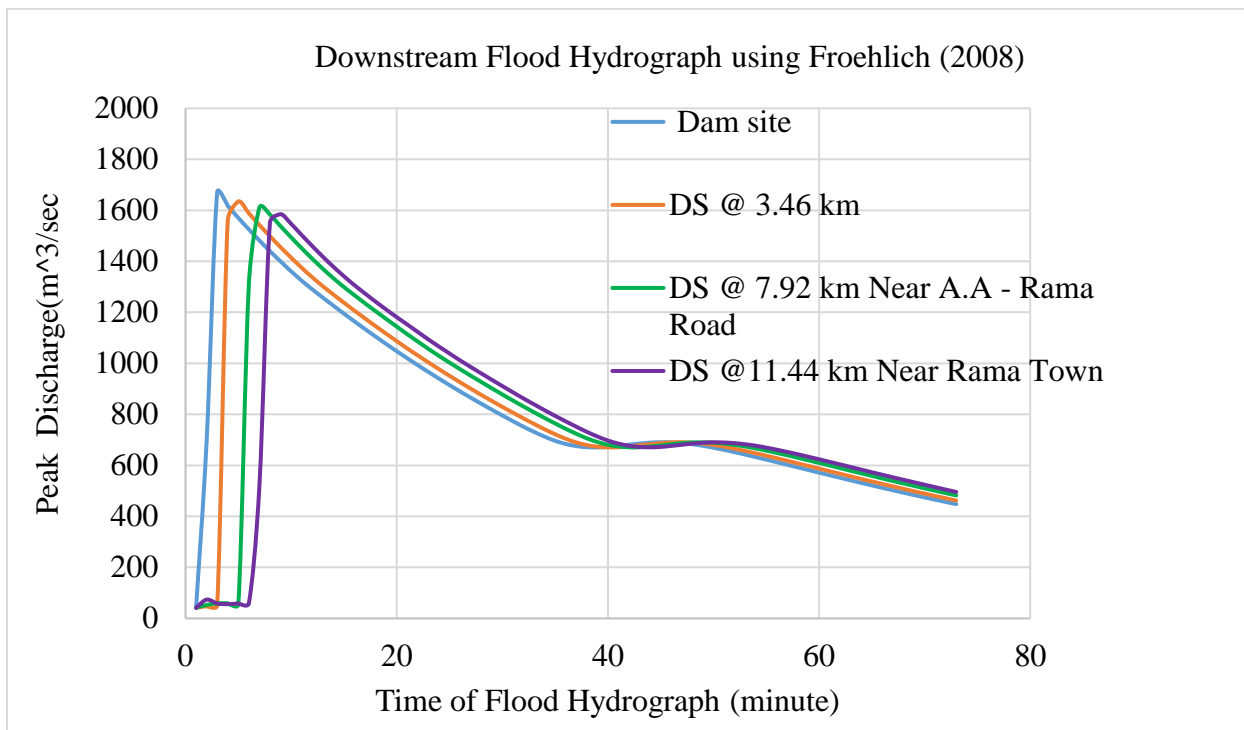
year	Adwa	Yiha	Rama
1998	84.4	105.5	94.9
1999	80.6	41.6	61.1
2000	52.5	33.9	43.2
2001	150.5	102.1	126.3
2002	28.7	24.1	58.1
2003	40.8	26.0	41.8
2004	45.7	26.9	35.0
2005	46.9	27.5	74.1
2006	84.9	59.5	43.0
2007	68.2	56.3	82.9
2008	37.6	18.8	50.2
2009	44.5	51.5	60.2
2010	72.6	56.0	59.8
2011	41.8	26.3	39.0
2012	61.2	45.3	44.6
2013	71.0	61.5	29.3
2014	55.5	92.5	52.0
2015	30.0	50.0	33.2
2016	55.5	78.6	55.0
P_{max}	150.5	105.5	126.3
P_{ave}	60.7	51.8	57.0
δ_n	27.9	26.7	23.8
$P_{ave(n-1)}$	55.7	48.8	53.2
$\delta_{(n-1)}$	17.9	24.0	17.4
km	5.3	2.4	4.2
1Day PMP	208.2	114.8	157.0

Appendix C: PMP disaggregation in to 24hr.

Time(hr)	Hourly cumulative distributed PMP	Incremental PMP	Net PMP
1	42.3	42.3	4.4
2	59.9	17.5	4.6
3	73.3	13.5	4.8
4	84.7	11.3	5.1
5	94.7	10.0	5.4
6	103.7	9.0	5.8
7	112.0	8.3	6.2
8	119.7	7.7	6.9
9	127.0	7.3	7.7
10	133.9	6.9	9.0
11	140.4	6.5	11.3
12	146.7	6.2	17.5
13	152.6	6.0	42.3
14	158.4	5.8	13.5
15	164.0	5.6	10.0
16	169.3	5.4	8.3
17	174.6	5.2	7.3
18	179.6	5.1	6.5
19	184.5	4.9	6.0
20	189.3	4.8	5.6
21	194.0	4.7	5.2
22	198.6	4.6	4.9
23	203.0	4.5	4.7
24	207.4	4.4	4.5

Appendix D: The downstream flood hydrographs taken at different station during the overtopping mode of failure





Appendix E: The downstream flood hydrographs taken at different station during the piping mode of failure

