Addis Ababa University

Institute of Technology

School of Civil and Environmental Engineering

Dam Breach Modeling and Flood Inundation Mapping For Middle Awash Dam

A thesis submitted to the school of Graduate Studies in Partial Fulfillment of the Requirements for the Master of Science Degree in Civil Engineering (Hydraulics Engineering Stream)

By

Kibire Tesemma

Advisor: Dr.Ing. Dereje Hailu

Addis Ababa

Nov, 2016

DAM BREACH MODELING AND FLOOD INUNDATION MAPPING FOR MIDDLE AWASH DAM

A thesis submitted to the school of Graduate Studies in Partial Fulfillment of the Requirements for the Master of Science Degree in Civil Engineering (Hydraulics Engineering Stream)

By

Kibire Tesemma

Advisor:

Dr.Ing. Dereje Hailu

Signature

Approved by

Internal Examiner:

Dr. Yilma Sileshi

Signature

External Examiner:

Dr. Belete Berhanu

Signature

Chairman (Department of Graduate Committee) :

Dr. Agizewu

Signature

Nov, 2016

ACKNOWLEDGEMENT

First and foremost I would like to thank the almighty God for getting me through this.

I am grateful to express my deepest gratitude to my advisor Dr. Ing Dereje Hailu for his guidance and for all his constructive comments. I also would like to appreciate his continuous support and encouragement and to all my instructors in Addis Ababa University School of graduates for the valuable knowledge I have gained.

Great thanks to my families and relatives for their unlimited support .

To all my friends and colleagues who have been helping me in every direction specially WWDSE design and hydrology team greatly appreciated.

Table of Contents

List of Figures

List of Tables

ABBREVATIONS

HEC-RAS	Hydrologic Engineering Center River Analysis System
GIS	Geographic Information System
FEMA	Federal Emergency Management Agency
2D	2Dimensional
1D	1Dimensional
DSO	Dam Safety Organization
ICODS	Interagency Committee on Dam Safety
NWS	National Weather Service
US	United States
HEC	Hydrologic Engineering Center
SMPDBK	Simplified dam break flood forecasting model
DSGL	Dam Safety Guideline
DEM	Digital Elevation Model
Bavg	Average Breach Width
Tf	Time of failure
MAMP	Middle Awash Multipurpose
PMF	Probable Maximum Flood
WWDSE	Water Works Design and Supervision Enterprise
OV	Overtopping
PPG	Piping
WSE	Water Surface Elevation
MA	Middle Awash
BC	Boundary Condition

ABSTRACT

Analysis of dam breach events and the associated flooded area are helpful in the risk assessment of dams. Because sudden failure of dam causes risk of flooding hazard to downstream area. The objective of this study is to model the dam break and map flood inundation areas to be used for estimating the potential consequence of dam breach and emergency planning purpose.

To meet these objectives materials used are HEC-RAS model in conjunction with HECGEORAS an extension of ARCGIS for the case of Middle Awash Multipurpose Dam which is located in Middle Awash sub-basin and DEM 30m of the area was used because of its availability for the extraction of river geometries in HECGEORAS then exported to HECRAS. In HEC RAS dam feature is entered as inline structure, and breach parameters were estimated using the built-in parameter calculator in HEC-RAS and the PMF flow data is entered and unsteady flow simulation is run. The flood map is done with RASMAPPER in HECRAS and Exported to ARCGIS for further work.

Accordingly the Middle Awash dam has been checked for both overtopping and piping mode of failure in HECRAS. The out flow hydrograph result was different for each breach method and breach parameters which in turn affects the inundation area and hazard. It is also different for both mode of failure in HECRAS.

The Middle Awash Dam was checked for overtopping failure with the PMF inflow and for piping starting elevation at the center of dam which occur due to internal erosion of dam material. The peak outflow hydrograph was routed down stream and flood inundation mapping was produced. Failure with overtopping inundate 31000ha area and the failure of the dam with piping inundate 30840 ha. The inundation extent, depth and hazard map of dam breach flood shows that the towns at downstream Melkasedi, MelkaWerer and the cultivated area, infrastructures, the animal and human life in the downstream can be affected with inundation caused due to flood from dam failure.

The flood inundation map helps in estimation of the severity and extent of dam break flood, warning time, consequence classification for emergency action planning and to alert the government body. Therefore, damages that could occur in the surrounding settlements, agricultural areas, on both lives and infrastructure can be minimized and even controlled

1. INTRODUCTION

1.1. Background

From the early period of civilization, construction of dams has been a long established practice. Dams are one of the infrastructures that play a vital role in meeting water demand of the socitiety by providing many benefits like flood control, water supply, irrigation, hydropower, navigation, and recreation. These benefits provided by dams come at a risk due to their potential to fail and cause catastrophic flooding. For this reason, mitigation of this risk is essential by identifying potential failure modes, simulate the potential failure and protect against them for the benefit of the society as floods resulting from the failure of dams produce devastating disasters(Wahl, 2010). This dam failure caused for a number of reasons and its consequence leads to requirement for preparation of dam breach inundation modeling and mapping to identify the flood risk and mitigate the consequences. As the report of FEMA(2013) Dam breach inundation mapping received attention following two significant failure incidents in California (Baldwin Hills Dam in 1964 and Lover Van Norman Dam in 1971).This prompted the state of California to prepare dam failure inundation maps.

So while planning and implementing dams, taking a good care of their safety is currently becoming an important issue. Potential consequences of a dam failure should be understood for planned dams and for those which are already built since the failure phenomena is unexpected an immediate mitigation measures cannot be taken to hinder the breaching process dam For dams under planning stage, one can use dam breach inundation information for classifying the dam (and its hazard class) and this classification can then be used to estimate spillway discharge capacity, seismic parameters, and others. Moreover, for both planned and existing dams, this information is essential for preparedness and emergency action planning related to dam failure (FEMA, July 2013). To put this into practice Different organizations and researchers have contributed their findings in the analysis of dam break and its consequences. They have derived regression equations based on data from historical dam failure events that are used in predicting the breach geometry. These include; the United States Bureau of Reclamation (USBR) and MacDonald and Langridge equations and Development of analytical models (Colorado, 2010).

In our country Ethiopia Construction of dams has been commenced since the first dam was built in 1939 and the dam was constructed on Akaki River to generate hydro-electric power. The dams built so far are being used in order to alleviate the water related problems of the population.

Nowadays there are several number of dams constructed and others are under construction for irrigation, water supply and hydroelectric power mainly. The dams Tendaho and Beles are constructed mainly for irrigation, koka and Gilgel gibe dams are constructed for hydroelectric power demand including the Great Ethiopian Renaissance Dam to boost the nation‟s economy (WIKIPEDIA, n.d.).

Middle Awash Multipurpose dam is also one of the dams planned for flood control and irrigation development which is located in Middle Awash sub basin Afar region Ethiopia. The dam is a rock fill embankment type .

In this study, a dam breach analysis and mapping the downstream area inundated by the resulting catastrophic flood shall be investigated for Middle Awash Multipurpose Dam.

1.2. Problem Statement

Understanding Potential consequences of a dam failure for planned dams and for those which are already built are essential because due to various reasons dam failure may occur that can cause devastating disaster to the downstream ecosystem. During construction failure may occur due to unpredictable inflow of extreme hydrologic event to the reservoir, during operation due to malfunctioning of spillway gates and operation errors overtopping may occur, seismic activity, excessive seepage and other hydraulic activities.

Nowadays in Ethiopia many large dams are under construction and some of them have been built in recent years for the purpose of development of irrigation, hydropower or water supply or the combination of these. Following the establishment of dams, the downstream ecosystem is highly changed that huge area is covered with irrigation farms, agro processing plants, new settlements and residence areas of inhabitants living on the farms and factories are formed. All these investments and newly settled inhabitants are highly exposed to flooding and are at risk for damage and death respectively and Middle Awash Dam faiure can cause loss of human life and property of the downstream. The flood from the damage of the dam can also damage towns, villages and its inrastructues on the downstream areas.

In addition causes ruining out of the embankment and its appurtenant structures which constructed with high investment, loss of impounded water that has been accumulated for year and that could irrigate an enormous area of land and flooding of the irrigation farms which is the main purpose of constructing the dam are consequences of Middle Awash Dam Break.

Apart from constructing dams in it is essential for every dam to have dam break analysis and its faiure consequence study. Hence, the intention of this study is to fill this gap.

So this study focuses on the dam break analysis aimed to investigate the p.ossible breaching of the proposed Middle Awash Multipurpose dam and to delineate the area that would be flooded out due to the hazardous wave front.

1.3. Research Questions

The following questions can be raised to initiate this study

- What are the different failure modes that would cause the dam to break and which of these is the most catastrophic?
- What are the breach parameters defining the cross section of the breach and how sensitive are they?
- How much area will be inundated?
- What are the parameter uncertainties that affect the analysis result?

1.4. Objective of the Study

1.4.1. General Objective

The general objective of this study is to model the dam breach , map the resulting inundation, identify and analyze the breach parameters that affect the result of the analysis for the iddle Awash Multipurpose Dam.

1.4.2. Specific Objectives

The specific objectives are:

- Mapping flood inundation areas to be used for estimating the potential consequence of dam breach , confirming the classification and emergency planning purpose
- Simulate the breach process or development applying different breach parameters and breach methods
- To determine dam breach parameters to make realistic estimate of the outflow hydrograph and downstream inundation extent
- To identify the possible failure mode

2. LITERATURE REVIEW

2.1. Dam Failure Overview

Dam is one of the infrastructures that provide several benefits for society but floods resulting from failure produce the most devastating disaster on property and life. The loss of life varies with the extent of the inundation area, the size of population at risk and the amount of warning time available (Tony L, 1998).

Modern dam-safety analysis has been an evolving science since the 1970s. After four notable dam failures occurred in the United between 1972 and 1977, President Carter issued a memorandum directing the review of federal dam-safety activities by a committee of recognized experts. According to this the analysis of dam breaching and the resulting floods became an essential. This helps in reducing loss of life and damage in the downstream flood plain (FEMA, 2013).

The research report of US Dam safety (Tony L, 1998) concludes that the Simulation of embankment dam breach events and their resulting floods are crucial in characterizing and identifying threats due to potential dam failures. This Characterization of the threat to public safety that a dam poses establishes the Hazard Classification of the dam and the associated standard of care to which the dam is held. The Hazard Classification of a dam determines the inflow design flood (IDF), which is the basis for spillway sizing. The Hazard Classification also triggers the requirement to prepare an Emergency Action Plan, requiring preparation of inundation maps which accurately predict dam breach flood depths and arrival times at critical locations.

As the report of (FEMA, July 2013) breaching in embankment dams may occur for a variety of reasons but breaches in embankment dams often modeled as overtopping or piping failures. Flow over an embankment dam (earth or rockfill) usually leads to erosion of material on the downstream slope and failure of the dam. Depending on the composition of the dam overtopping failures can occur very differently.According to a study by Ralston (1987), a small headcut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as the breach 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. 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 downstream main channel. 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 reached bedrock (Gee, 2010) .

FEMA(2013)summarizes that piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse.

There are several possible options to identify the breach initiation time. 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. For breaches associated with a hydrologic event, the initiation can be considered to begin when the reservoir water level reaches a certain elevation or after the water level has exceeded a certain elevation for a specified duration. For fair weather breach analysis, an initiation time should be specified regardless of pool elevation (Gee, 2010). 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, is uncontrolled, and will result in the failure of the dam (FEMA, 2013).

The failure consequence classification is to determine the design requirements for a particular dam. Dams with higher failure consequence are required to be designed to higher standards. Regulatory requirements such as maintenance, operation and surveillance are also based on failure consequence classification (CDA, 2013). In addition (Tony L, 1998) reported that flood inundation mapping is an important tool for municipal and urban growth planning, emergency action planning, flood insurance rates and ecological studies. By understanding the extents of flooding and flood water inundation decision makers are able to make choices about how to best allocate resources to prepare for emergency and to generally improve the quality of life. The resulting flood inundation maps are useful for municipal planning purposes, emergency action plans, flood insurance rate and ecological study (Goodell, 2006).

2.2. Dam Failures

2.2.1. Causes of Dam Failures

Depending on the type of dam and site-specific conditions, a dam may be susceptible to failure from multiple causes. The breach shape and timing of a dam failure varies depending on the type of dam. Concrete gravity dams tend to have a partial breach, as one or more monolith sections formed during dam construction fail, whereas concrete arch dams tend to fail suddenly and. In contrast, embankment dams do not usually have a complete or sudden failure, but rather tend to breach to the point where the reservoir is depleted or to where the breached materials resist erosion, such as at the dam foundation (FEMA, 2013).

Breach analysis for rigid structures is generally straightforward. It typically involves the instantaneous removal of a portion of the structure, or, in some cases, the entire structure. (Colorado, 2010)

Embankment dam failure or breaching may occur for a variety of reasons, breaches in embankment dams are most often modeled as overtopping or piping failures (FEMA, 2013).

2.2.2. Failure Modes

The many causes of dam failures are commonly summarized using five types of failures modes: hydrologic, geologic, structural, seismic, and human-influenced.

Table 2:1. Possible Failure Modes for Various Dam Types

Failure Mode	Earthen/ Embankment	Concrete Gravity	Concrete Arch	Concrete Buttress	Concrete Multi-Arch
Overtopping	X	X	X	X	X
Piping/Seepage	X	X	X	X	X
Foundation Defects	X	X	X	X	X
Sliding		X	X	X	
Overturning		X		X	
Cracking	X	X	X	X	X
Equipment failure	X	X	X	X	X

Source: (Brunner, 2006)

2.2.3. Hydrologic Failure Modes

Hydrologic dam failures are induced by extreme rainfall or snowmelt events that can lead to natural floods of variable magnitude. The main causes of hydrologic dam failure include overtopping, structural overstressing, and surface erosion due to high velocity flow and wave action (FEMA, 2013)

2.2.3.1. Overtopping

Overtopping occurs when the water surface elevation in the reservoir exceeds the height of the dam; water can then flow over the top crest of the dam, an abutment, or a low point in the reservoir rim. Overtopping usually results from a design inadequacy of the dam/spillway system and reservoir storage capacity to handle the resulting flooding event. A failure may also occur when a reservoir‟s outlet system is not functioning properly, thereby raising the water surface elevation of the dam. Overtopping of a dam as a result of flooding is the most common failure mode for embankment dams. During a severe overtopping event, the foundation and abutments of concrete dams may also be eroded, leading to a loss of support and failure from sliding or overturning (FEMA, 2004).

Dam failure begins when appreciable amounts of water begin flowing over or around the dam face and begin to erode the face of the dam. For embankment dams, the failure typically begins at a point on the top of the dam and expands in a generally trapezoidal shape. The water flow through the expanding breach acts as a weir; however, depending on conditions such as headwater and tail water, various flow characteristics can be observed during a breach development including weir flow, converging flow, and channel flow (FEMA, 2013).

|As (Colorado, 2010) Overtopping failures of earthen dams typically begin with head cutting at the downstream toe and advance upstream until the erosion reaches the dam crest and reservoir surface. A dam failure resulting from an embankment slide can also lead to an overtopping type of failure when the slide encroaches upon the high water line. Once the reservoir is connected to the progressing breach, down cutting of the embankment and lateral erosion occur until the breach expands to its final dimensions. The above process assumes a level dam crest. Uneven dam crest surfaces can result in concentration of flow and erosion of the crest itself, accelerating the process of connecting the reservoir to a progressing breach.

2.2.3.2. Structural Overstressing of Dam Components

Higher loading conditions are typically found in dams where the reservoir elevation is increased due to a hydrologic event. While the dam itself may not be overtopped, the surcharge may be increased, overstressing the dam‟s structural components. This overstressing may then result in an overturning failure, sliding failure, or failure of specific components of the dam (FEMA, 2004).

2.2.3.3. Surface Erosion From High Velocity And Wave Action

Surface erosion can occur along earthen spillways, the upstream or downstream embankment slopes, or along other appurtenant structure inlet and outlet channels. Surface erosion is primarily caused by high velocity runoff, reservoir wave action, and ice action. High flow velocities may cause head cutting along spillway sides that can progress towards the spillway crest, eventually leading to a full dam beach (FEMA, 2004).

2.2.3.4. Geologic Failure Modes

Geologic failure modes include piping and internal erosion as well as slope instability and hydraulic fracturing. For embankment dams, geologic failures are typically caused by long-term seepage of water stored in the reservoir; the water seeps through the dam or the foundation and abutments, weakening the embankment over time. If seepage is uncontrolled it may lead to internal erosion or piping of the embankment materials within the dam. A geologic failure may also result from inadequate geotechnical design of the embankment and foundation, inadequate seepage controls, or increased load situations such as the rapid increase or drawdown of water level due to a hydrologic event, landslide, earthquake, or wave action (FEMA, 2013).

2.2.3.5. Piping and Internal Erosion

Piping: Piping occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam embankment or foundation leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Once the erosion reaches the reservoir, it can enlarge and cause catastrophic dam failure (FEMA, 2013).

Internal Erosion: similar to piping, internal erosion is the occurrence of erosion where two adjacent zones interface within the embankment or at the contact between the embankment and foundation. Internal erosion is differentiated from piping in that internal erosion originates internally, whereas piping originates externally when voids of the material into which seepage is flowing are larger than a critical size required to retain the particles, the particles of the up-gradient material can be transported into or through the adjacent material, thereby resulting in internal erosion (CDA, 2013).

2.2.3.6. Structural Failure Modes

Structural failures can occur when there is a failure of a critical dam component. Structural failures may be related to an inadequate initial design, poor construction, poor construction materials, inadequate maintenance and repair, or gradual degradation and weakening over time Additionally, structural failure may be inter-related with other modes of failure for example, structural failure of the main embankment may be related to internal piping, or a critical dam component could fail due to overstressing during a flood event(FEMA, 2013).

2.2.3.7. Seismic Failure Modes

Earthquakes are another important cause of dam failures, especially in seismic zones of the United States. Seismic failures are generally related to either ground movement or liquefaction. Ground movements may cause a dam to shift, settle, or crack into an undesirable configuration that prevents the dam from performing as designed (FEMA, 2013).

Failure Mechanisms Due To Seismic Activities Include:

- Slope instability
- Permanent deformations
- Fissures or cracking
- Differential settling
- Rupture of principal spillway outlet pipeline
- Liquefaction

2.2.3.8. Human-Influenced Failure Modes

Human-influenced dam failure incidents can be related to improper design or maintenance, mis operation including scheduled volume releases, or terrorist acts.

Maintenance:

Misoperation: Misoperation is the sudden or accidental and/or non-scheduled operation of a water retaining element of a dam that releases stored water to the downstream channel in an uncontrolled manner. Mis-operation also includes the deliberate release of floodwater because of an emergency situation, but without the issuance of a timely evacuation warning to the downstream interests. It also includes the inability to operate a gate in an emergency, a condition that could lead to overtopping of the dam and potential breach (FEMA, 2013).

Scheduled volume releases: The release of reservoir volume is a common practice for maintenance purposes, and to provide additional flood storage volume in a reservoir in anticipation of an extreme flooding event. The rapid release of reservoir volume in an upstream dam may result in dam overtopping at a downstream dam, resulting in dam failure. A rapid release of storage volume in a reservoir may also result in a rapid drawdown and a geologic failure. Improper releases of storage volume may result in a dam failure (FEMA, 2013).

Terrorist incidents: Terrorist activities can range from purposeful mis-operation of the dam to physical attacks on the structure itself. Two common scenarios are typically considered when analyzing human-influenced dam failure: rapid failure of spillway gates, and a lowering of the dam crest. For an embankment dam, the rapid lowering of the dam crest could subjugate the dam to overtopping and subsequent erosion (FEMA, 2013).

Table 2:2 : Typical Dam Failure Modes

Hydrologic	Overtopping due to: • Inadequate spillway design • Blocked spillway • Loss of freeboard* due to embankment settlement or erosion • Structural overstressing of dam components Surface erosion due to: • High velocity water • Wave action
Geologic	Piping and internal erosion caused by: • Internal cracking, hydraulic fracture, or differential settlement • Inadequate filters • Outlet pipeline failure • Pipes through the embankment formed by roots or animal/insect burrows Slope instability and hydraulic fracturing: • Load exceeds sliding resistance at base or at joints of structure
Structural	Concrete dam: Failure of critical structural components Embankment dam: Failure of the upstream or downstream face
Seismic	Earthquakes/ground movement; also liquefiable foundations or embankment materials
Human influenced or caused	Misoperation: • Sudden rise in reservoir level causes flow through transverse cracks in embankment • Incidents including gate failures, power interruption
Terrorist activities: • Purposeful MisOperation of the dam • Impact of object that removes part of the dam crest	

(FEMA, 2013)

2.2.3.9. Consequences Of Dam Failure

Despite efforts to improve the safety of the dams, we must still address the concerns of what may happen should a dam fail. The potential loss of life and property damage likely to occur during a catastrophic dam failure may be mitigated through an understanding of the resulting flood wave characteristics and inundated area. The flood results may then be applied to develop emergency response plans and future land use planning (Cameroon T. Ackreman, n.d.)

2.2.3.10. Hazard Potential Classification

In most situations, the investigation of the impacts of failure on downstream life and property is sufficient to determine the appropriate hazard potential rating; however, there may be circumstances where further evaluation is appropriate. For example, the reservoir of a dam that would normally be considered to have a low-hazard potential based on insignificant flooding due to failure may be known to contain toxic sediments, such as may exist in a tailings pond. Therefore, a low-hazard potential rating may not be appropriate and instead a higher standard may be more appropriate to classify the hazard potential (FEMA, 2013). FEMA guidance recommends that the hazard potential rating be based on consideration of the effects of a failure or mis operation during both normal and flood flow conditions. FEMA further recommends that the hazard potential should be based on the worst-case probable scenario of failure or mis operation of the dam.

Table 2:3 FEMA Hazard Potential Classification System Hazard Potential

Hazard Potential	Loss of Human Life	Economic, Environmental and Lifeline Losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable. One or more expected	Yes

Source: (FEMA, 2013)

2.3. Dam Breach Analysis

2.3.1. History of Dam Breach Analysis

In the early 1980‟s, computer programs were developed to analyze the dam breaching process. As indicated by MacDonald & Langridge-Monopolis 1984 that those programs were limited by the accuracy of the breach geometry and failure timing information that was typically used as input. After that MacDonald & Langridge-Monopolis 1984 performed the first systematic analysis of a database of 42 existing dam failures in order to establish empirical relationships relating reservoir/dam dimensions to breach width, timing and peak discharge (Colorado, 2010). Similar statistical (regression) analyses were performed by the USBR 1988, Von Thun and Gillette 1990, Dewey and Gillette 1993 and Froehlich 1995a, 1995b to create their own empirical methods. A few empirical methods were also developed to predict breach peak discharge like the equation developed for the National Weather Service (NWS) Simplified Dam Break Model (SMPDBK) (Wermore, 1984).

2.4. Breach Models

2.4.1. Dam Breach Analysis Tools and Methods

The two primary tasks in the analysis of dam breach are the prediction of the reservoir out flow hydrograph and the routing of that hydrograph through d/s valley, predicting the breach characteristics such as shape, depth, width, rate of breach formation and routing the reservoir storage and inflow through the breach (Wahl, 2010).

There are four critical elements of any breach analysis:

1) Breach parameter estimation (breach size/shape and time of failure),

2) Breach peak discharge and breach hydrograph estimation,

3) Breach flood routing and

4) Estimation of the hydraulic conditions at critical locations. The most commonly used approaches for the required elements of the analysis are described briefly as follows (Colorado, 2010).

2.4.1.1. **Comparative Analysis**

Comparative analysis 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 (Colorado, 2010).

2.4.1.2. Empirical Methods

Empirical methods are used to predict time to failure and breach geometry, as well as to predict peak breach discharges. The empirical approach relies on statistical analysis of data obtained from documented failures. The four most widely used and accepted empirically derived enveloping curves and/or equations for predicting breach parameters are: MacDonald & Langridge – Monopolis 1984, USBR 1988, Von Thun and Gillette 1990, and Froehlich 1995a, 1995b, 2008. These methods have reasonably good correlation when comparing predicted values to actual observed values (Colorado, 2010).

2.4.1.3. Physically-Based Models

A physically-based 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. When the input is changing with time it may become complex. 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. The National Weather Service‟s BREACH program (NWS BREACH or BREACH) is 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. 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 (Tony L, 1998).

2.4.1.4. Parametric Models

HEC-1, HEC-HMS and HEC-RAS are parametric computer models that estimate the peak discharge and breach hydrographs from dam breaches based on parameters (breach geometry and breach development time) provided by the user. They can also be used to calculate the flood routing of the hydrograph downstream, and, in the case of HEC-RAS, can be used to estimate the hydraulic conditions at critical downstream locations (Colorado, 2010).

2.4.1.5. Hydrologic Models

Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end as indicated in USACE, 1994. Hydrologic routing models provide attenuated flow hydrographs at locations of interest, but do not provide useful information on water surface elevations or flow velocities. HEC-1 and HEC-HMS are the most widely used hydrologic models for dam safety analysis, and both contain a parametric dam breach routine that calculates the breach hydrograph (Colorado, 2010).

2.4.1.6. Hydraulic Models

Hydraulic models are more physically based than hydrologic models since they only have one parameter the roughness coefficient to calibrate. The basic data requirements for hydraulic routing techniques include: flow data, channel geometry, roughness coefficients, and internal boundary conditions. Hydraulic modeling is further subdivided into steady flow analysis and unsteady flow analysis.

HEC-RAS is the most widely used hydraulic model for dam safety analyses in the United States and can be utilized for steady and unsteady flow analyses. The latest versions of HEC-RAS have a parametric dam breach routine that can calculate a breach outflow hydrograph within an unsteady flow simulation (Colorado, 2010).

Another hydraulic model that has been widely used for unsteady flow analyses is the NWS DAMBRK model. The model is based upon the same basic unsteady routing hydraulic principles as HEC-RAS, but DAMBRK was specifically developed for modeling dam failures. The cross-section input requirements for routing dam break floods require the same number of points to represent every cross section, which limits its usefulness (Colorado, 2010).

2.5. Description Of The Model

2.5.1. HEC-RAS

The model package "River Analysis System" (RAS) by the US Army Corps of Engineers – Hydrologic Engineering Center (HEC) includes: a steady flow model and unsteady flow model the consideration of a wide range of hydraulic works, bridges, storage areas and facilities for hydraulic design such as computation of localized scour at the piles of a bridge (USACE, 2010). Due to its capability of describing that wide range of physical processes it has proven very helpful in supporting all phases of river management planning (Pistocchi, n.d.).

HEC-RAS supports both overtopping and piping failure modes with the failure trigger being a target water surface, water surface and duration, or specific time. To model a dam failure in RAS, enter the failure mode, breach size, and breach time. The breach size is defined by a trapezoid and the duration over which the breach occurs. Lastly, RAS allows the user to customize the progression of the breach over the full formation time(Cameroon T. Ackreman).

HEC-RAS has a very easy to use graphical users‟ interface (GUI) and this provides a highly efficient file management, data entry and editing, hydraulic analyses, and tabulation and graphical displays of input and output data (USACE, 2010).

Basically, the software has four 1D river analysis components: steady flow water surface computations; unsteady flow simulation; sediment transport computations; and water quality analysis. HEC-RAS also has a quite a number of options, such as mixed flow regime analysis ,allowing analysis of both sub- and supercritical flow regimes in a single computer run, culvert and bridge routines allowing for multiple openings of different types and sizes, quasi 2-D velocity distributions, and xyz graphs of the river channel system. The stream flow profile follows the basic physical laws: principle of conservation of mass and principle of conservation of momentum. These laws are expressed mathematically and referred as continuity and momentum equations. In unsteady flow, time dependent changes in flow rate are analyzed explicitly as a variable, while steady flow analysis models neglect time all together(USACE, 2010).

Steady flow water surface computation: Steady flow analysis can determine a water surface elevation and flow velocity at a given cross section for a given flow using Manning‟s equation under the assumption of gradually varied flow conditions. This component computes water-surface profiles and energy grade lines in 1-D for a steady and gradually-varied flow. It can handle a single reach, and full network of channels or streams. This component can model subcritical, supercritical, and mixed flow regimes‟ water surface profiles. The flow computations are based on basic physical laws: principle of conservation of mass (continuity equation) and principle of conservation of momentum (momentum equation).Energy losses are evaluated by Manning's equation and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied (USACE, 2010).

Unsteady Flow Simulation: Unsteady flow analysis can be used to evaluate the downstream attenuation of the flood wave, providing a more accurate estimate of flood magnitude and velocity at critical locations this component models 1D unsteady flow through a single or full network of channels or streams. The unsteady flow component was developed primarily for subcritical flow regime calculations (USACE, 2010).

2.5.2. HEC-GEORAS

HEC-Geo RAS is an ArcGIS extension developed by the HEC. This model contains a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. It assists in creating data sets in GIS to extract information essential for hydraulic modeling (Cameron T. Ackerman, n.d.).

After steady or unsteady flow simulation, HEC-RAS results can be exported for processing in the GIS by Geo RAS. The user can read the HEC-RAS results into the HEC-Geo RAS and perform the flood inundation mapping(Brunner, 2006).

HEC-Geo RAS is a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. The HEC-Geo RAS assists in creating datasets (referred to collectively as RAS Layers) in Arc-GIS to extract information essential for hydraulic modeling. The latest release of HEC-Geo RAS supports the extraction of elevation data from Digital Elevation Models (DEMs) in either TIN or GRID format. In this thesis, the GRID format is used (USACE, 2012).

Table 2:4 Summary of Hec_Geo RAS Layers and Corresponding output for HEC-RAS

RAS Layer	Description
Stream centerline	Used to identify the connectivity of the river network and assign river stations to computation points.
Cross-sectional cut lines	Used to extract elevation transects from the DEM at specified locations and other cross-sectional properties.
Bank lines	Used in conjunction with the cut lines to identify the main channel from overbank areas.
Flow path centerlines	Used to identify the center of mass of flow in the main channel and overbanks to compute the downstream reach lengths between cross sections.
Land use	Used to assign flow roughness factors (Manning‟s n values) to the cross sections.
Ineffective flow areas	Used to identify the location of non-conveyance areas.
Blocked obstructions	Used to identify obstructions to flow
Bridges	Used to extract the top-of-road data from the DEM at specified locations

RAS Layer	Description
Inline Structures	Used to extract the weir profile from the DEM for inline structures (i.e. dams).
Lateral structures	Used to extract the weir profile from the DEM for structures the pass flow perpendicular from the main channel.
Storage areas	Used to define the extent of detention areas and develop the elevation-volume relationship from the DEM.
Storage area connections	Used to extract the weir profile from the DEM for connections between storage areas.

(Tariku, 2015)

2.5.3. ARC GIS

The geographical information system, GIS is a system capable of capturing, storing, analyzing and displaying geographically referenced information. Hence, in our context, this model with its HEC-Geo RAS extension shall be used to study the areal distribution of the flood on downstream reach and to delineate the boundary of inundation

2.6. Dam Breach Parameter

In the analysis of dam breach the breach parameter have significant importance b/c accurate prediction of breach parameters is necessary to make reliable estimation of peak outflow and resulting downstream inundation in close proximity to the dam. For realistic modeling of inundation map for a dam breach event, the breach shape, size, location, and timing must be estimated for the considered type of dam and its modes of failures. A key element for calculating a dam breach hydrograph for a specific dam involves estimating the dam breach parameters for dam breach modeling related to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation (Tony L, 1998).

The following definitions are commonly accepted for use in evaluating and selecting dam breach parameters.

• **Breach formation time (also time-to-failure)** – The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached it full geometry.

• **Breach depth (also breach height)** – The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.

• **Breach width** – The breach width is the average of the final breach width, typically measured at the vertical center of the breach.

Breach side slope factor – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

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, is uncontrolled, and will result in the failure of the dam (FEMA, 2013).

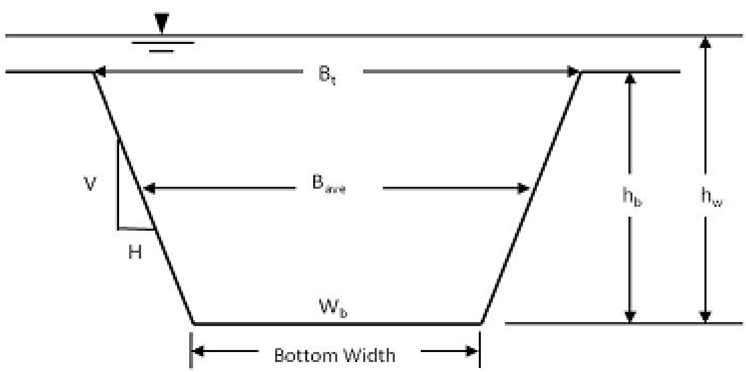


Figure 2:1 Breach Dimension

Table 2:5 Suggested Breach Parameters for Earth Dams

Source	Average Breach Width (ft)	Breach Side Slope (1V:ZH)	Breach Failure Time (hrs)
NWS (1988)	1H to 5H	Z = 0 to 1	0.1 to 2.0
COE (1980)	0.5H to 4H	Z = 0 to 1	0.5 to 4
FERC (1991)	1H to 5H	Z = 0 to 1	0.1 to 1.0
USBR (1982)	3H	N/A	0.00333b
BossDAMBRK (1988)	0.5 to 4H	Z= 0 to 1	0.5 to 4
Harrington (1999)	1H to 8H	Z= 0 to 1	H/120 to H/180
Note: H = Height of water against dam above breach bottom elevation in feet.			

(FEMA, 2013)

2.7. Dam Breach Parameter Estimation

2.7.1. Breach Parameter Estimation

There are various empirical equations available for estimating breach parameters on the basis of dam and reservoir characteristics i.e. dam height, and reservoir‟s volume and other physical characteristics. The most important component of a dam break analysis is the definition of reasonable breach parameters, which are highly difficult to be accurately predicted (Wahl, 1997). When population centers and associated critical sections are located well downstream of a dam, details of the breaching process and the calculated peak discharge may have little effect on the results. In this case, travel time, attenuation, and other routing effects tend to predominate. However, in a growing number of cases, the location of population centers near a dam makes accurate prediction of breach parameters (e.g. breach width, depth, and rate of development) crucial to the analysis. If breach parameters cannot be predicted with reasonable accuracy, more conservative assumptions and associated increased costs may be required(Wahl, 1997). A number of approaches and empirical equations were put to test for this analysis and the following equations were used widely used for various dam breach modeling studies: Froehlich (1995), Froehlich (2008), MacDonald and Langridge-Monopolis (1984), and Von Thun and Gillette (1990) (USACE, 2016) .

Froehlich (1995): The Frohelic 1995 utilized 63 earthen, zoned earthen, earthen with core and rockfill data sets to develop a set of equations to predict average breach width, sideslpes & failure time.

BREACH DEVELOPMENT TIME : The expression developed for the breaching time (*t*, hr.) is (Froehlich 1995)

$t = 0.00254V^{0.53}\ h^{-0.9}$

BREACH WIDTH :An expression for the breach width (*B*, m) was also developed (Froehlich 1995)

$B = 0.1803V^{0.32}\ h^{0.19}$. (USACE, 2016)

Froehlich (2008): Froehlich analyzed a total of 74 earthen, zoned earthen, earthen with a clay core wall, and rockfill dams‟ data sets and undertook regression analysis. Following these,the derived equations for the average breach width and breach formation time (USACE, 2016).

$$B_{avg} = 0.27K_oV_w^{0.32}h_b^{0.04}$$

$t_f = 63.2\sqrt{\frac{V_w}{gh_b^2}}$

Where:

B_{avg}= Average breach width (meters)

K_o= Cnstant (1.3 for overtopping failures, 1.0 for piping)

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

h_b= Height of the final breach or dam height from river bed level (i.e. which is taken as dam height) (meters)

g= Gravitational acceleration

t_f= Breach formation time (seconds)

Froehlich states the average side slopes for overtopping failures as 1.0 H: 1V. (USACE, 2016).

MacDonald and Langridge–Monopolis (1984): MacDonald and Langridge-Monopolis analyzed a total of 42 earthen, zoned earthen, earthen with a clay core wall, and rockfill dams" data sets to develop "Breach Formation Factor". Breach Formation Factor is basically a product of the height of water above the dam crest and the volume of water leaving the dam. Then, they related this factor to the volume of embankment material eroded (USACE, 2016).

Following are equations for volume of eroded material and breach formation time:

For earthfill dams:

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

$t_f = 0.0179(V_{erroded})^{0.364}$

For earthfill with clay core or rockfill dams:

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

Where:

V_{eroded} = volume of eroded material from the dam embankment (m^3)

V_{out} = volume of water leaving through the breach (m^3) i.e. storage volume at time of breach plus volume of inflow after breach begins, minus any spillway and gate flow after breach begins.

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

t_f = breach formation time (hours).

V_{out} is not accurately known prior to performing the breach analysis; thus, the first estimate is to set it as the volume of water in the reservoir when breach starts. With this first estimate analysis can be run and one can re-evaluate the initial estimate then make better estimates of the actual volume of water passing through the breach and redo the breach analysis. Basically, this is an iterative process where calculations continue till the estimated volume at the start of each calculation and the end of the analysis converge to some degree.

Here, breach shape is assumed trapezoidal with side slopes of 0.5H: 1V. The bottom width of the breach is then estimated with the following equation (State of Washington, 1992):

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

Where:

W_b = bottom width of the breach (meters)

h_b= height of the final breach or dam height from river bed level (i.e. which is taken as dam height) (meters)

C = crest width of the top of dam (meters)

$Z_3 = Z_1 + Z_2$

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

Z_2 = average slope (Z_2:1) of the downstream face of dam

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

Von Thun and Gillette (1990): Von Thun and Gillette utilized 57 dams from the Froehlich (1987) paper and the MacDonald and Langridge-Monopolis (1984) papers to derive their methodology and equations. With this, the equation for average breach width is presented as (USACE, 2016):

$B_{avg} = 2.5h_w + C_b$

Where:

B_{avg}= average breach width (meters)

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

C_b= coefficient which is a function of reservoir size

Reservoir size (Mm3)	Cb
< 1.23	6.1
1.23-6.17	18.3
6.17-12.3	42.7
>12.3	54.9

Here, breach shape is assumed trapezoidal with side slopes of 1H:1V, except for dams with cohesive soils, where the method stated side slops here should be between 0.5H:1V to 0.33H:1V. As for breach formation time, Von Thun and Gillette used two sets of equations. The first set is only a function of water depth above the breach bottom, and the second set is a function of water depth above breach bottom and average width of the breach.

The first set of equations shows breach development time as a function of water depth above the breach bottom:

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

$t_f = 0.015h_w$ (Easily erodible)

Where: t_f= breach formation time (hours)

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

The second set shows t_f as a function of water depth (h_w) above the bottom of the breach and average breach width (B_{avg}).

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

$t_f = \frac{B_{avg}}{4h_w+61}$ (Easily erodible) (USACE, 2016)

2.8. Dam Classification Systems

The hazard potential classification of a dam, along with its size (height and capacity) classification, is used by State agencies to regulate dam design and dam breach modeling. In USA Common practice among Federal and State Dam Safety agencies is to classify a dam according to the potential consequences of a dam failure on areas located downstream of the dam. FEMA guidance recommends a three-step rating system that defines low-, significant-, and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure. In addition, guidance developed by the USACE incorporates size classification determined by the dam"s height and storage volume. Some States have additional hazard potential categories such as “extreme hazard” and “very low hazard” and/or have added additional classifications to account for the size of the dam (height and capacity) (FEMA, 2013). The selection of the inflow design flood (IDF), also referred to as the spillway design flood (SDF), according to the assigned hazard potential rating and size classification. The assigned ratings establish the flood events (dam breach scenarios) used in dam breach modeling for design purposes and for use in EAPs (FEMA, 2013).

2.8.1. Dam Hazard Classification

Hazard Potential Classification is a system that categorizes dams according to the degree of adverse incremental consequences of a failure or mis-operation of a dam. The hazard potential classification does not reflect in any way on the current condition of the dam (e.g., safety, structural integrity, flood routing capacity (FEMA, 2004). Downstream Hazard Classification does not correspond to the condition of the dam or appurtenant works, nor the anticipated performance or operation of the dam. Rather, it is descriptive of the setting in areas downstream of the dam and is an index of the relative magnitude of the potential consequences to human life and development should a particular dam fail (Damsafety, 2007). In most situations, the investigation of the impacts of failure on downstream life and property is sufficient to determine the appropriate hazard potential rating; however, there may be circumstances where further evaluation is appropriate. For example, the reservoir of a dam that would normally be considered to have a low-hazard potential based on insignificant flooding due to failure may be known to contain toxic sediments, such as may exist in a tailings pond. Therefore, a low-hazard potential rating may not be appropriate and instead a higher standard may be more appropriate to classify the hazard potential (FEMA, 2013). FEMA guidance recommends that the hazard potential rating be based on consideration of the effects of a failure or mis operation during both normal and flood flow conditions. FEMA further recommends that the hazard potential should be based on the worst-case probable scenario of failure or mis operation of the dam.

Dams are classified according FEMA, 2004

• *High-hazard potential* – Dams assigned the as high-hazard potential classification are those where failure or mis operation will probably cause loss of human life.

• *Significant-hazard potential* – Dams assigned the significant-hazard potential classification are those dams where failure or mis operation are not likely to result in loss of human life but may cause economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns. Significant-hazard potential classification dams are often located in predominantly rural or agricultural areas but may be located in areas with population and significant infrastructure.

• *Low-hazard potential* – Dams assigned the low-hazard potential classification are those where failure or mis- operation are not likely to result in loss of human life and only low economic and/or environmental losses. Losses experienced are likely limited principally to the owner‟s property.

Table 2:6 USACE Dam Hazard Potential Classification system

Category	Hazard Potential Classification		
	Low	Significant	High
Direct Loss of Life	None expected (due to rural location with no permanent structures for human habitation)	Uncertain (rural location with few residences and only transient or industrial development)	Certain (one or more extensive residential, commercial or industrial development)
Lifeline Losses	No disruption of services – repairs are cosmetic or rapidly repairable damage	Disruption of essential facilities and access	Disruption of critical facilities and access
Property Losses	Private agricultural lands, equipment and isolated buildings	Major public and private facilities	Extensive public and private facilities
Environmental Losses	Minimal incremental damage	Major mitigation required	Extensive mitigation cost or impossible to mitigate

Source (FEMA, 2004)

2.8.2. Population at Risk

The potential for loss of life is the primary factor in determining the downstream hazard classification. For purposes of classification, the Population at Risk (PAR) is used to represent the potential for loss of life. This essentially corresponds to the number of people who would have to be evacuated from downstream areas in the event of a dam failure. Population at risk is defined as the number of people who may be present in areas downstream of a dam and could be in danger in the event of a dam failure (Damsafety, 2007).

2.8.3. Property Damage and Economic Losses

Property damages would include damage to inhabited dwellings, commercial and production buildings, agricultural lands and crops, livestock, roads, highways and utilities and the associated economic losses both permanent and temporary. The intent, in considering the potential property damage and economic loss, is to identify the relative magnitude of losses against a broad scale of values (Damsafety, 2007).

2.8.4. Environmental Damages

Consideration of environmental damages would address situations where the reservoir contains materials which may be deleterious to human or aquatic life or stream habitat. This applies to projects such as: domestic and agricultural waste lagoons; industrial waste lagoons; and mine tailings dams where the reservoir may contain trace amounts of heavy metals, chemical residues from ore processing, or large volumes of sediment in a loose or slurry condition. This would apply to streams with fisheries of regional significance where large scale channel scour and sediment deposition are likely to result from a dam break flood (Damsafety, 2007).

2.8.5. Current/Future Development

The downstream hazard classification should reflect the current downstream development and the associated consequences of dam failure. When using the classification it is advisable to investigate the effect that future downstream development may have in increasing the classification and increasing the minimum design standards/criteria at a given dam (Damsafety, 2007).

3. STUDY AREA DESCRIPTION

3.1. Location

The Awash basin is part of the Great Rift Valley in Ethiopia located from 8.50 N to 12N.

It covers the central and northern part of the rift valley and is bounded to the west, southeast and south by the Blue Nile, the Rift Lakes and the Wabi- Shebele Basins respectively. (Behailu, 2004)

The Awash Basin has been traditionally divided into four distinct zones. These are; Upper Basin, Upper Valley, Middle Valley and Lower Valley. (Behailu, 2004)

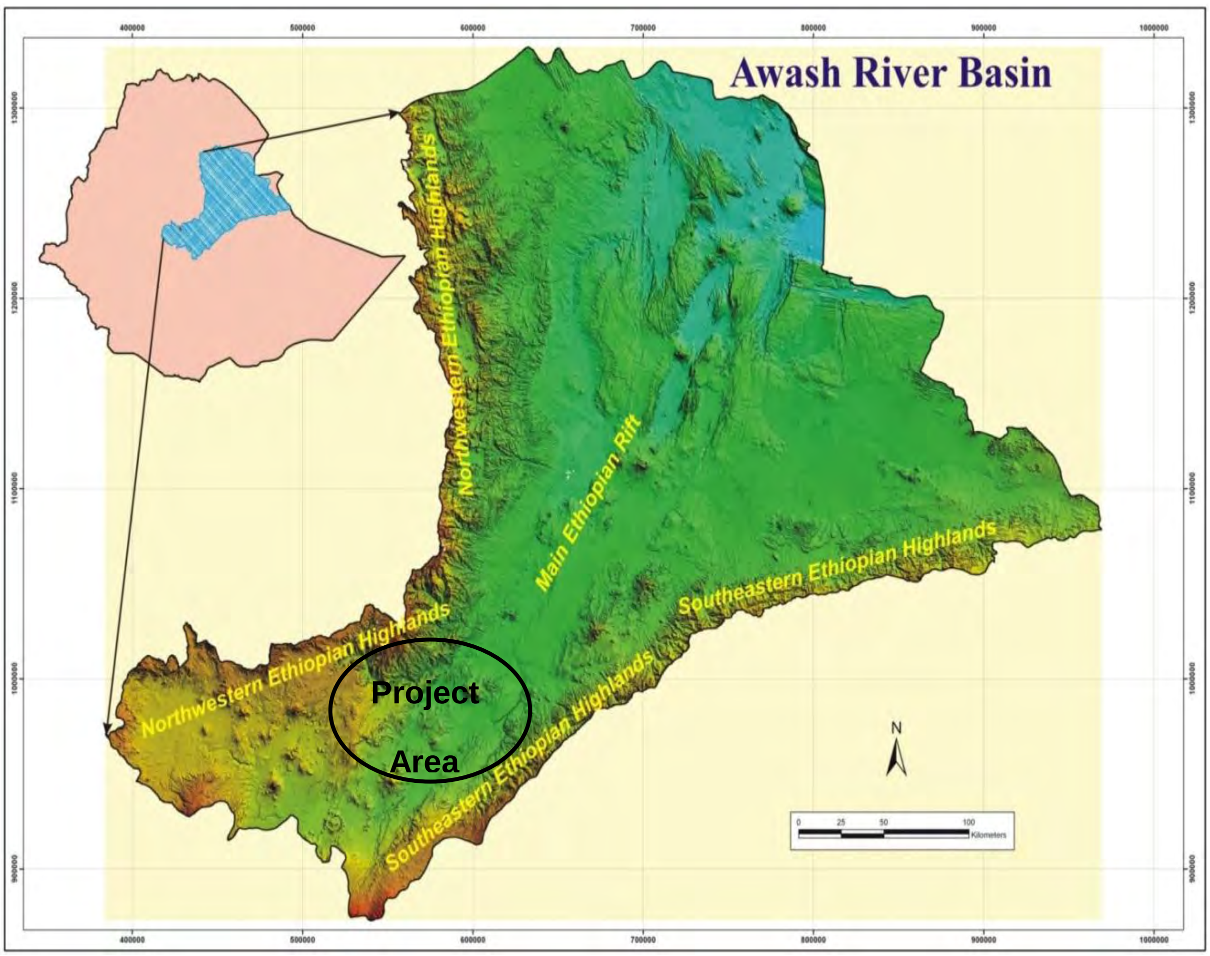


Figure 3:1. Awash River Basin and Middle Awash Multipurpose Dam Project Area(Source: WWDSE dam design feasibility report

3.1.1. Study Project Location

The Middle Awash Multipurpose dam project is found in the Middle Awash valley which is in Afar National Regional State and Oromia National Regional State. As WWDSE(2015) the dam site is located close to Awash town about 200kmfrom the capital, Addis Ababa on Main Awash River just 7km upstream of the Addis Ababa-Djibouti road main bridge on Awash River. The proposed height of the dam is 120m..

The purpose of the middle awash multi-purpose project is for flood control to protect the downstream community and infrastructure and serve to harness the water resources of Awash River for irrigation. (WWDSE, 2015)

Figure 3:2 Study Area

3.2. Topography

The area is prone to flooding by river Awash, which has a tendency to change its course very often (WWDSE, 2007). The altitude of the study area varies from 1598m upstr.a.m.s.l. to 709m.a.m.s.l. at downstream. For reach and floodplain cross-sectional geometry generation and for downstream inundation mapping Study domain DEM is used.(Figure 3:3. Study Area DEM)

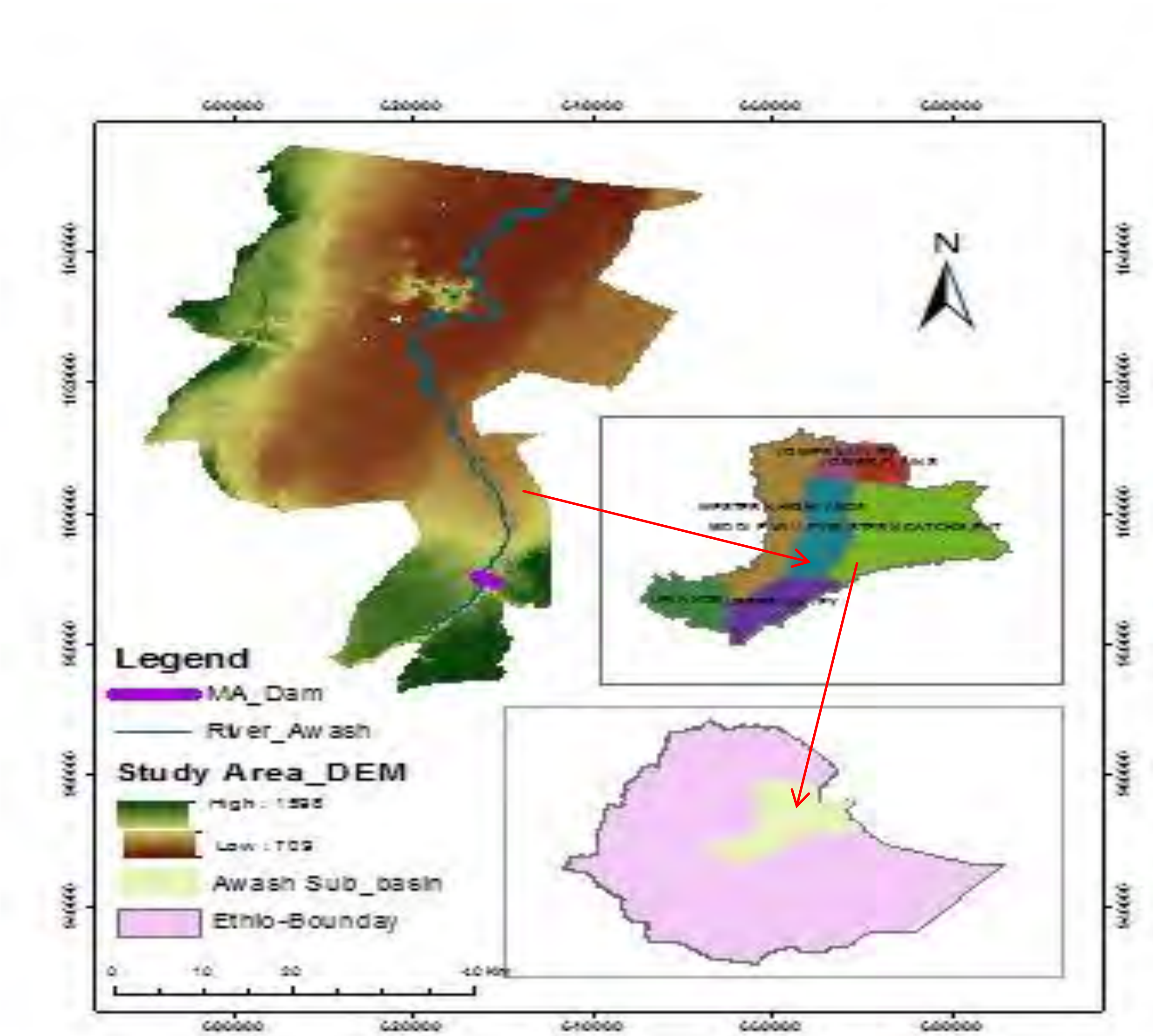


Figure 3:3. Study Area DEM (Source: ETHIO-GIS)

3.3. Climate

The climate of the Awash basin is influenced the Inter-Tropical convergence (ITCZ). This zone of lower pressure makes the convergence of tropical easterlies and the moist equatorial westerlies. The seasonal rainfall distribution within the basin results from the south, bringing the small or spring rains. In June & July it gets its most rains, and then returns south wards during August to October restoring the drier easterly air stream which prevails until the cycle repeated itself in March. .The mean annual rainfall varies from about 1600mm at Ankober in highlands north east of Addis Ababa observatory to 160mm at Asayita on the Northern limit of basin (WWDSE, 2015).

3.4. Catchment characteristics.

The Awash river drains to northerly part of the Rift Valley in Ethiopia from approximately 8.5 ºN to 12 ºN with total drainage area of 112. 211 km^2 (Halcro-, 1989).

3.5. Land Use /Cover

At downstream of Middle Awash dam site Amibara Woreda is found. Based on the population and housing census of 2007, Amibara Wereda has total population of 78,105.Livestock husbandry. state irrigated mechanized farm producing cotton and sesame , Cultivation at , maize, sorghum, Teff, wheat, cotton, sugarcane, sweet potato, mango, onion, tomato, banana, crops .Others are shrubs and some wood land around river bank (WWDSE, 2014).

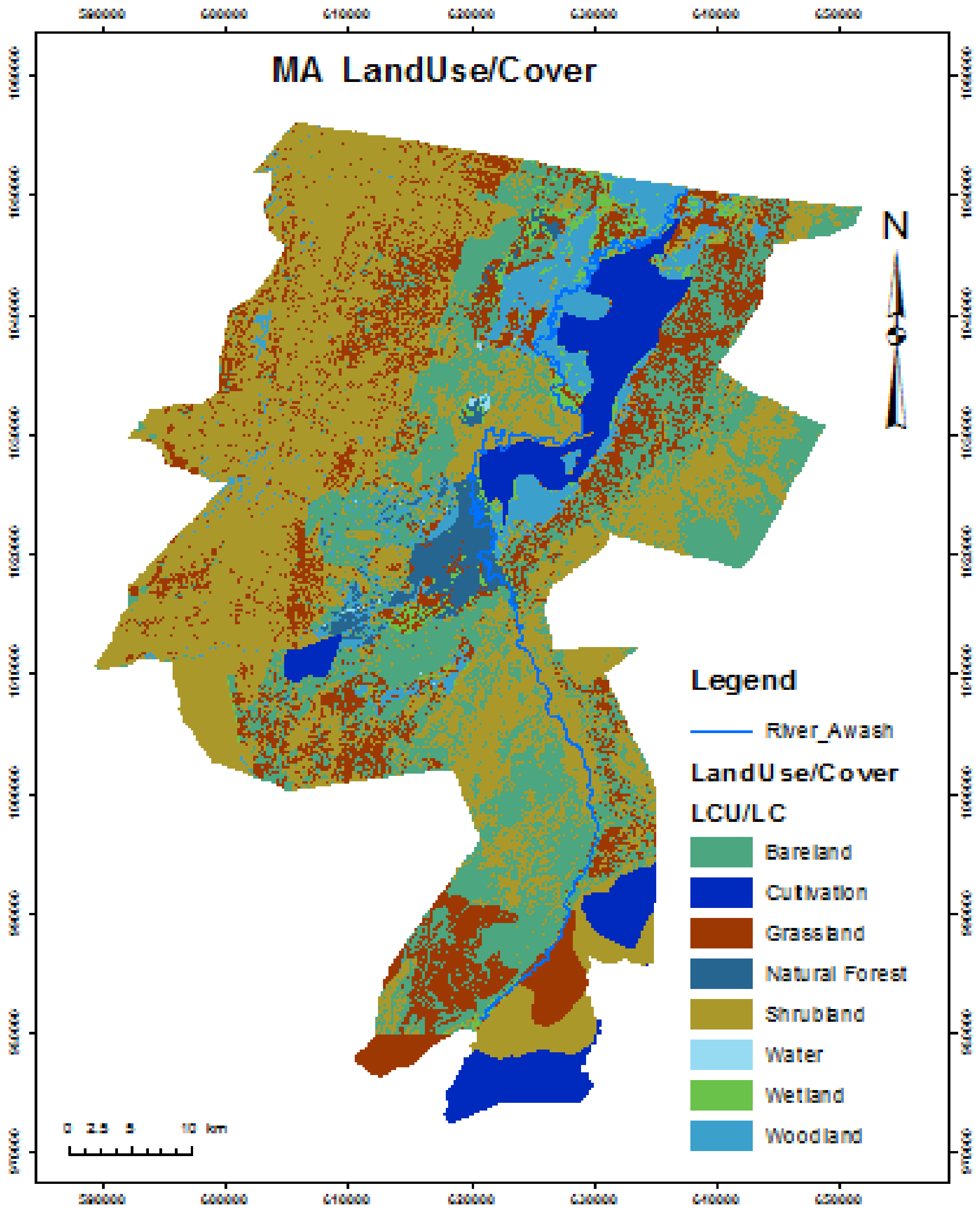


Figure 3:4 Land Use/Cover(Source:WWDSE S.-E. R., 2014)

3.6. The Dam

Midde Awash Muti purpose dam is constructed on Awash river. The dam is a zoned (clay core rock fill) embankment type dam, Its height is 120m at river bed section having the dam crest level at 943 m. The main dam body is designed Rock fill with central clay core, downstream and upstream filter, drainage zone and transitions are provided for protection to drain seepage water and smooth transition from fine to coarse textured dam fill materials (WWDSE, 2015). As WWDSE(2015) the dominant lithologic units around the foundation area are rocks (Rhyolite, Ignimbrite, and Basalt), old alluvial, colluvial deposits, fault breccia and tuff with layers of ignimbrite material. In order to minimize seepage through the foundation, cutoff trench and cement grouting provisions of seepage curtailing measures are considered.

The reservoir is intended to accommodate a total of 501 million cubic meter of water.The maximum water level adapted for Middle Awash Dam design is fixed as the water head over the spillway corresponding to the maximum water level is approximated to be 938.0m (WWDSE, 2015). The reservoir area volume elveation is avaiabe(Annex9).

Table 3:1: Middle Awash Dam Characteristics

Parameters	**Middle Awash Dam**
Dam height	120m
Dam crest elevation	943m
Spillway crest elevation(NPL)	926m
Capacity	501Mm^3
Dam Type	Earth Dam: clay core rock fill dam

(Source WWDSE Design Report)

4. METHODOLOGY

4.1. Data Collection

The Geometrical and other data required to simulate the study dam failure in Hec-Ras were used. Most of the data are generated and most are gathered from Water Works Design and Supervision Enterprise.

A 30 meter resolution DEM map of the study area were available. Due to the difficulty of collecting land surveying because of the fact that the study area is very wide to cover in a short period of time, inaccessibility of river channel and flood plain to some extent because it is covered with bushes and shrubs and in general the financial constraint hinders not to overcome. Hence, a DEM with 30x30m resolution has been used for the study.

These data were imported to ArcGIS for preprocessing of the necessary inputs to HEC-RAS model. The Geometric data are extracted from DEM in ArcGIS with HEC-Geo RAS which ultimately be fed to HEC-RAS for dam break modeling. In addition topographical data, land use map and hydrological data of the area (i.e. Probable Maximum Flood (PMF) inflow hydrograph (Annex 9).

Dam characteristics includes data about name of dam, dam type, dam size, location of the dam, design water storage pool elevation, spillway crest elevation, crest of dam elevation, and height of the dam measured from downstream toe to the crest, and category of the dam and general information of the infrastructure such as city, town, and country area, geographic information, watershed boundary, and others.

Geometric data such as river route, banks, cross sections, and flow paths were extracted from DEM using HEC-GeoRAS and then these data were imported into HEC-RAS and reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported. Data gathered under Downstream includes bank stations, reach stations, downstream developments, cross section plots.

These data were extracted from study area DEM and others are collected from the Water Works Design and Supervision Enterprise (WWDSE), which is undertaking the design of MAMP dam and information from the books, Papers, journals and internet.

4.1.1. Identifying Physical Descriptions of Dam

This step includes the identification of dam height, dam crest width, spillway elevation and width and coefficient of discharge. In this study, data about the physical characteristic and cross section of the Middle Awash Dam is used as an input in HEC-RAS modeling.*(Table 3:1)&Figure(4:1)*

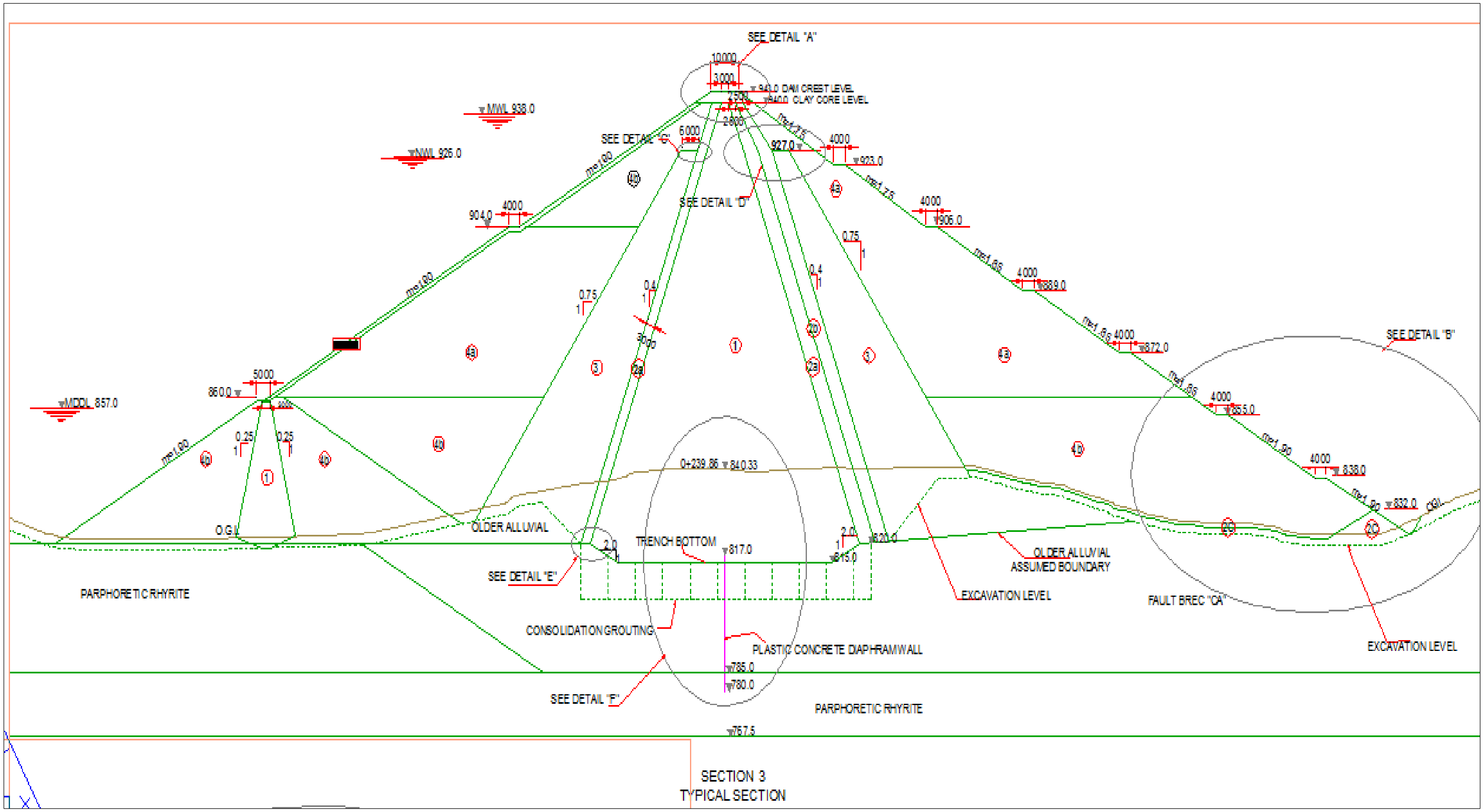


Figure 4:1 Dam Cross Section(Source:WWDSE, Design Report)

ZONING OF DAM

ZONE 1	-	IMPERVIOUS CLAY CORE
ZONE 2A	-	FINE FILTER F1 SEE Detail "E" and "C"
ZONE 2B	-	COURSE FILTER F2 SEE Detail "E"
ZONE ZONE 2C	-	HORIZONTAL BLANKET AND TOE DRAIN
ZONE 3	-	GRANULAR INNER SHELL
ZONE 4A	-	ROCK FILL EXCAVATED FROM QUARRY
ZONE 4B	-	ROCK FILL EXCAVATED FROM SPILLWAY
ZONE 5	-	RIPRAP

4.1.2. Inflow Hydrograph to the Reservoir

In order to analyze the worst case of dam breach analysis the inflow design flood is used. The dam to breach inflow hydrographs generated from PMF was used for the breach analysis.

PMF is estimated from Probable Maximum Precipitation (PMP) which is a theoretical maximum precipitation that a given watershed can experience. HEC HMS model was used to estimate the PMF. PMF inflow hydrograph for the catchment at upstream end of of Middle Awash is taken from the Middle Awash hydrological study report of WWDSE(2015) (Annex…9.). As WWDSE(2015) the 10,000 year return period flood peak estimated is 4483.6 m3/s.

4.2. Materials Used

The dam break analysis is done using HECRAS model which is widely used for river analysis with HEC_GEORAS an extension of ARCGIS. The model was selected for the study because of its availability, universality and usability.

To obtain River network and cross sectional geometry and to produce inundation mapping Arc Map GIS 10.0 and HEC-GeoRAS 10.0 tools were used, HEC-RAS 5 Beta Version model for dam break simulation and unsteady flood routing at downstream, Arc GIS study area infrastructure shape files and Google earth for remote sensing of the developed infrastructures like hydraulic structures, towns and current land use information..

4.3. Data Analysis

After getting relevant information and data of the study area the next step is analysis of the collected data. In this thesis study, only secondary data sets were collected and used as input data to HEC-Geo RAS model to generate reach and floodplain cross-section in the study area for input to HEC-RAS Model.

4.3.1. HECGEORAS Model Setup

HEC-Geo RAS is an extension developed by the USACE for use with ArcGIS to process geospatial data to support hydraulic model development and analysis of water surface profile results. The HEC-Geo RAS assists in creating datasets (referred to collectively as RAS Layers) in Arc-GIS to extract information essential for hydraulic modeling. HEC-Geo RAS supports the extraction of elevation data from Digital Elevation Models (DEMs) in either TIN or GRID format. (USACE, 2012)

In this thesis HEC-Geo RAS is used for the development of RAS Layers for extracting cross section data like stream center line, bank lines, flow path and cross section cut lines from Digital Elevation Models (DEMs) in GRID format. In the pre-processor stage, the river centerline was defined using the GIS -Hydro‟ results of the watershed delineated using GIS. Both left and right banks were determined off setting the river centerline to a distance measured on „Google Earth‟ and cross-checked with data collected from site office. Flow path lines, which are centerlines of the flood flowing through the channel, left flood plain and right flood plain, are arbitrarily provided and then adjusted after observing the flood extent.

4.3.1.1. Defining the River Geometry

Using HECGEORAS Ras Geometry Geometric data such as river route, banks, cross sections, and flow paths were extracted from Digital Elevation Model (DEM of 30mx30m resolution) with Ras Geometry in HECGEORAS of the study area Middle Awash and then these data were imported into HEC-RAS. In addition to these data, reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported. (Figure 4.1) The cross sections were located to adequately describe geometric features roughness changes, grade breaks, expansions and contractions, and the numerical requirements for the solution scheme used by HEC-RAS. The cross sections were drawn to remain perpendicular to the expected maximum flood wave flow lines.

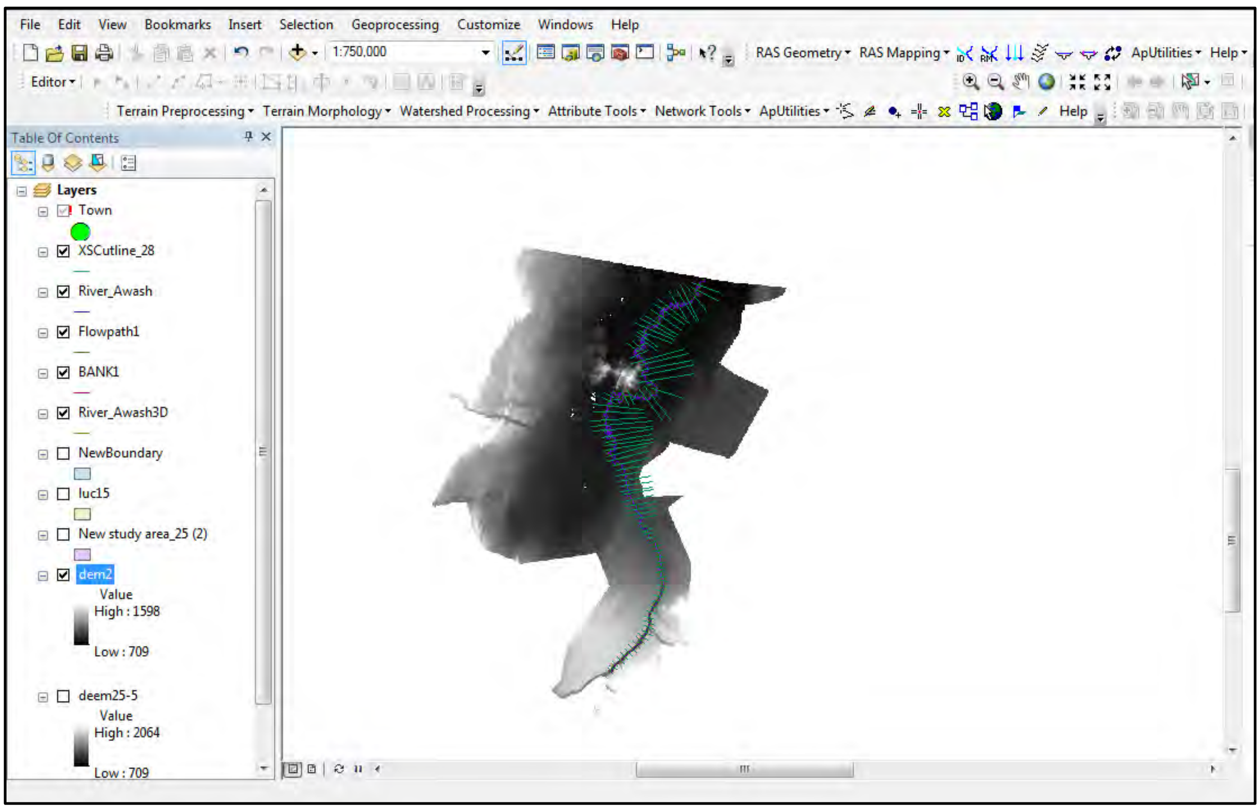


Figure 4:2 . ArcGIS HECGEORAS Window extracted cross sectional data from DEM

4.3.2. HECRAS Model Setup/Data in HECRAS

HEC-RAS has the ability to model flood events and produce water surface profiles over the length of the modeled stream..

4.3.2.1. Geometric Data

After River Geometry is imported from HECGEORAS to HEC-RAS in GIS format by adjusting the unit system to SI unit system the HECRAS project is saved.

Geometric data such as river route, banks, cross sections, and flow paths were imported into HEC-RAS form HECGEORAS. In addition to these data, reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported.

These exported Cross sections are used to define the shape of the stream and its characteristics, i.e. roughness, expansion and contraction losses, and ineffective flow areas. Cross sections were extracted from the DEM to define the terrain of the expected flood path. They were located in such a way that geometric features such as roughness changes, elevation breaks, expansion and contractions, and numerical requirements for model stability were to be adequately met. The geometry of the Awash River generated from ArcGIS was exported to HEC-RAS. With respect to this, cross sections were spaced at 100m intervals (here interpolations were done in cross sections where the interval is greater than 100m). *(Figure4.2)*

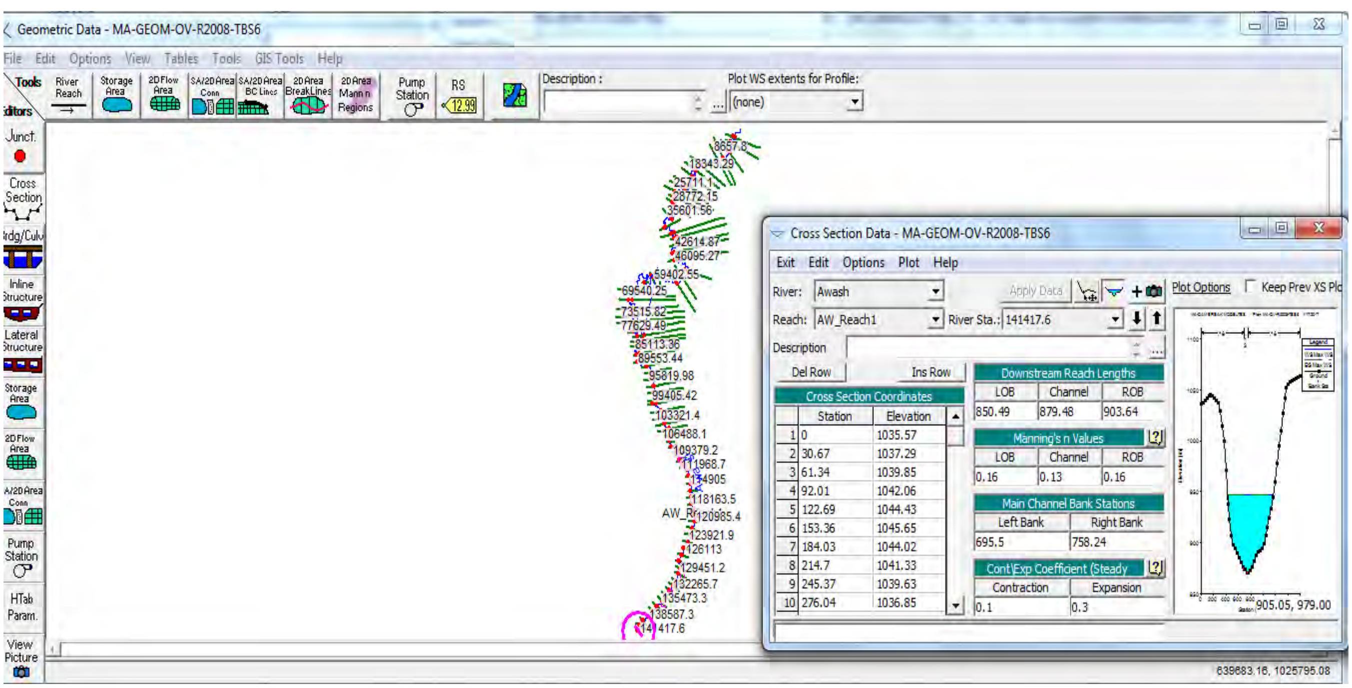


Figure 4:3 Imported Geometric data in HEC-RAS: (a) River and cross section cutline; (b) cross section data editor

4.3.2.2. Breach Model in HECRAS

In Dam Break analysis the dam feature is entered to HECRAS as Inline Structure Either extracted in HECGEORAS and exported to HECRAS or entered in HECRAS with geometry HECRAS window using Inline structure data editor. Then, the dam embankment and spillway crest data were provided using inline structure weir station elevation editor, and breach parameters were estimated using the built-in parameter calculator in HEC-RAS. For this study the Middle Awash Dam geometry is entered with weir/embankment data editor in inline structure editor window.

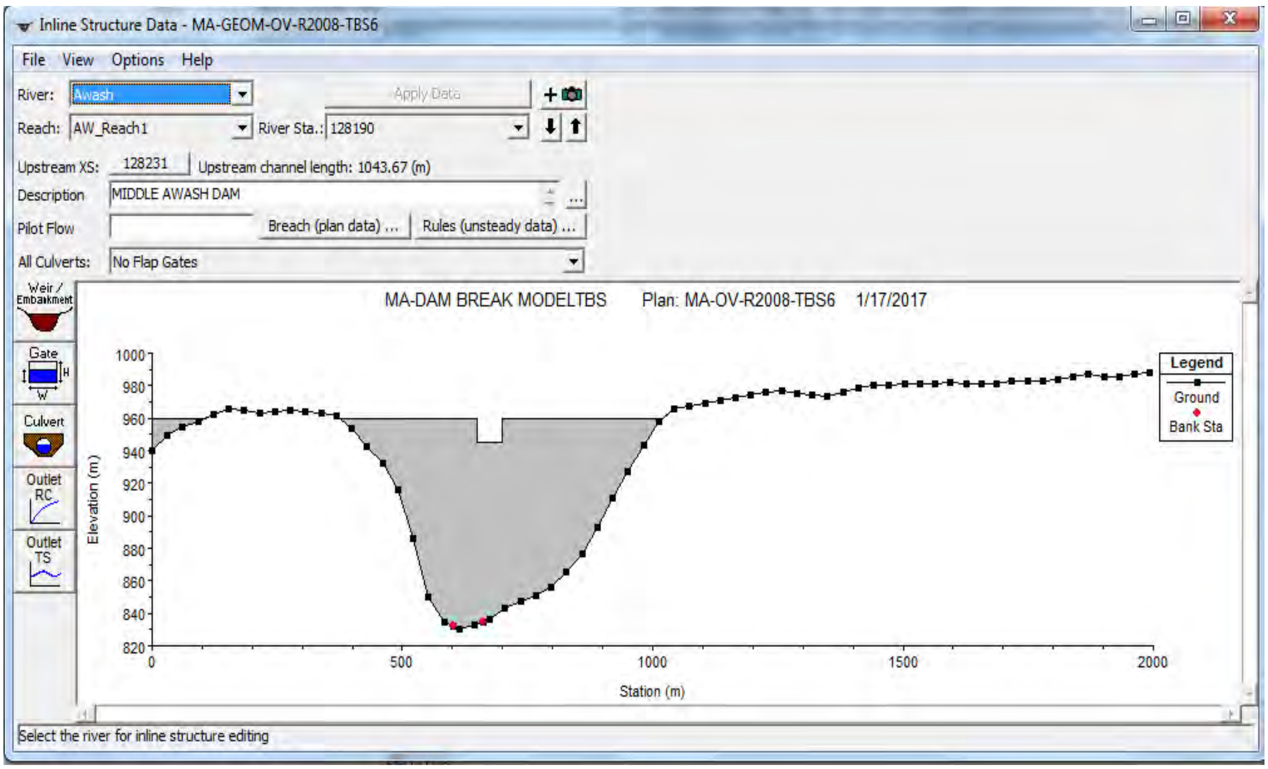


Figure 4:4 Inline Structure (Middle Awash Dam)

In the inline structure data window, breach parameters can be added using the breach plan data editor. The breach parameters were done for each failure mode and breach method. Here, the trigger breach elevation was set to 943.1 m. which is 0.1m above the MWL i.e. 943m. The pool volume at failure was taken from the dam design capacity.

Moreover, the breach progression with time was also defined using the same editor under breach progression menu.

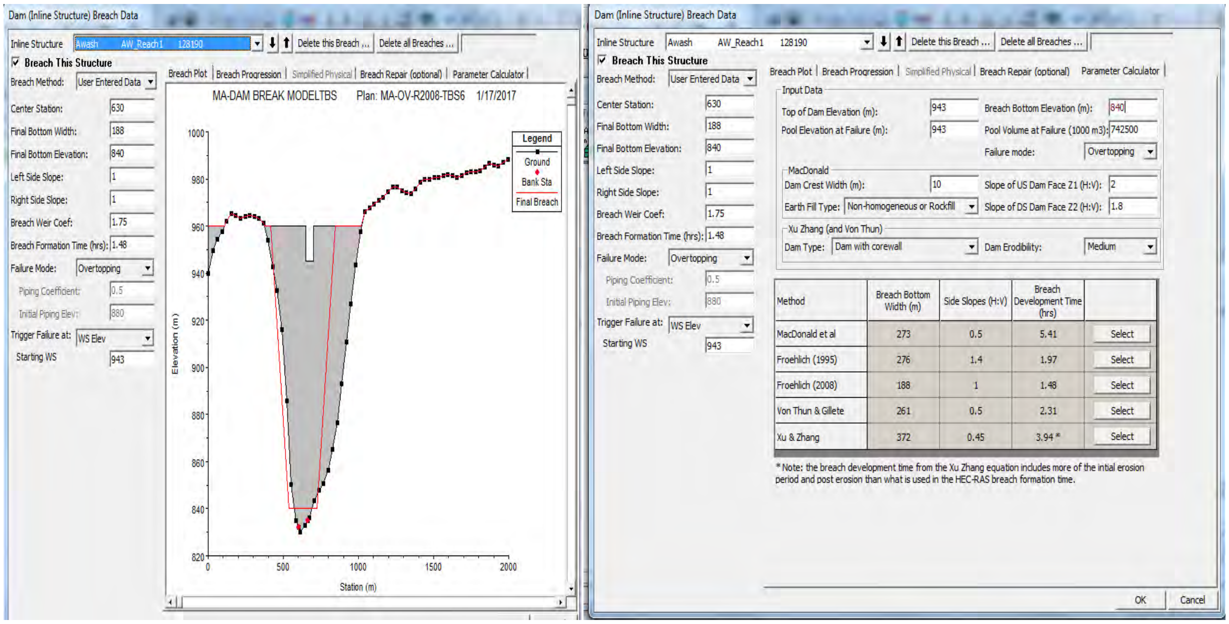


Figure 4:5 Breach Plan Data Editor

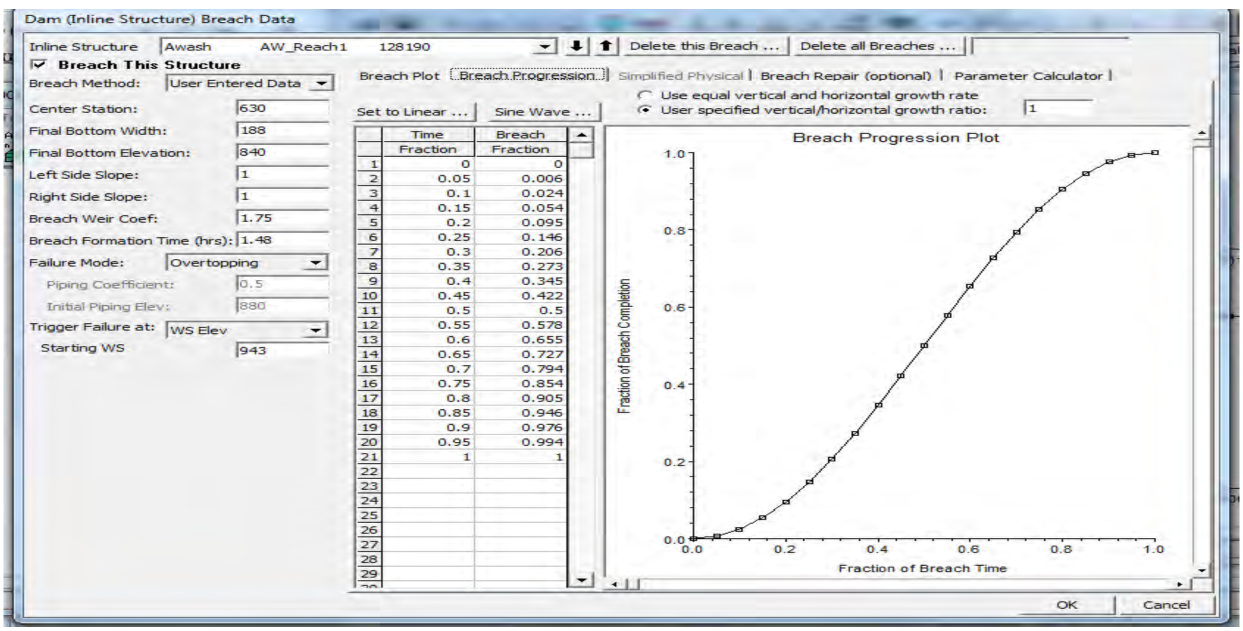


Figure 4:6. Breach Progression with Time

4.3.2.3. Estimating Dam Breach Characteristics

For realistic modeling of inundation map for a dam breach event, the breach shape, size, location, and timing must be estimated for the considered type of dam to determine a dam breach hydrograph, its modes of failures and flood exent.

4.3.2.3.1. Breach parameters

The most important component of a dam break analysis is the definition of reasonable breach parameters, which are highly difficult to be accurately predicted. There are various empirical equations available for estimating breach parameters on the basis of dam and reservoir characteristics i.e. dam height, and reservoir‟s volume and other physical characteristics.

The breach development and breach outflow hydrograph modeling in HEC-RAS demands parameters which define breach geometry such as breach shape, average breach width (B_{avg}), time to failure (t_f), pool elevation at time of failure, weir and orifice coefficients, and breach side slope

A trapezoidal breach which progresses with time was assumed, and the location of the breach was assumed at the centerline station of the dam. The breach width and breach time have a great influence on the forecast of the outflow and the flooded area downstream the dam. The developing time for a breach is defined as the point where dam failure is imminent and end when the breach has reached its maximum size.

The breach development and breach outflow hydrograph modeling in HEC-RAS demands parameters which define breach geometry such as breach shape, average breach width (Bavg), time to failure (tf), pool elevation at time of failure, weir and orifice coefficients, and breach side slope

In this study breach parameters are estimated in HECRAS. Each breach method considers different approach for the estimation of the outflow done in HECRAS because gives different result of the same dam characteristics input. The dimensions of the dam are known from the plan for the breach methods Macdonald, Frohelic (1995) and Frohelic (2008). The breach discharge is the out flow produced from the HECRAS simulation.

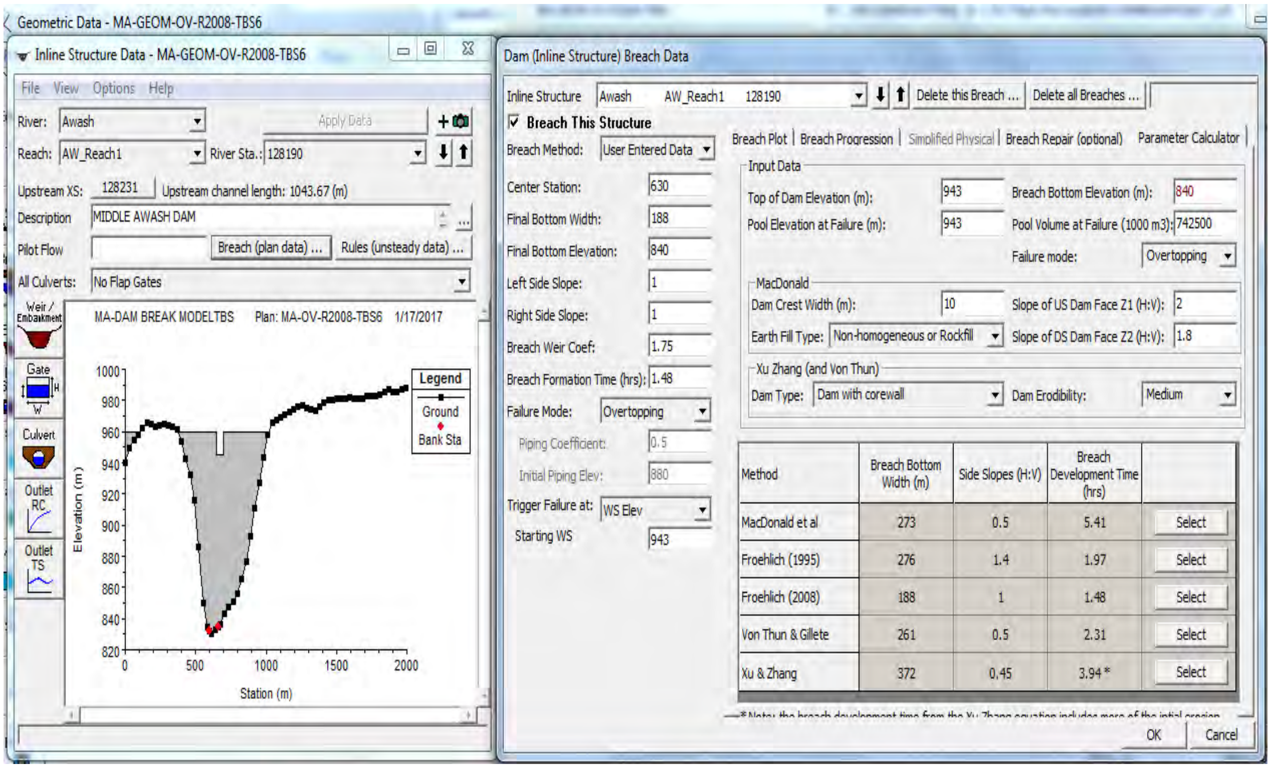


Figure 4:7 Breach Plan Window

With these, the breach parameters for MAMP dam were estimated and the parameters are presented.

4.3.2.4. Unsteady flow data

To model the dam breach process in HEC-RAS, an unsteady flow calculation is performed for 20 days period of simulation selected by the modeler were used.

Boundary condition data must be entered for the unsteady analysis. For the purposes of unsteady flow analysis, upstream boundary conditions are typically flow hydrographs; PMF hydrograph (for hydrological induced failures), or a constant inflow (for sunny day failure analyses).

Downstream boundary conditions can be set to normal depth, a rating curve, a known water surface elevation, or critical depth.

The upstream boundary condition for unsteady flow analysis is taken for PMF hydrograph of Awash River upstream of the proposed Middle Awash Multi-purpose Dam was estimated through rainfall-runoff modeling of the basin.

Here, the downstream boundary conditions are input as a normal depth with friction slope of 0.0001.

Table 4:1: Boundary condition data for MA Dam breach model

Boundary Condition	
Upstream BC	Downstream BC
Flow Hydrograph(PMF)	Normal Depth

4.3.2.5. Running unsteady flow simulation

Once the geometries and flow data entered, HEC-RAS simulates unsteady flow through the channel and flood plain in unsteady flow simulation window. The program is run for Geometry processor, unsteady flow simulation and post processor. Simulation date and time duration is set by the modeler. The simulation time window is run for 20 days and computed for 1 minute interval and mapping, hydrograph and detailed output interval of 1 hour for mixed flow regime.

4.3.3. Flood Mapping

After running the model, the in-built RAS Mapper tool in HEC-RAS used for flood mapping, and other computations related with the flood such as depth, velocity, flood arrival time and duration.

5. RESULTS AND DISCUSSIONS

The dam break analysis was done for Middle Awash Dam and mapping of flood inundation for downstream area to meet the objectives of the study by using HECRAS MODEL. The major inputs for dam breach analysis are data such as geometry and properties of the dam, reservoir information, inflow hydrograph and breach characterstics. There are different dam failure modes (Table2:2). Middle Awash dam is of rock fill dam with central clay-core material, and the failure modes considered for analysis in HEC-RAS are piping (i.e. under sunny day condition) and overtopping failure(i.e. under hydrological induced condition). After simulation it is reaveld that Middle Awash Dam can fail for both overtopping for event of peak PMF and piping with erosion of embankment material.. During dam breach with overtopping the unsteady flow profile is observed through the downstream channel so the unsteady flow analyses were used for dam breach simulation. Dam Breach development is simulated for different breach methods in HEC-RAS Macdonald, Frohelic(1995) and Frohelic(2008) and their breach outflow depending on the breach parameter routed through its downstream channel is and the results of the HEC-RAS model were used to map the inundation limits.

5.1. River Cross Section

This river cross section is the result of river geometry analysis done in Hec-Geo Ras and Exported to HecRas model for further work.

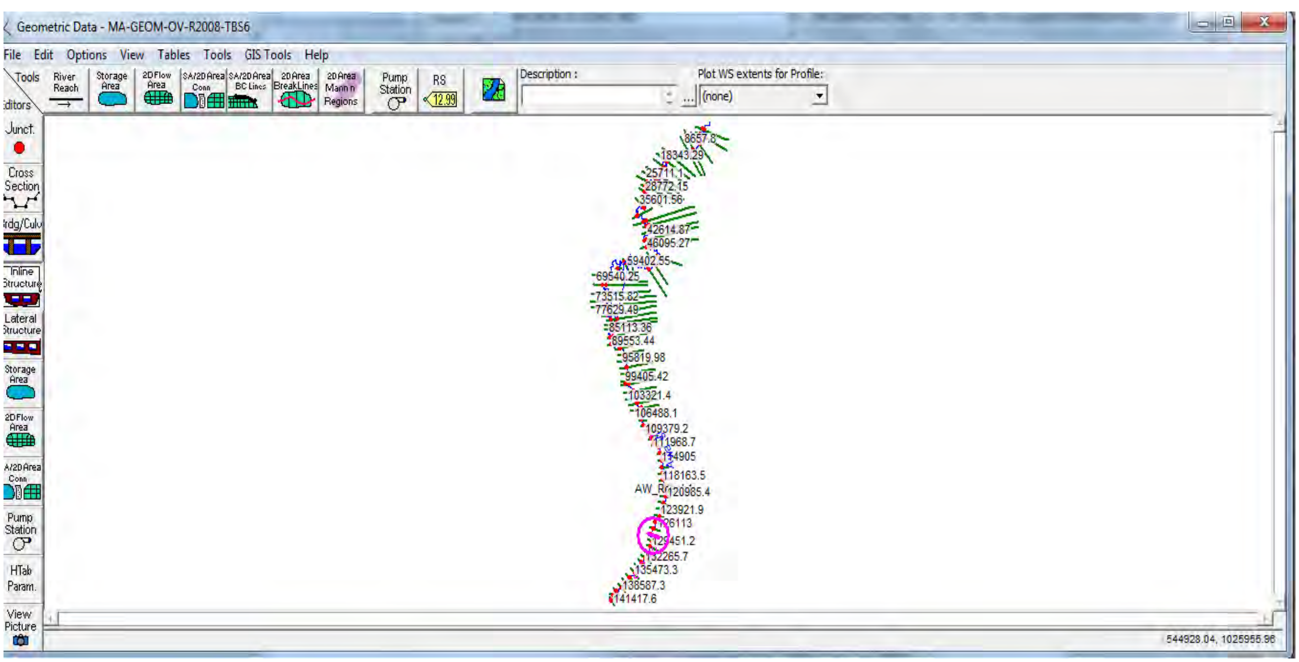


Figure 5:1 Middle Awash River cross Section exported to HecRas

5.2. Dam breach parameter estimation

HEC-RAS allows the modeling of the breach development by entering key data and assumptions regarding the dam, the reservoir and the breach characteristics. The breach parameters breach formation time, bottom width of the breach and breach side slope.

Breach parameters and related dam break peak outflow have been defined, using the three methods Macdonald, Frohelic(1995) and Frohelic(2008) done in HECRAS. The simulation for dam break model was done in Breach plan data window in which the dam characteristics and dam breach parameters are entered. (*Figure5.2*)

The Dam Breach Model is simulated for each breach method, breach parameters and mode of failure for the given dam characteristics *(Table 3.1).*

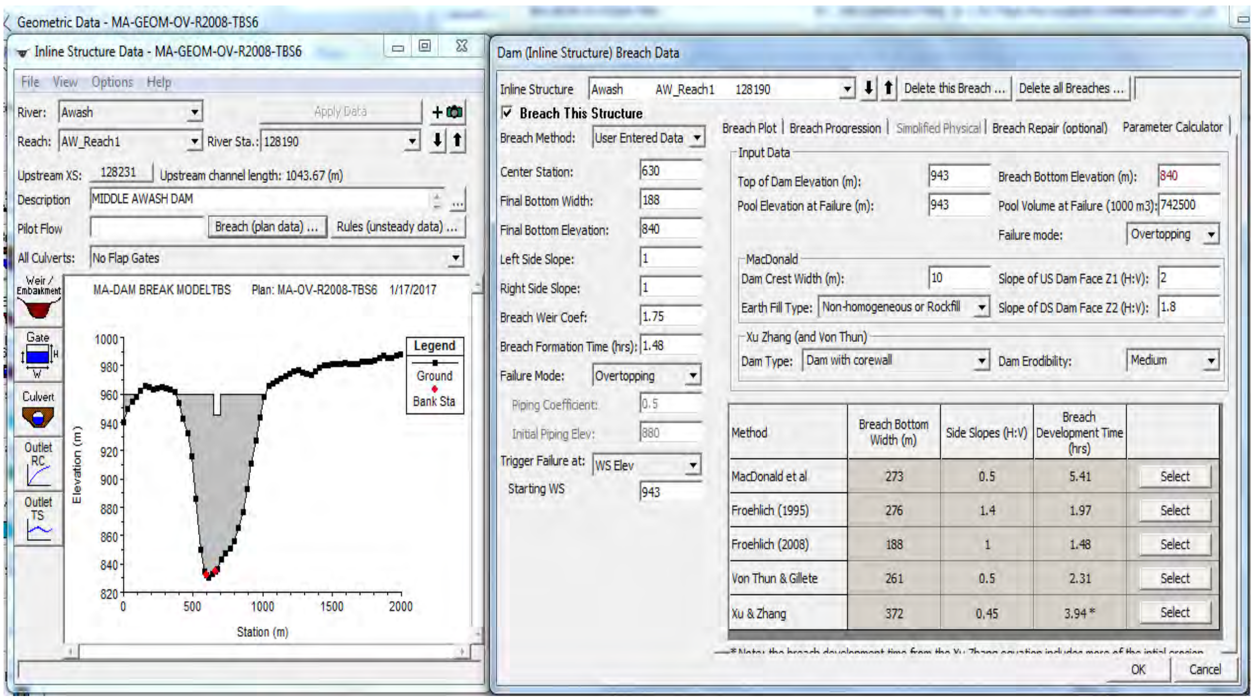


Figure 5:2 Breach Plan window

In dam break analysis first estimate the characteristics of the breach. Once the breach characteristic is estimated then HECRAS used to compute the outflow hydrograph of the breach and perform the downstream routing. Here are the breach parameters estimated in HECRAS for both overtopping piping mode of failure mode (Table 5.1&5.2).

The dam breach parameters Breach Bottom Width, Breach side slope and Breach formation time are different for each breach method during both overtopping and piping mode of failure.

Breach parameters are different for each breach method. Froehlich (1995) have a greater breach bottom width, side slope and breach time as compared with the other methods for both overtopping and piping mode of failure. The breach parameters estimated with Macdonald are equal for both overtopping and piping modes. For Froehlich (1995) and Frohelic2008) parameters with overtopping mode are greater (Table 5.1&5.2).

Table 5:1 Estimated Dam breach parameters for overtopping

Breach Parameters	Macdonald	Froehlich (1995)	Froehlich(2008)
Breach Bottom Width(m)	273	276	188
Side slope (H:1V)	0.5	1.4	1
Breach Development Time (hrs.)	5.41	1.97	1.48

Table 5:2 Estimated Dam breach parameters for Piping

Breach Parameters	Macdonald	Froehlich(1995)	Froehlich(2008)
Breach Bottom Width(m)	185	172	126
Side slope (H:1V)	0.5	0.9	0.7
Breach Development Time (hrs.)	4.79	1.60	1.22

5.3. Dam Breach Out Flow Hydrograph and Downstream Routing

The two primary tasks in the hydraulic analysis of a dam breach are the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley The flood hydrograph is a plot of discharge versus time. Flood routing is the term used to describe the movement of a flood wave as it traverses a reach of channel.

The dam breach outflow hydrograph for the Middle Awash embankment dam resulted from the HEC-RAS dam break modeling , The peak breach outflow from the hydrograph for each breach method MacDonald, Froehlich(1995) and Froehlich (2008) estimated geometries for both Overtopping and Piping modes of failure for the simulation period of 20 days to complete*(Fig 5.3-5.8).*

MacDonald: - For Mac Donald dam breach parameter peak outflow of 62888.9m3/s and 4.63Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of72688m3/s and 4.66 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

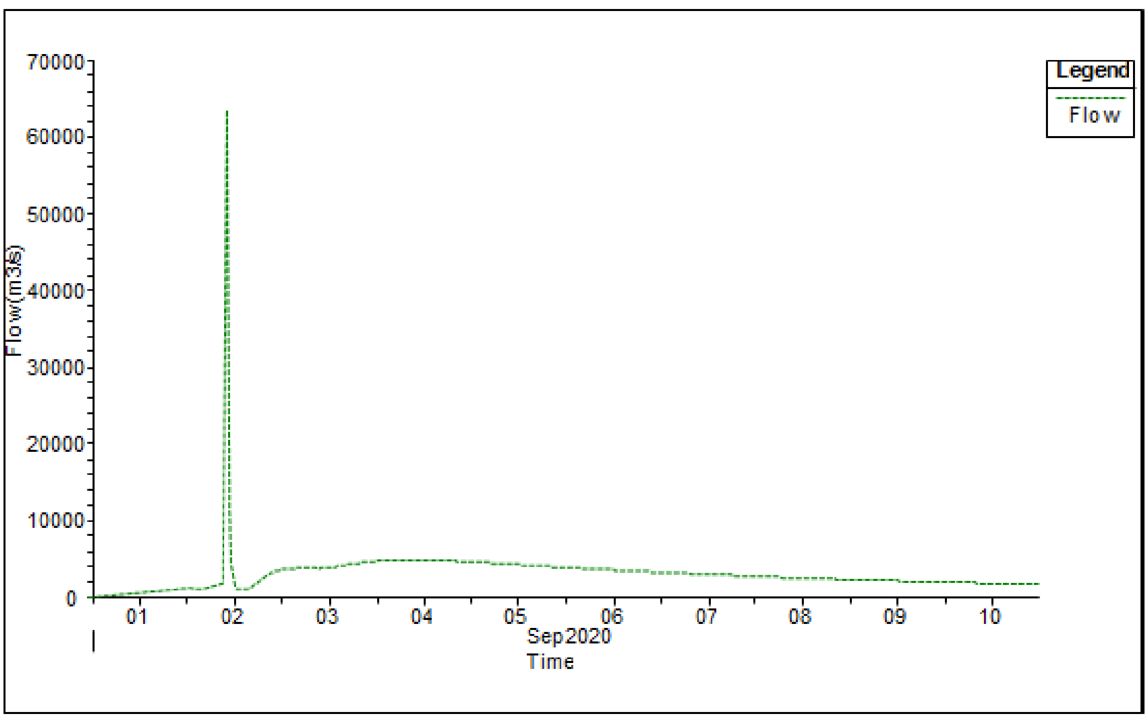


Figure 5:3-Breach Outflow hydrograph Middle Awash Dam MacDonald Dam Breach Parameters (Overtopping)

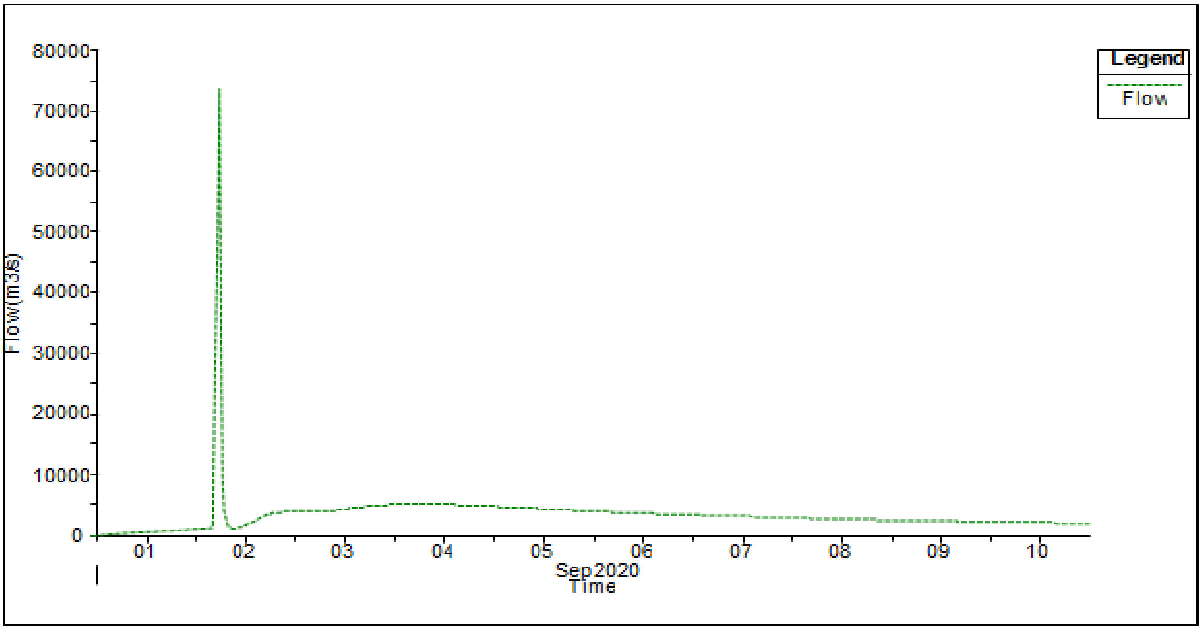


Figure 5:4 Breach outflow hydrograph Middle Awash dam MacDonald Dam breach parameters (PIPING)

Froehlich (1995): - For Froehlich (1995) dam breach parameter peak outflow of 111663m3/s and 4.74Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of 118039m3/s and 4.76 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

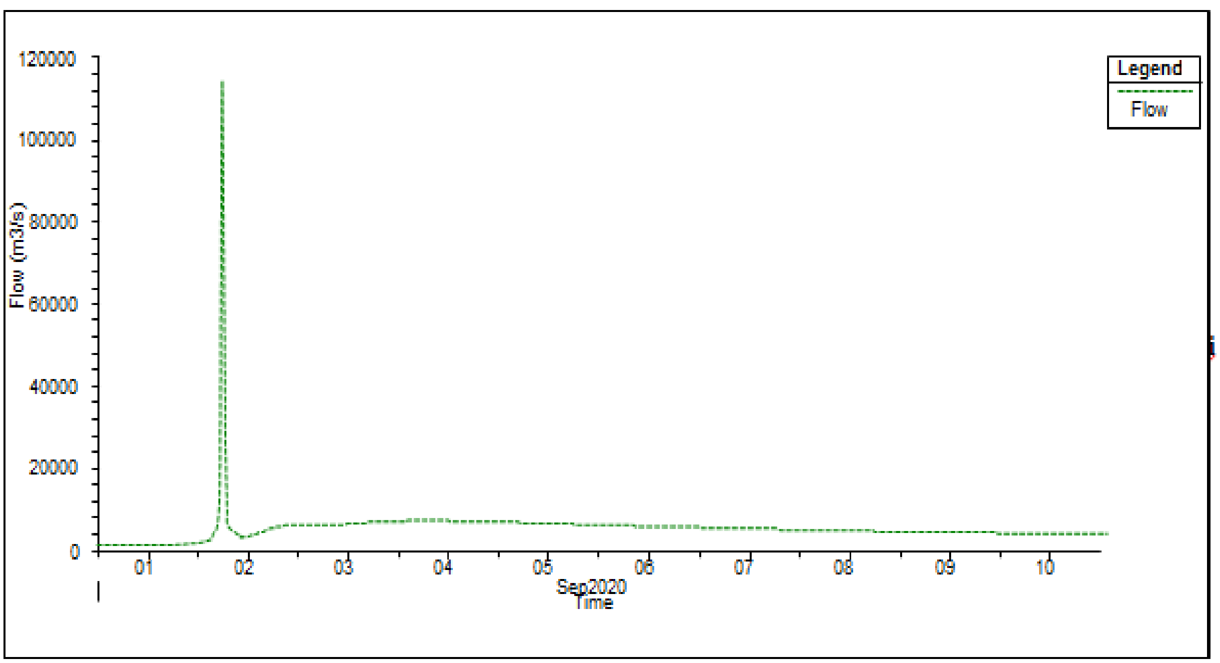


Figure 5:5-Breach Outflow hydrograph at Middle Awash Dam Froehlich (1995) Dam Breach Parameters (OverTopping)

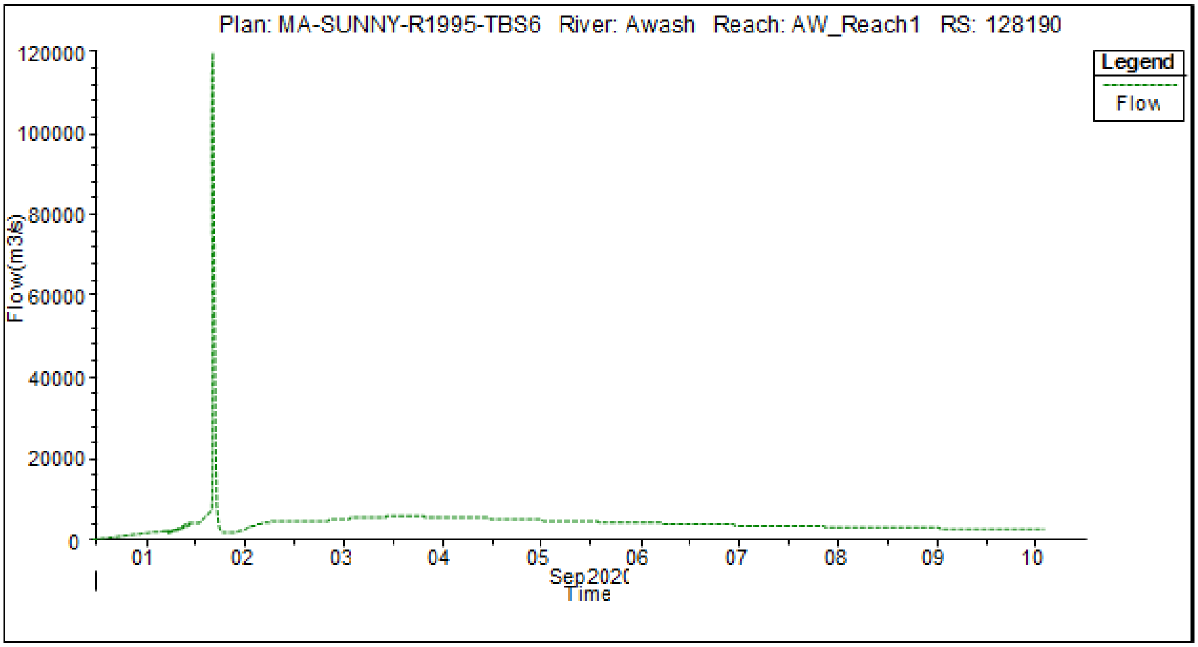


Figure 5:6-Breach Outflow hydrograph at Middle Awash Dam Froehlich (1995) Dam Breach Parameters (Pipng)

Froehlich (2008): - For Froehlich (2008) dam breach parameter peak outflow of 104814m3/s and 4.71Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of 100045m3/s and 4.7 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

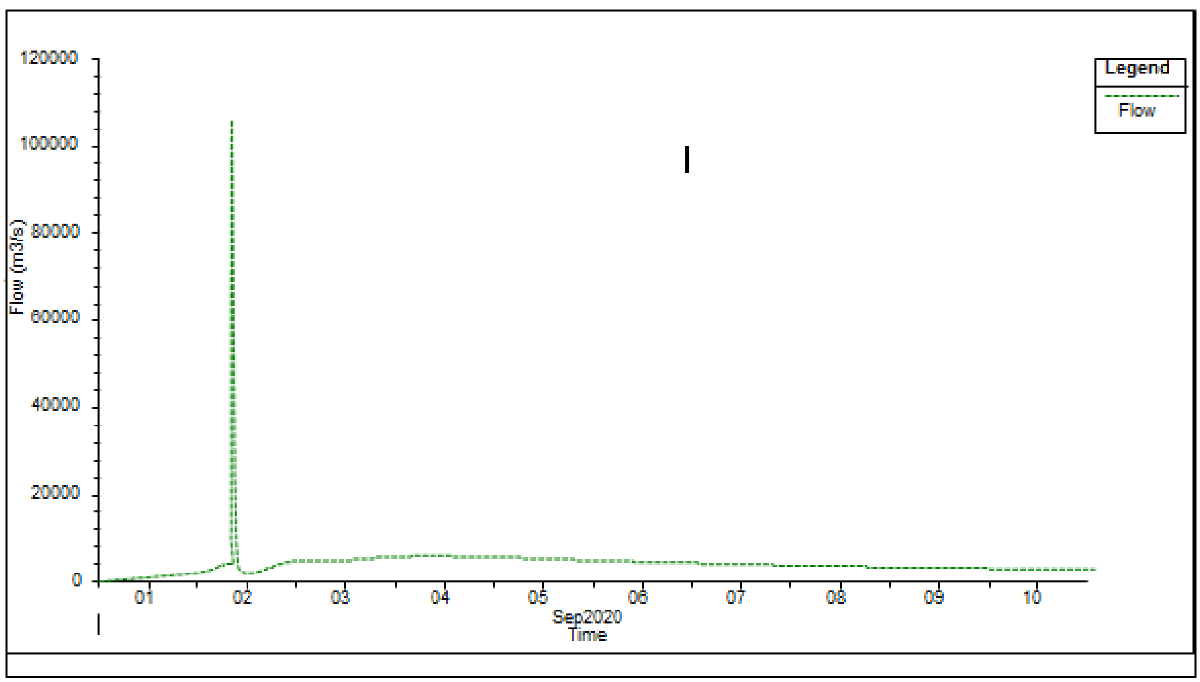


Figure 5:7-Breach Outflow hydrograph at Middle Awash Dam Froehlich (2008) Dam Breach Parameters (Overtopping)

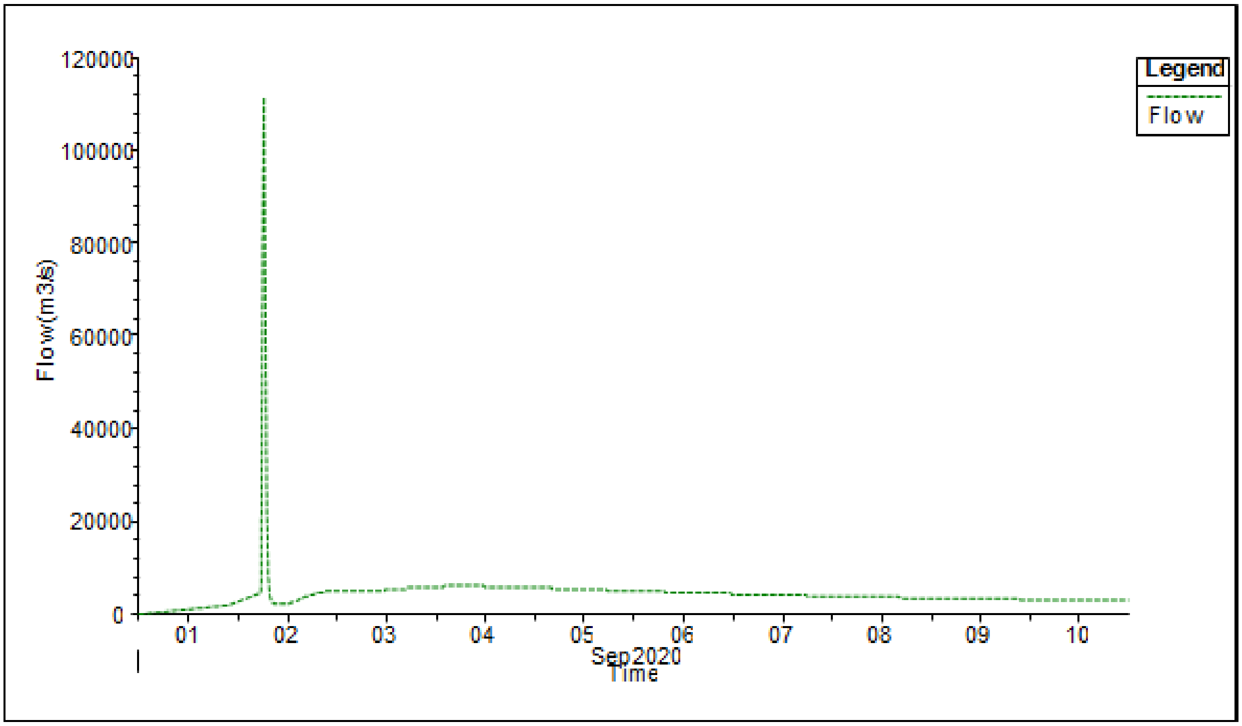


Figure 5:8-Breach Outflow hydrograph at Middle Awash Dam Froehlich (2008) Dam Breach Parameters (Piping)

The dam failure occurs for both Overtopping and Piping mode simulation under the three methods. The hydrograph is routed downstream at selected cross sections (Table 5.3) and the flow is different through downstream reach.

Table 5:3 List of Outflow value from MAMP Dam at selected towns and cross sections

	MacDonald				Froehlich(1995)				Froehlich(2008)			
	Overtopping		Piping		Overtopping		Piping		Overtopping		Piping	
	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume B(m^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)
0m from the Dam(at the dam)	62888	4.62	72688	4.65	111663	4.74	118039	4.76	104814	4.71	100045	4.7
10km from Dam	38030	4.61	46607	4.64	49259	4.64	55973	4.64	49490	4.64	25195	4.63
MelkaSedi(44km from Dam)	19141	4.6	21667	4.62	18480	4.6	21245	4.61	18414	4.6	21172	4.61
MelkaWerer(80km from Dam)	3892	4.3	3957	4.38	3888	4.3	3954	4.4	3888	4.3	3954	4.4
120 km from Dam	3830	4	3905	4.1	3827	4.0	3901	4.1	3826	4	3902	4.1
End of the XS(126km)	3825	3.98	3899	4	3821	3.97	3896	4.0	3820	3.97	3896	4.0

Overtopping flow simulation of the three breach methods for indicates that the flow for Frohelic 1995 reaches greater as going down stream but shows some little difference with frohelic 2008 *(Figure 5:9)*. Macdonald shows low flow through downstream reach as compared to others.

Piping flow simulation of the three breach methods for indicates that the flow for Frohelic 2008 reaches greater as going down stream but shows some difference with frohelic 1995*(Figure 5:10)*. The same as overtopping macdonald shows low flow through the downstream reach as compared to others.

For both overtopping and piping Overtopping Shows greater flow than piping through downstream reach for the three methods *(Figure 5:11)*.

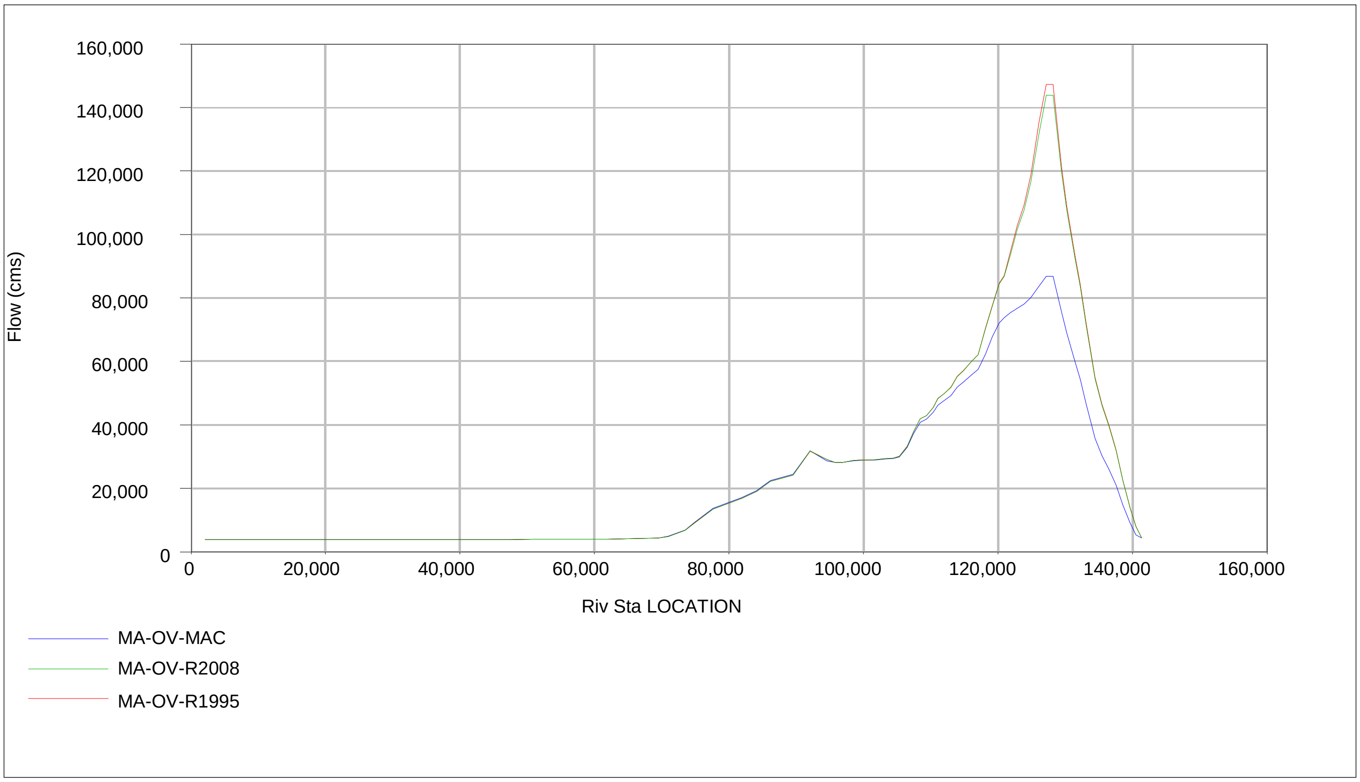


Figure 5:9 Flow of the three breach methods throughthe river station- overtopping

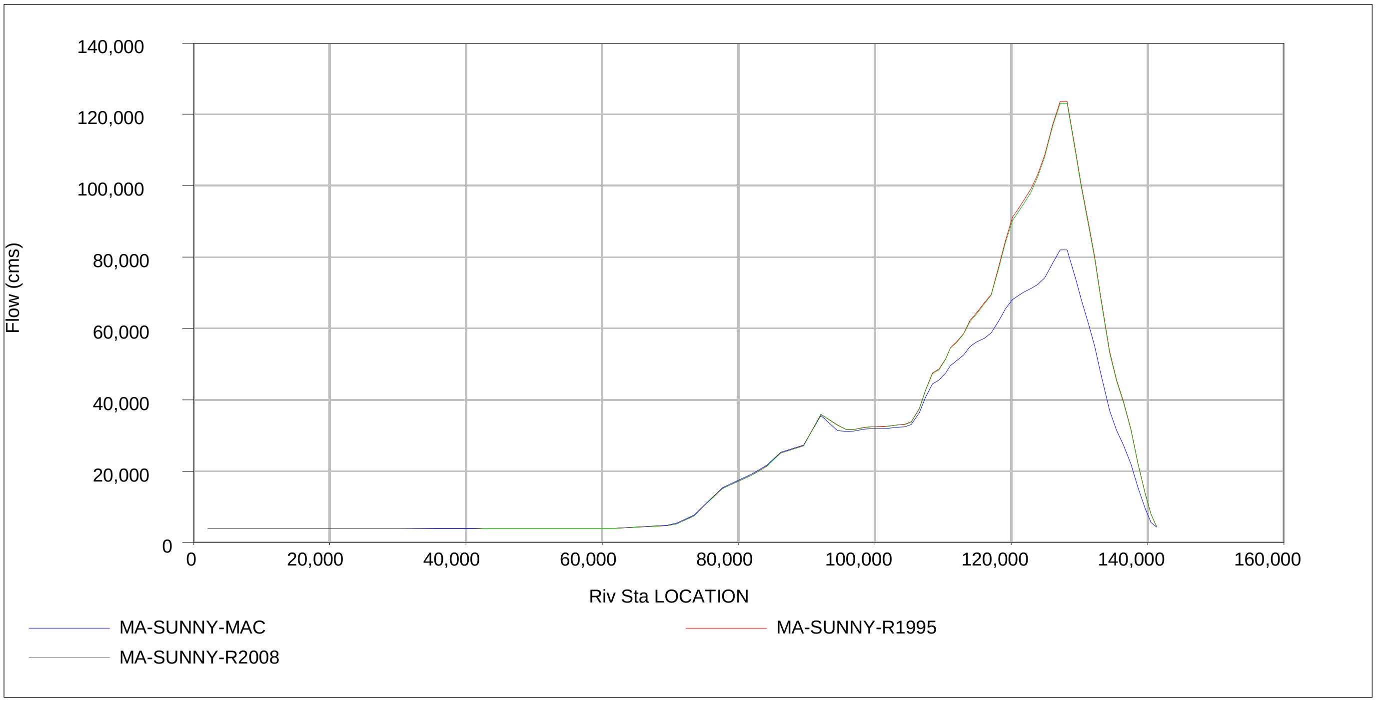


Figure 5:10 Flow of the three breach methods throughthe river station- piping

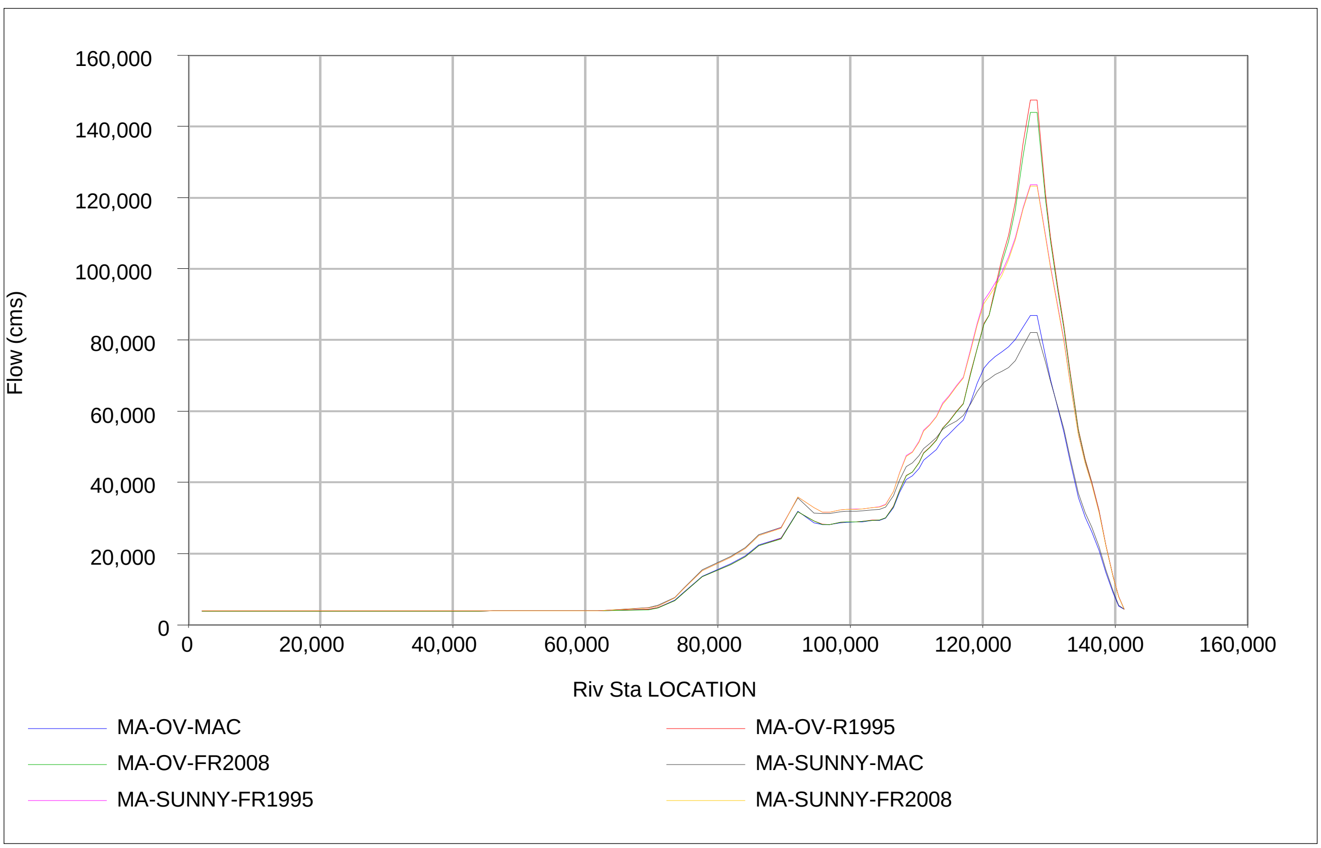


Figure 5:11 Flow of the three breach methods throughthe river station- overtopping&Piping

- After the dam failure simulation of unsteady flow routing the dam break cross section view and the water surface profile for both overtopping and piping (Figure5:12,5:13, 5:14 &5:15).

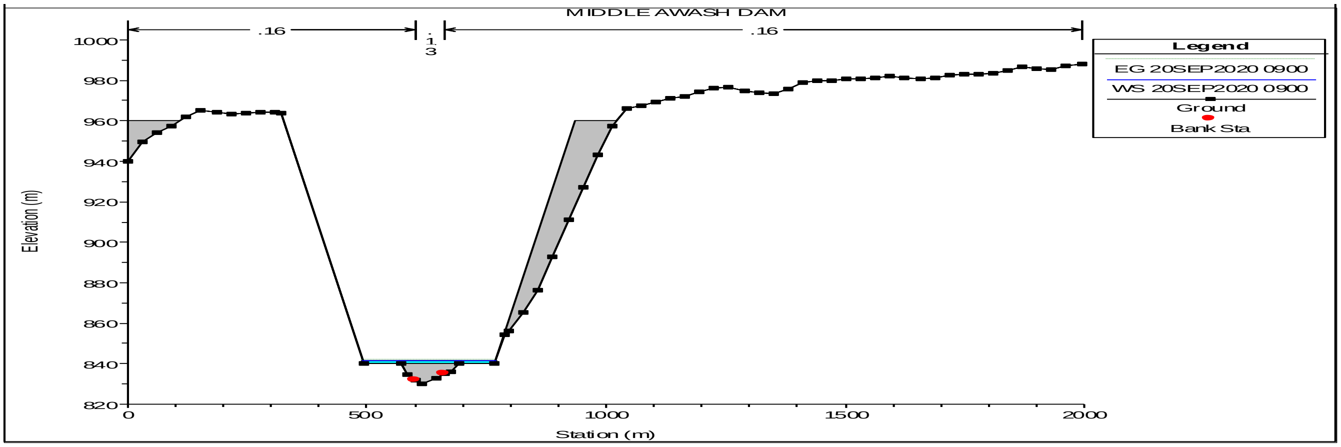


Figure 5:12 Sample Dam Break Cross Section View for Overtopping (Froheilic1995)

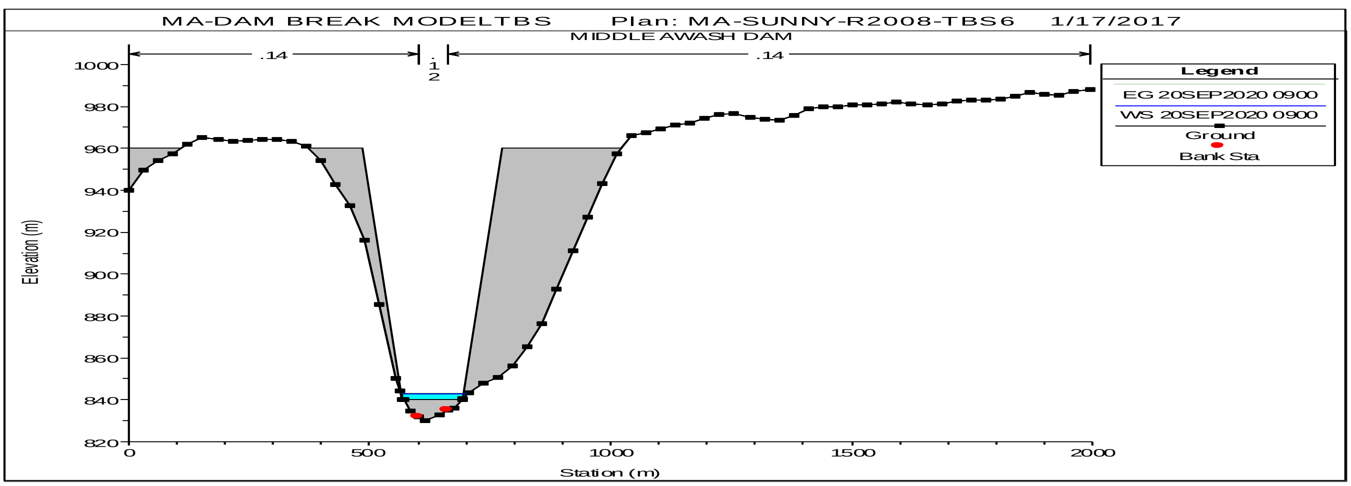


Figure 5:13 Sample Dam Break Cross Section View for Piping (Frohelic 2008)

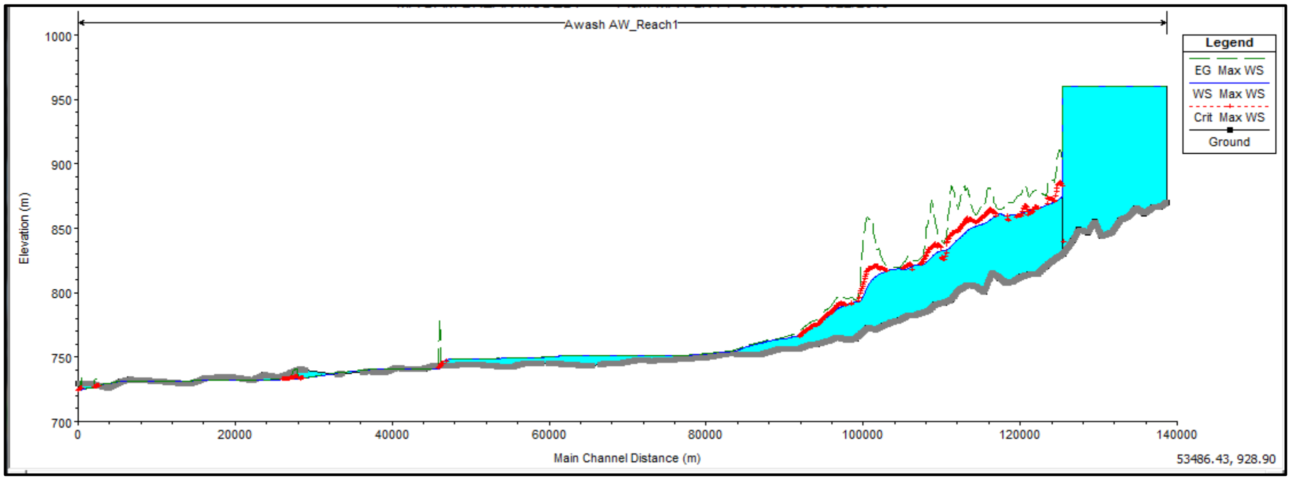
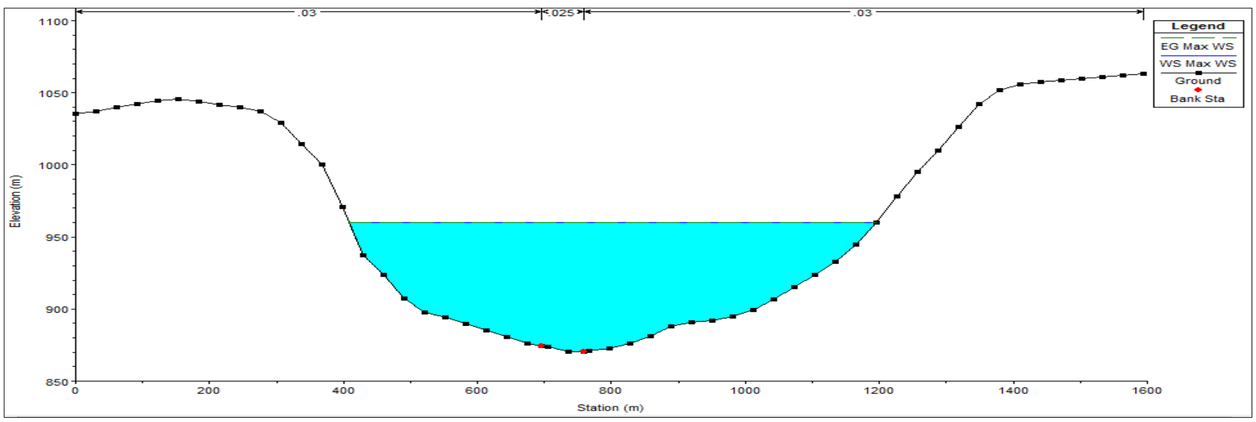
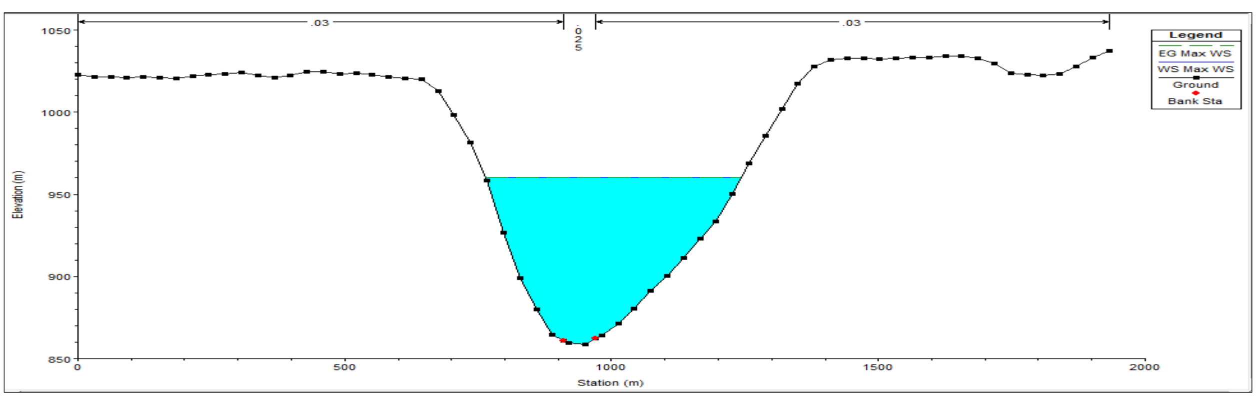


Figure 5:14 Water Surface Profile Plot

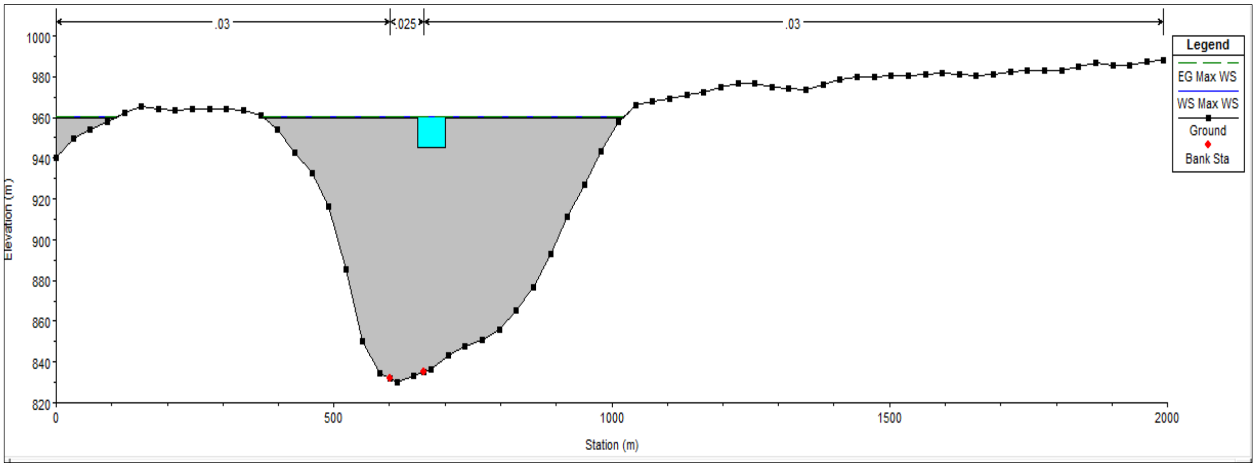
- The cross section view at selected stations:



Station=141417.6



Station=136520



Station=128190IS

Figure 5:15 Graphical Representation of river cross sections

5.4. Flood Inundation Map of Middle Awash Dam Failure

The inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It also identifies zones of high velocity flow and depth of inundation with time at representative cross-sections of the channel.

For Middle Awash Dam Break study the flood map was done using Ras Mapper in HEC-RAS and GIS. The method that has the shortest breach time arrives soon has selected for the inundation mapping as it would produce the most conservative results.

The area inundated with overtopping and piping failure is delineated. The flood depth and flood hazard map is presented. *(Fig5.16- Fig 5.21)*)

For emergency planning purposes, it is important to determine the flood arrival time, flood depth, flood hazard class, velocity, flood recession and flood duration and the time to maximum discharge in the event of a dam breach. *(Table 5.4&5.5)*

Table 5:4 Inundation Map Result Froehlich (2008) - Overtopping

Location	Depth-m	Velocity	Hazard Map(V^D)	Flood inundation Boundary	Arrival Time -day	Recession Time -day	Duration	Flow	Percent Time Inundated	WSE
0m from the Dam(at the dam)	55	1.5	450	31000ha	0	0.125	0.42	115322	0.88	919
10km from Dam	20	0.8	30		0.083	0.166	2	75624	0.65	865
MelkaSedi(44km from Dam)	11	0.15	3		0.25	8.5	13.6	19359	100	754
MelkaWerer(80km from Dam)	10.2	0.045	0.6		0.4	19	19	3891	100	746
120 km from Dam	9.8	0.042	0.52		0.5	19	19	3829	100	736
End of the XS(126km)	9.5	0.041	0.45		0.6	19	19	3821	100	735

(ANNEX 9) –OV-INUNDATION MAPS

Table 5:5 Inundation Map Result Froehlich (2008)-Piping

Location	Depth	Velocity	Hazard Map(V^D)	Food Boundary	Arrival Time	Recession Time	Duration	Flow	Percent Time Inundated	WSE
0m from the Dam(at the dam)	40	1.11	370	30840ha	0	0.125	0.125	101971	0.65	913
10km from Dam	15	0.9	46		0.04	0.166	0.125	56378	0.47	865
MelkaSedi(44km from Dam)	9	0.12	3		0.05	19	19	21616	100	754
MelkaWerer(80km from Dam)	8.6	0.04	0.4		1.08	19	19	2467	100	747
120 km from Dam	8	0.04	0.3		2.7	19	19	1657	100	737
End of the XS(126km)	6	0.04	0.2		2.9	19	19	1609	100	735

(ANNEX 9) –PPG-INUNDATION MAPS

5.4.1. Flood Inundation Boundary

Inundation boundary is the generated polygon boundary of flood extent. During Middle Awash Dam Failure with overtopping and piping the area of 31000h and 30840 ha inundated respectively. This Flood inundates towns and roads (*Figure5:16&5:17*). The failure of the dam and inundation of the downstream environment leads to the life loss, environmental damage, infrastructure damage and economic loss.

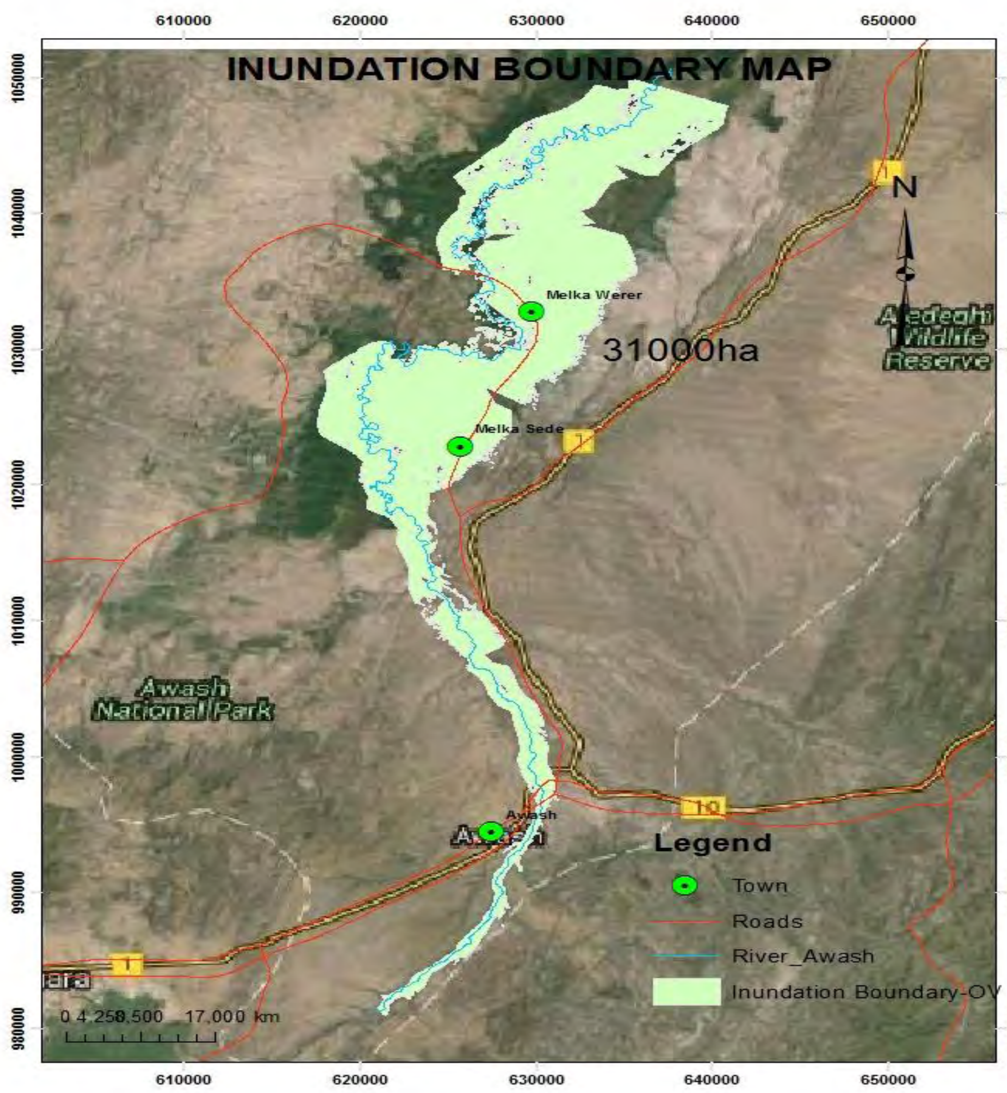


Figure 5:16 Flood Inundation Boundary Overtopping Failure

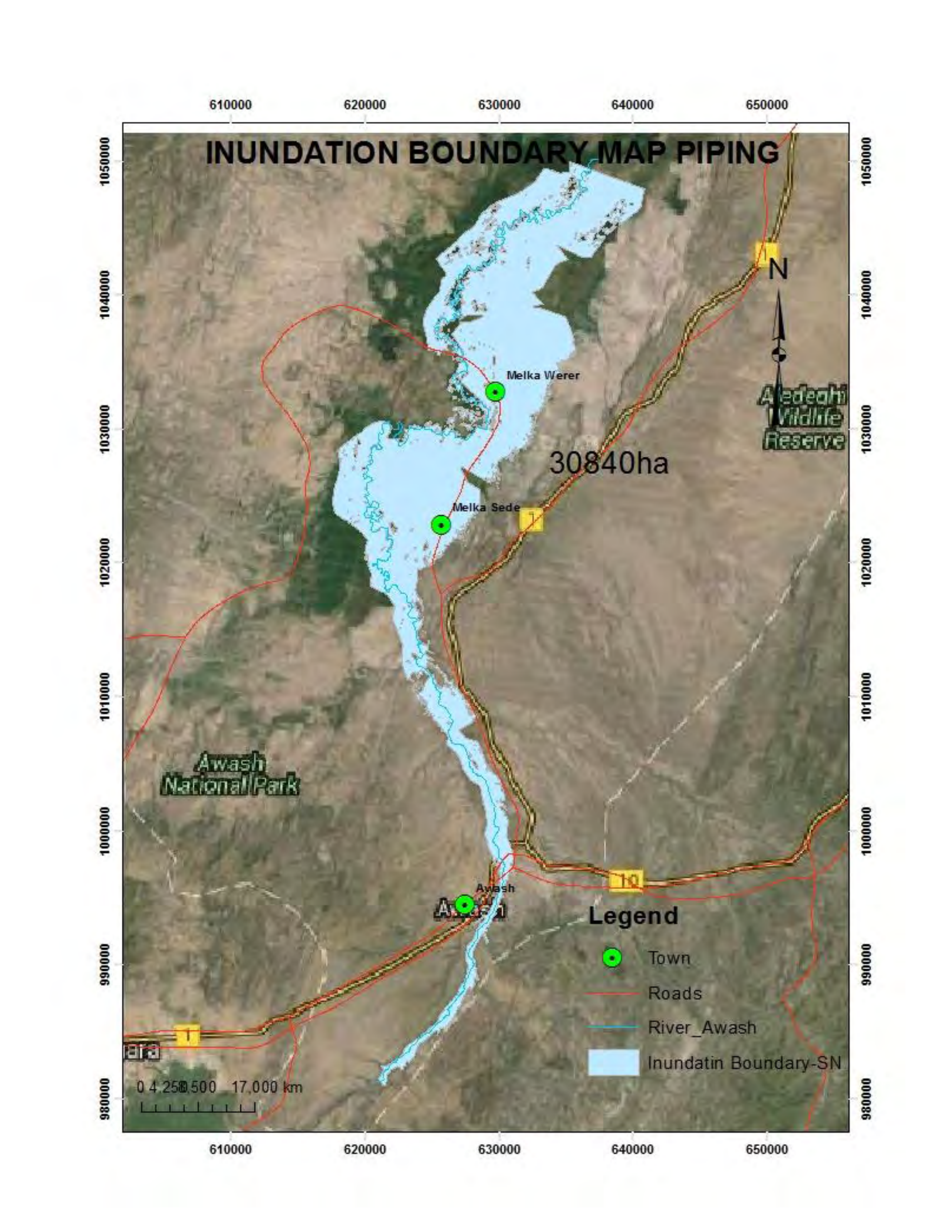


Figure 5:17 Flood Inundation Boundary Piping Failure

5.4.2. Flood depth

The depth map is computed based on the difference in water surface elevation and terrain layer based on the interpolated water surface. The depth of flood in the downstream around the towns where population, cultivation areas and infrastructures available is about 0m to 20m (*Figure 5:18&5:19) & (Table5:4&5:5)* for both overtopping and piping.

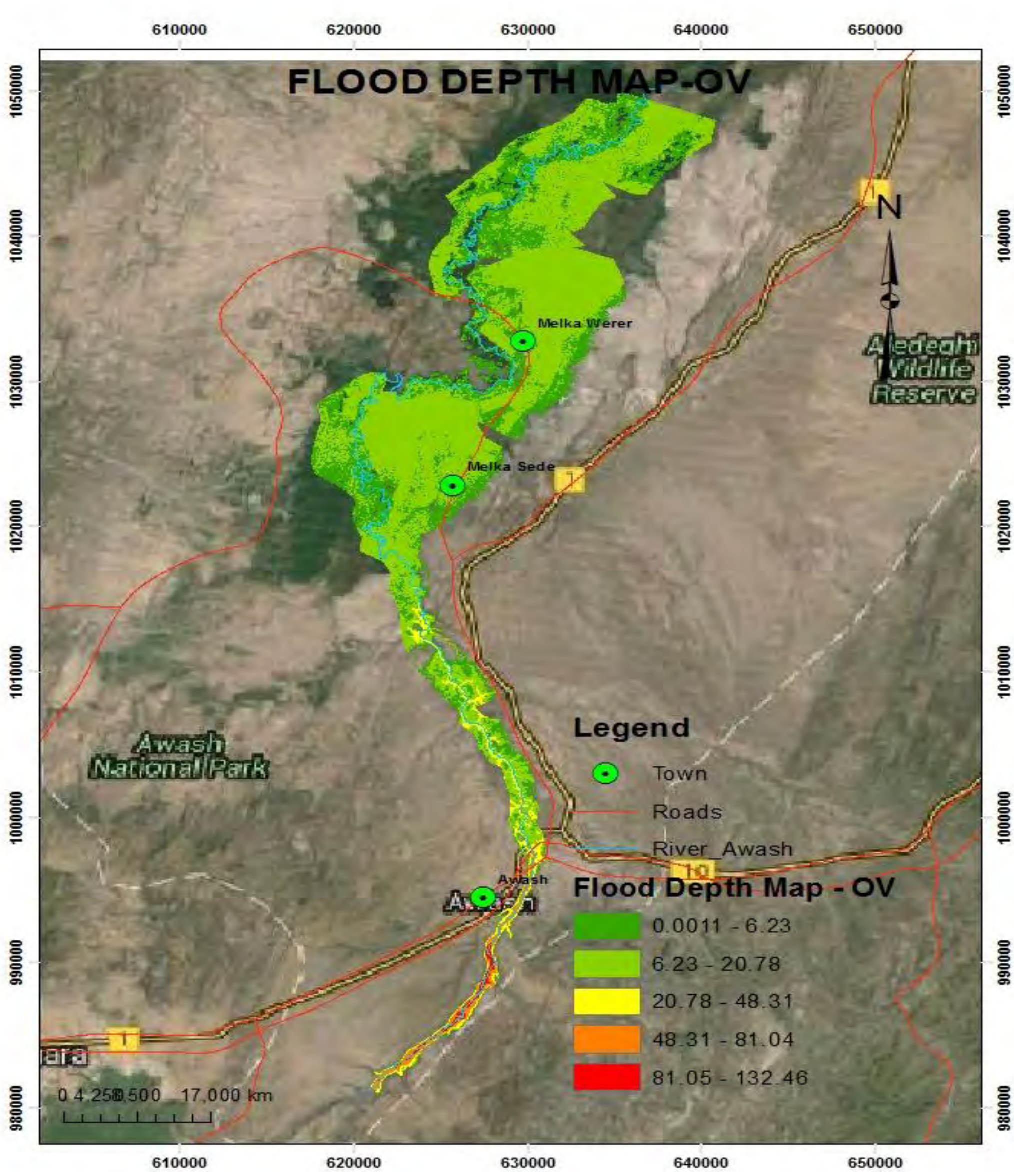


Figure 5:18 Flood Depth Map – Overtopping

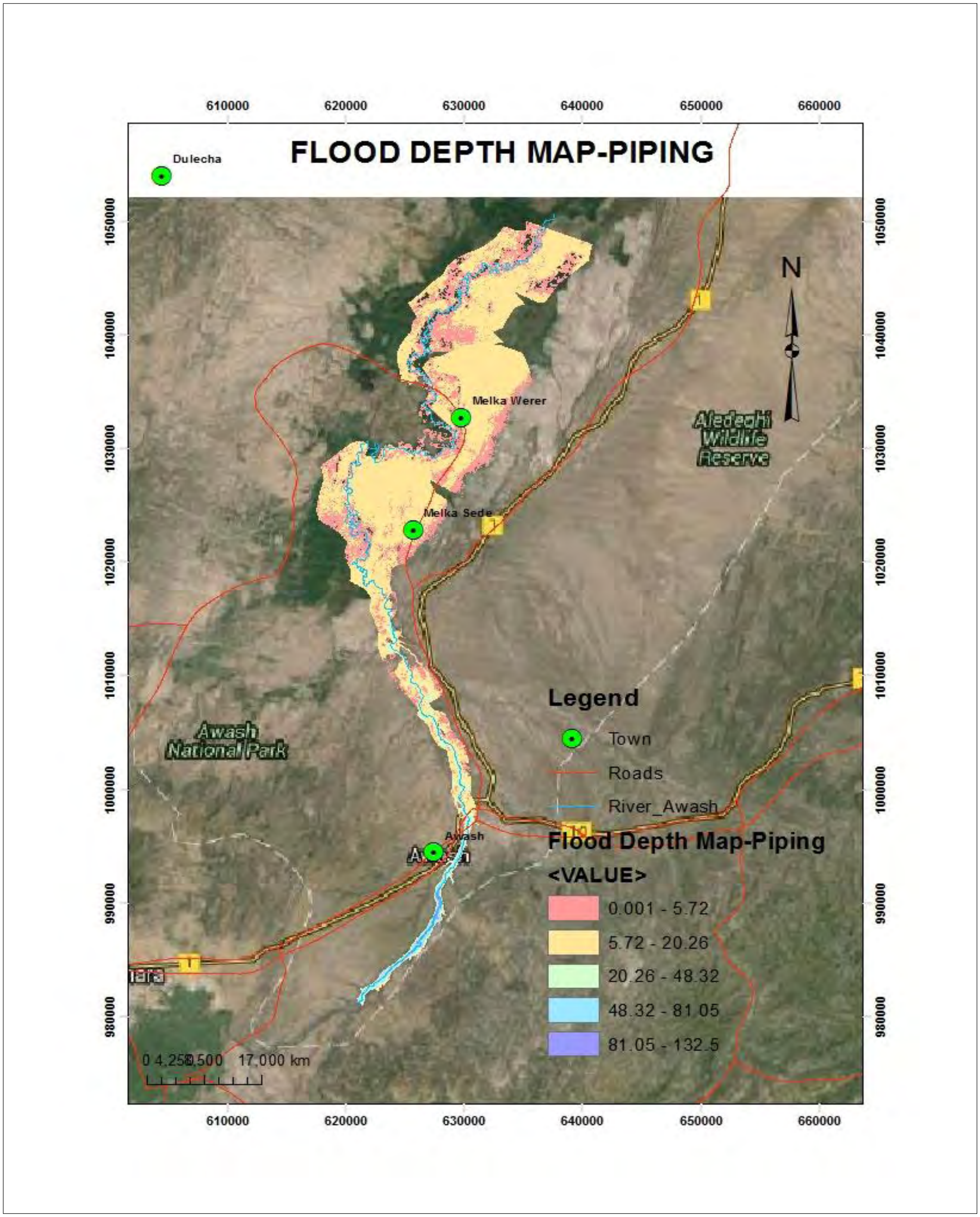


Figure 5:19 Flood Depth Map – Piping

5.4.3. Flood Hazard Map

The inundation areas were mapped as zones of hazard class based on a combination of flow depth and velocity and the overtopping is more hazardous than piping *(Fig 5.20 &5.21)*. First the failure of the dam itself is one economic loss. Because the Dam structures are constructed with high cost, the aim the dam is constructed like irrigation farms, water supply schemes and infrastructures developed following the construction of dam like newly developed industries and socio-economic establishments will be flooded with failure of the dam. In addition failure affects the towns at downstream area, farm lands, the population, animal life, environmental and the economic losses. The total population of 78,105of the downstream area located in Amibara Wereda of Afar Region (WWDSE S.-E. R., 2014) , towns at downstream Melkasedi and Melkawerer and infrastructures, Livestock Husbandry, state irrigated mechanized farm producing cotton, several crops, fruits and vegetables and vegetations shrubs and some wood land around river bank *(Fig5.22)*. The flood inundation boundary includes this area.

According to USACE Dam hazard potential classification*(Table2:6)* and flood hazard map *(Figure 5:20 &5:21)* the flood flow at these sites is classified as hazardous.

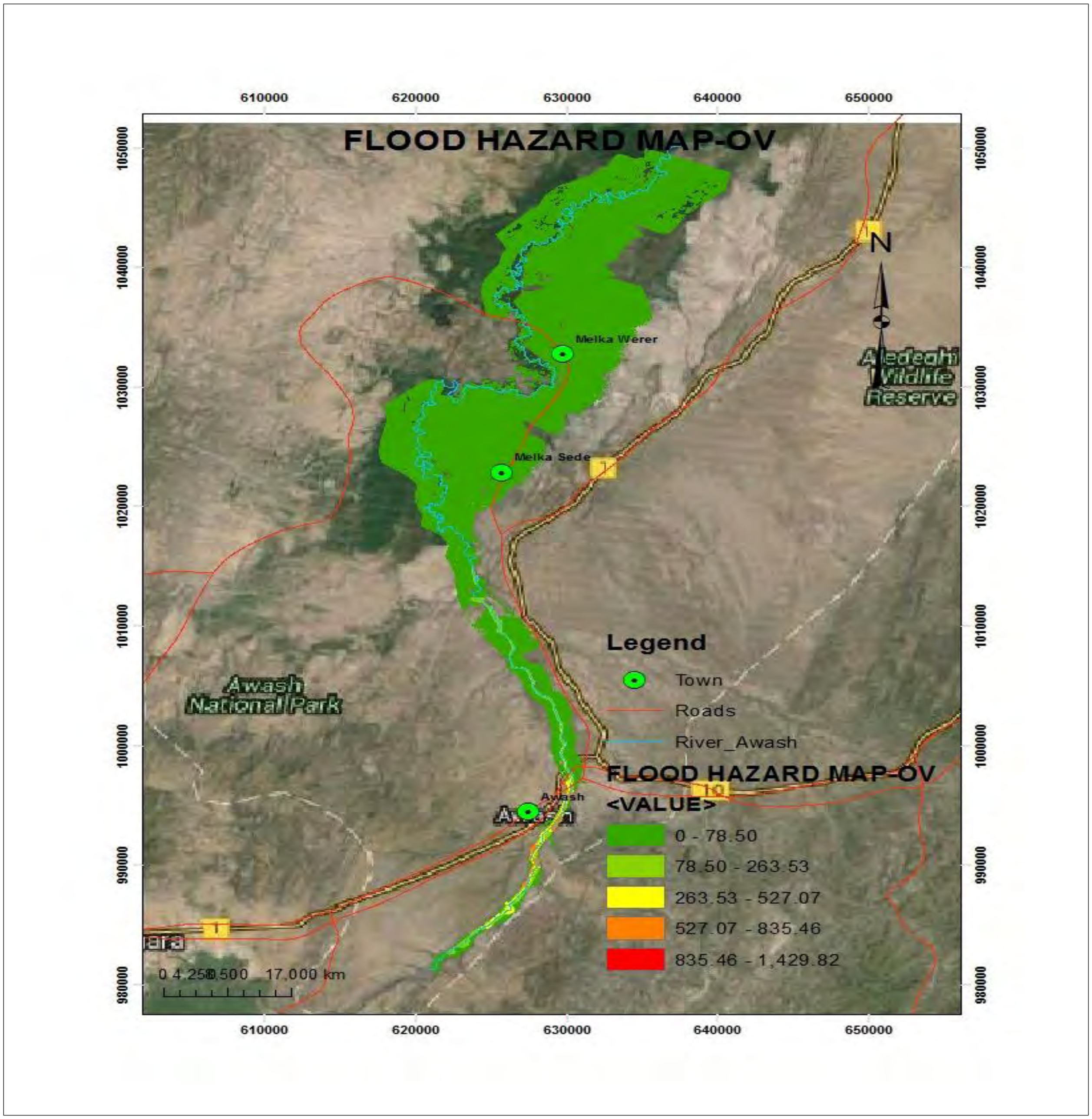


Figure 5:20 Flood Hazard Map -Overtopping

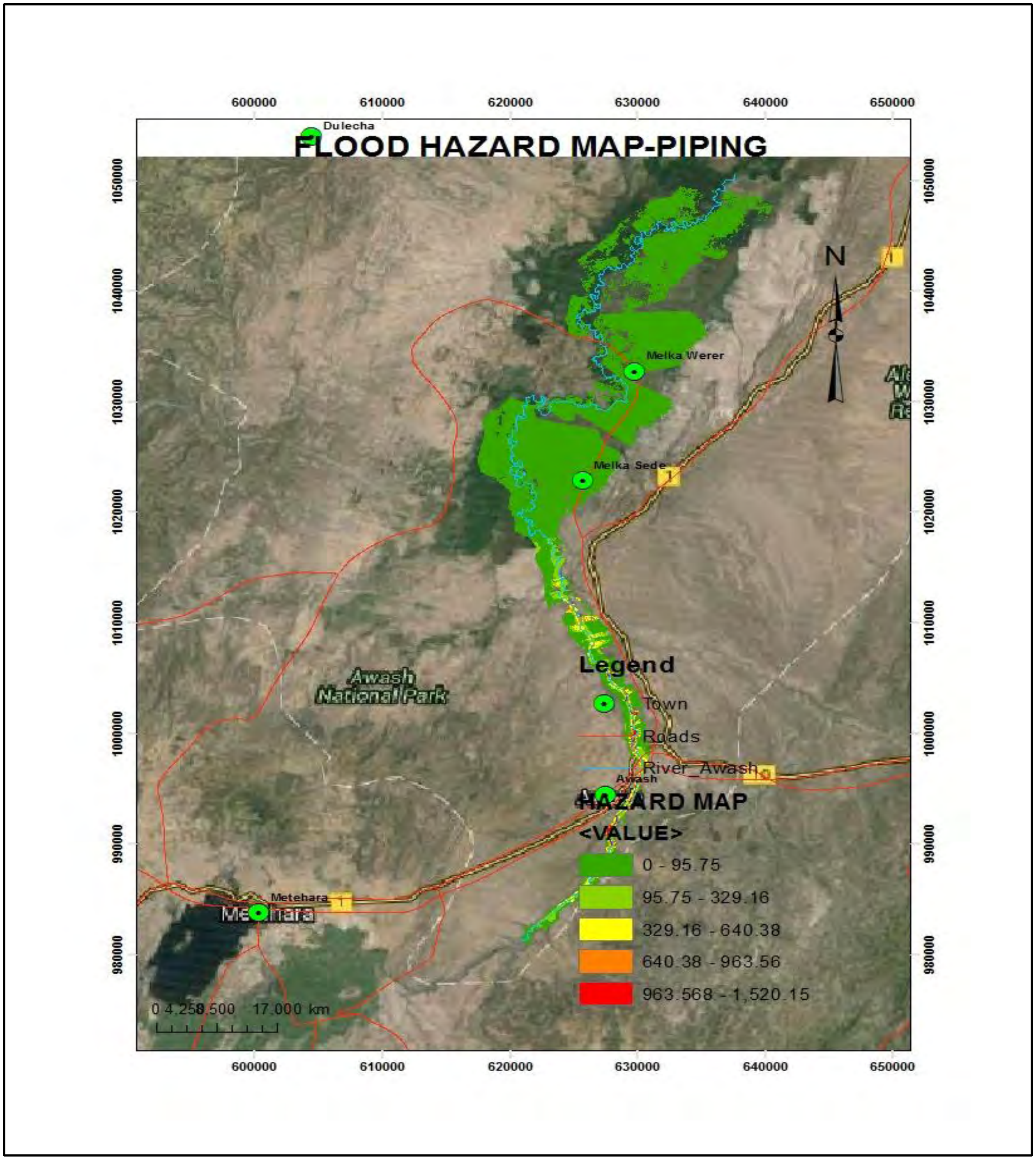


Figure 5:21 Flood Hazard Map –Piping

6. CONCLUSION

6.1. Breach Parameters, Methods and Flood hydrograph

In this thesis work, dam breach analysis for Middle Awash Multipurpose Dam, which is located in Afar region of Ethiopia, were done. Middle Awash dam failure was checked for two failure scenario using the HECRAS to model breaching of the embankment dam and the inundation extent during piping(sunny day) failure mode at normal pool level and hydrologically induced failure with PMF.

Piping occurs by erosion of fine sediments out of the soil matrix of the embankment dam starting from the dam face or downstream toe and its opening expands upward.The model is simulated at the reservoir normal pool elvation and initial piping elevation of 880m. The resulted piping breach outflow is 100045m3/s with the volume 4.7B m3. As the dam is rockfill embankment dam this failure can occur if impervious core material used is poor in quality and construction procedure due to the succeptibility of the dam and foundation material to concentrated seepage.

PMF was checked if it can overtop the dam and cause any threat. The model revealed at peak event the PMF can overtop the dam at full reservoir conditionis. The resulting breach outflow is 104814m3/s with the volume of 4.72B m3.

For the outflow hydrograph resulted from the breaching dam unsteady flow routing through the downstream river channel and flood plain was done and different maps of the resulting water surface extent, water depth and hazard map are produced.

The failure phenomena is unexpected and an abrupt. Hence, evacuating the downstream community after the failure process has already commenced may not be possible to save lives and property due to the aggressive nature of the wave front.

This research contribute some useful information for Middle Awash Dam to minimize these catastrophes of the dam failure and for information regarding dam safety issues that should be considered and precautions to bear in mind while implementing infrastructures on downstream areas and input data in preparing Emergency Action Planning. Moreover,designers, consultants and contractors engaged on dam works may gain an input data for their work and forecasting the possible dam break flood to develop disaster management plans such as constructing flood protection dykes, structuring evacuation techniques, transmitting inundation information and setting early warning systems.

6.2. Flood Inundation map

Estimating the inundation begins with an estimate of the flood hydrograph; how much water pours through the breach and how fast it pours through. Middle Awash Dam flood inundation maps were generated using RASMAPPER and ArcGIS.

Flood Map done for the area inundated with breach outflow of Middle Awash Dam with both overtopping and piping dam failure for selected sites at downstream.

Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at a given location in the event of a dam breach.

Information how much warning time will there be before the flood helsp in estimating failure consequences.

In addition, information of the flood warning time, time to peak, peak elevation, and peak discharges, flood depth , flow, velocity, arrival time, duration and recession time, WSE and hazard were depicted on the inundation maps at key locations, which is important for the purpose in providing information for the society awareness and preparedness for emergency action plan.

The dam break analysis helps in order to determine the failure consequence of the dam: the population at risk, the estimated loss of life, the cultural and environmental consequences, the economic losses and the impact on infrastructure considered which helps in emergency plan purpose

The area 31000ha and 30840ha are inundated with the overtopping and piping failure of Middle Awash dam.

The distance downstream form the dam 10km, 44km (MelkaSedi town), 80km(MelkaWerer), 120km and 126km.

- Inundated area is higher for-Overtopping.
- Flood hazard class is higher at the dam and 10km downstream from the dam site

The towns at downstream Melkasedi and Melkawerer and the farms are affected. The flood flow at these sites is classified as hazardous.

So the failure of this dam hazardous. It damages the towns at downstream area, farm lands, the population at risk, loss of animal life, and environmental damage and the economic losses..

7. RECOMMENDATION

- For the dam at the planning stage carefull selection of construction material and following the proper engineering standard procedure for design and construction is recommended to minimize the risk of failure of embankment dam with piping due to erosion of materials and overtopping.
- The breach parameter and breach method affects the result of breach out flow and flood inundation area during modeling these should be selected carefully..
- The dam breach modeling results inform the extents of flooding. There should be accurate estimation of the severity and extent of dam break flood prior to the construction of a dam Therefore, damages that could occur in the surrounding settlements, agricultural areas, on both lives and infrastructure can be minimized and even controlled.
- The maps used for many land use planning purposes considering the dam failure consequence flood warning, , hazard classification and updating the classification for a gap of year for existing dams, for dams under planning stage to identify flood prone areas,
- Also helps for Preparedness and emergency action planning related to dam failure by alert concerned government bodies to take a precaution on dam safety plans, prepare an early flood warning system, timely aware downstream inhabitants about the hazardous flood and plan a proper emergency evacuation method on disaster time

8. BIBLIOGRAPHY

Anon., n.d.

Behailu, S., 2004. *Stream flow simulation for UpperAwash Basin*. Addis Ababa University.

Brunner, G.W., 2002. *HEC-RAS, River Analysis System Hydraulic Reference Manual*. Davis: US ARMY CORPS OF ENGINEERS.

Brunner, G.W., 2006. *HEC-RAS, River Analysis Systme User's Manual*. Davis: US ARMY CORPS OF ENGINEERS HYDROLOGIC ENGINEERING CENTER (HEC).

Cameron T. Ackerman, P.E.a.G.W.B.P.E., n.d. Dam Failure Analysis Using HEC-RAS and HEC-GeoRAS.

Cameroon T. Ackreman, P.E.a.G.W.B.P.E., n.d. *Dam Failure Analysis UsingHec-Ras and Hec-GeoRas*. Davis,CA 95616: Hydrologic Engineering Center.

CDA, 2013. *Dam Safety Guidelines*. Canadian Dam Association.

Colorado, D.S.B., 2010. Guidelines for Dam Breach Analysis. In *State of Colorado Department of Natural Resources Division of Water Resources*. State of Colorado: Colorado Division of Water Resources. p.68.

Damsafety, G., 2007. Dam Break Inundation Analysis and Downsream Hazard Classification. Washington: MGS Engineering Consultants. p.34.

FEMA, 2004. *Hazard Potential Classification system for Dams*. Federal Guidelines for Dam Saftey.

FEMA, 2013. *Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures*. Federal Guidelines.

FEMA, July 2013. *Federal Guidelines for Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures*.

Fread, D.L., 1988. *An Erosion Model for Earthen Dam Failures*. Silver Spring, Maryland: National Weather Service, National Oceanic and Atmospheric Administration.

Gee, M., 2010. Use of Breach Processes Models to Estimate HEC-RAS Dam Breach Parameters. In *2nd Joint Federal Interagency Conference, Las Vegas, NV. June 27-July 1, 2010.*, 2010.

Goodell, C., 2006. *Flood Inundation Mapping Using Hec-Ras*. Obras Y Proyectos.

Guner, B.&., 2001. Pre-Event Dam Failure Analysis for Emergency Management. *Turk J Engin Environ Sci*.

Halcro-, 1989. Addis Ababa.

ICODS, 1983. *Dam Safety Emergency Action Planning Guidelines*. Integracy Committee on Dam Safety.

ICOLD, C.O.H.F.D., 1998. *DAM-BREAK FLOOD ANAYSIS*. Paris: International Committee on Large Dams.

Litani River Basin, M.S.P., January, 2012. *Dam Break Modeling for Qaraoun Dam Main report*.

MacDonald, T.C.a.L.-M., 1984. Breaching Characterstics of Dam Failures. *Journal of Hydraulic Engineering*.

Pistocchi, A..M.P., n.d. Use of HEC-RAS and HEC-HMS models with ArcView.

Ravindra, 1.S.L.2., , 2012. DYNAMIC FLOOD ROUTING AND UNSTEADY FLOW. *International Journal of Advanced Engineering Technology*.

Reclamation, B.o., 1988. *Downstream Hazard Classification Guidelines*. Denver,Colorado: Assistant Commissioner-Engineering and Research.

RIVER, L., LITANI.

Singh, V.a.S.P., 1998. *American Society of Civil Engineers*. [Online] Available at: http://dx.doi.org/10.1061/(ASCE)0733-9429(1988)114:1(21) [Accessed 27 June 2014].

Tariku, 2015. *Dam Break analysis and Risk assessment*. Addis Ababa.

Tony L, W., 1998. *Predicting Embankment Dam Breach Parameters*. Research Report. US: Dam Safety Office water resource research laboratory.

USACE, 2010. HECRAS REFERENCE MANUAL.

USACE, 2012. *Hecgeo Ras Manual*. USACE Hydrologic Engineering Center.

USACE, 2016. *HECRAS REFERENCE MANUAL*.

VanRijn, n.d. *Principles of Sediment Transport in Rivers, Estuaries and Coastal Seas*. Delft Hydraulics.

Wahl, T.L., 1997. *Prediction of Embankment Dam Breach Parmeters*. U.S. Department of the Interior Bureauof Reclamation.

Wahl, T.L., 2004. Uncertainty of Predictions of Embankment Dam. *JOURNAL OF HYDRAULIC ENGINEERING © ASCE*, pp.389-97.

Wahl, T.L., 2010. Dam Breach Modelling. In *2nd Joint Federal Integracy Conference on Sedimentation and Hydrologic Modeling*. Las Vegas, 2010. Hydraulic Engineering, Bureau of Reclamation, Denver, CO,twahl@usbr.gov.

Wermore, J.N.a.D.L.F., 1984. The NWS Simplified Dam Break Flood Forecasting Model for Desk-Top and Hand-Held Microcomputers. Federal Emergency Management Agency. p.122.

WIKIPEDIA, n.d. *WIKIPEDIA*.

Wurbs, R.A., 1987. Dam Breach Flood Wave Models. *Journal of Hydraulic Engineering*.

WWDSE, 2005. *Final Dam Design Report*. Addis Ababa.

WWDSE, S.-E.R., 2014. *Middle Awash Socio economic study feasibilty report*.

WWDSE, 2015. *Design Report*. Addis Ababa.

WWDSE, 2015. *Hydrologic Analysis*. Addis Ababa.

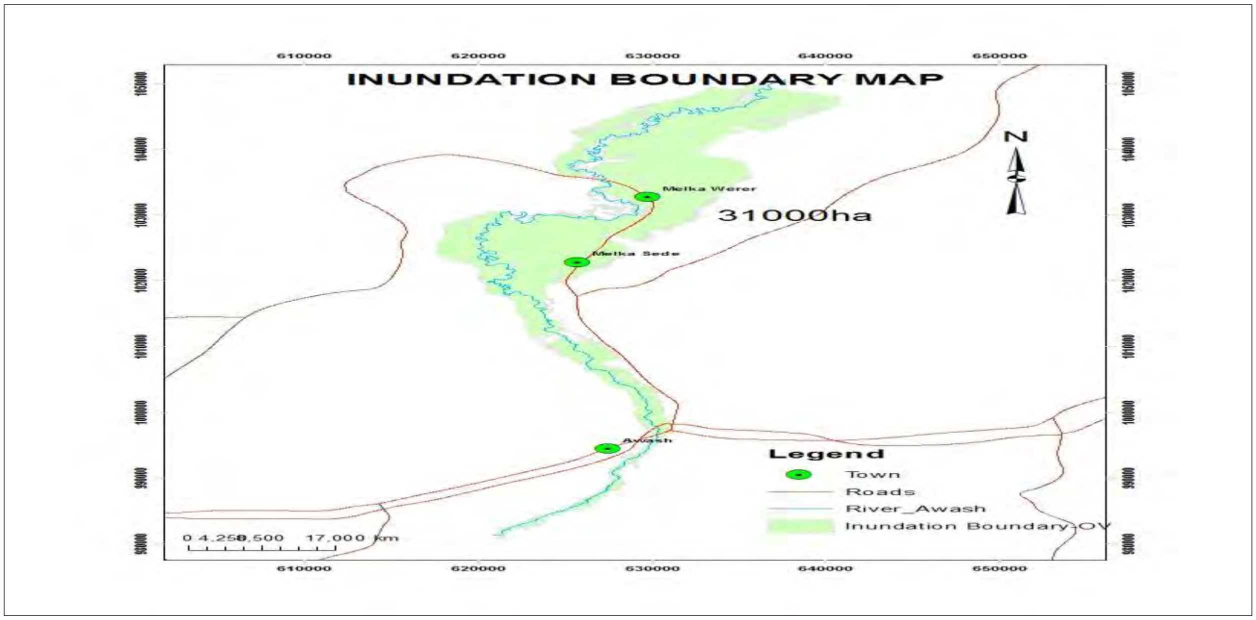
ZAGONJOLLI, M., 2007. *Dam break Modelling, Risk Assessment and Uncertainity Analysis for Flood Mitigation*. London, UK: Taylor & Francis.

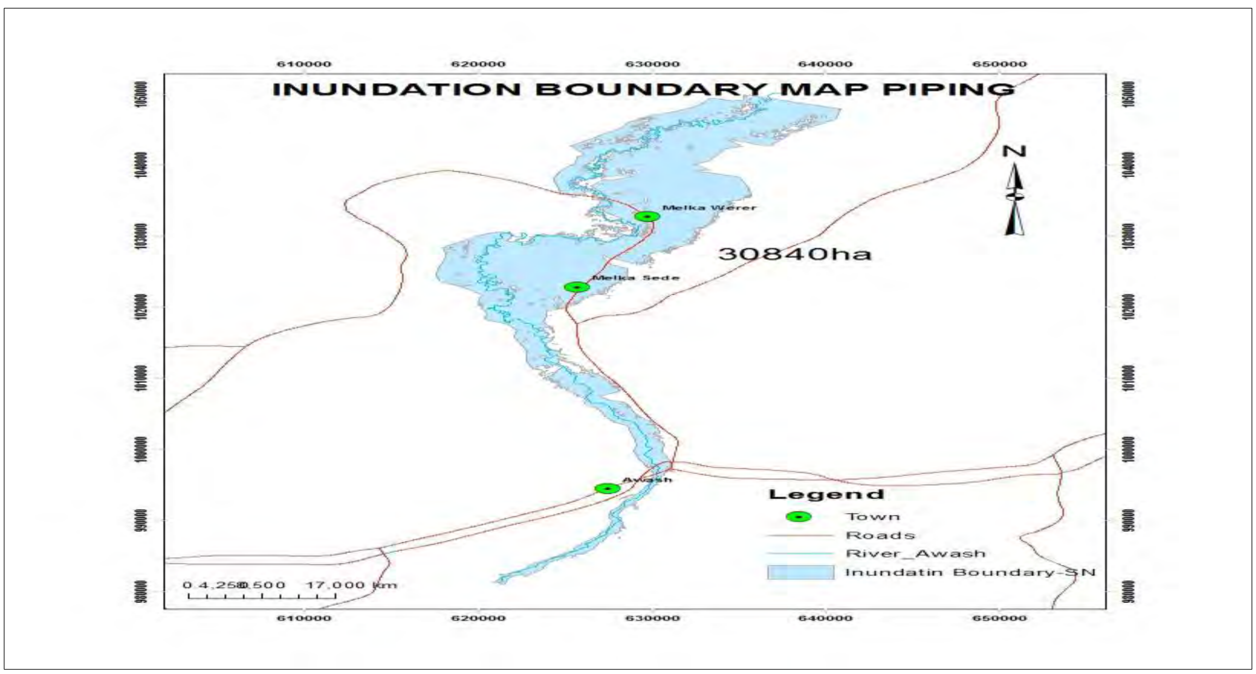
ZAGONJOLLI, M., 2007. *Dam break Modelling, Risk Assessment and Uncertainity Analysis for Flood Mitigation*. London, UK: Taylor & Francis.

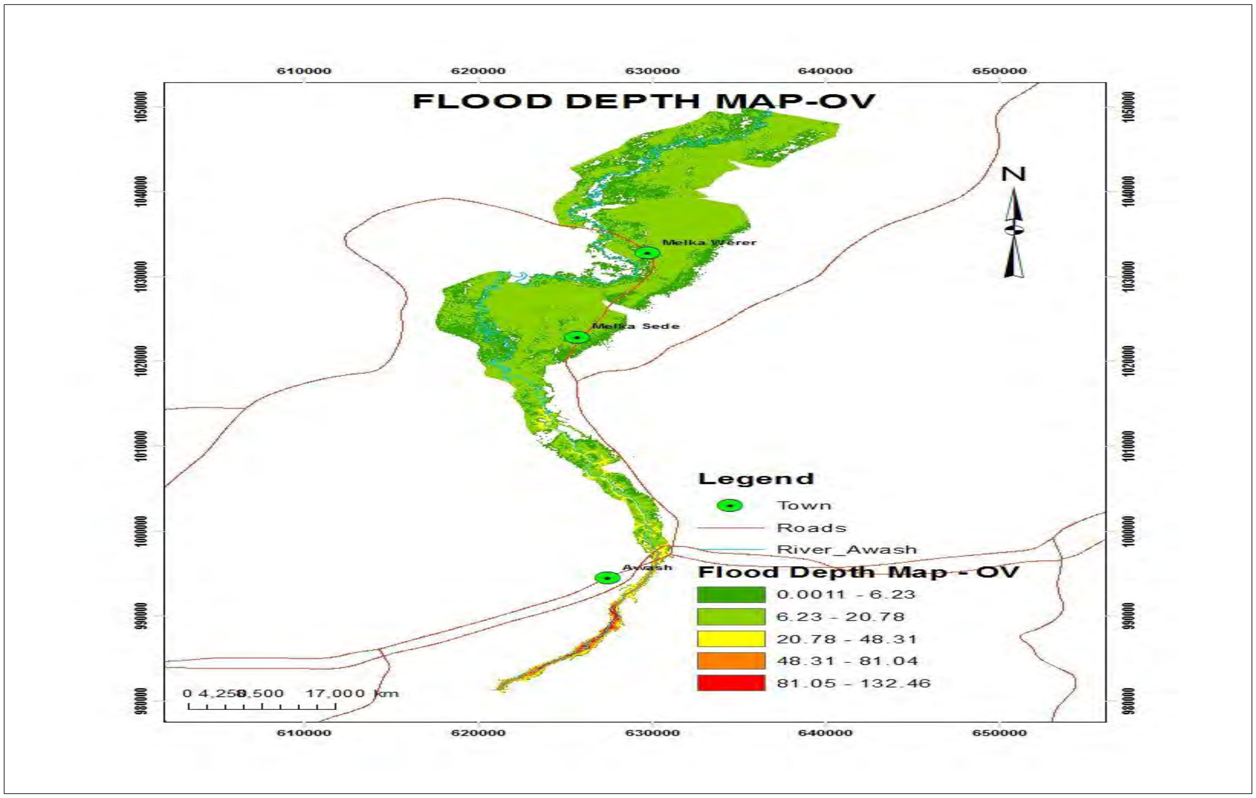
9. ANNEX

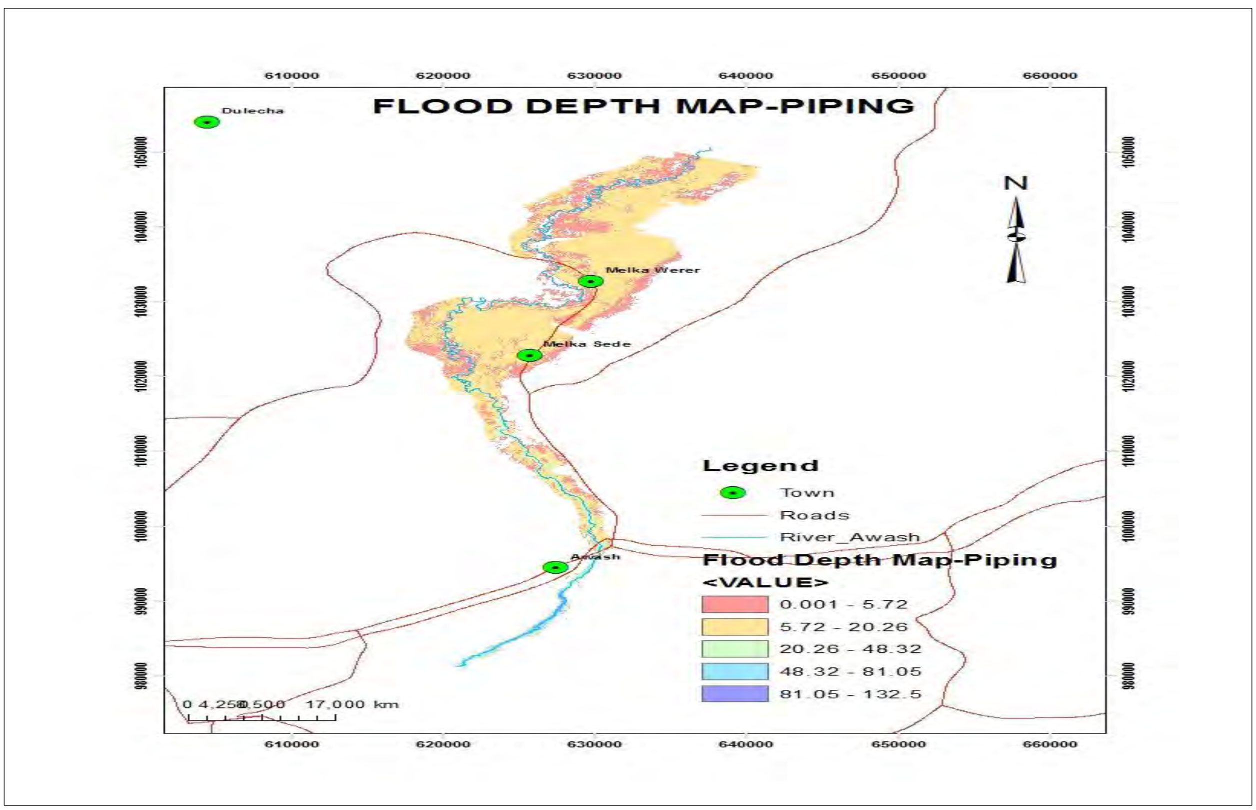
9.1. INUNDATION MAPS

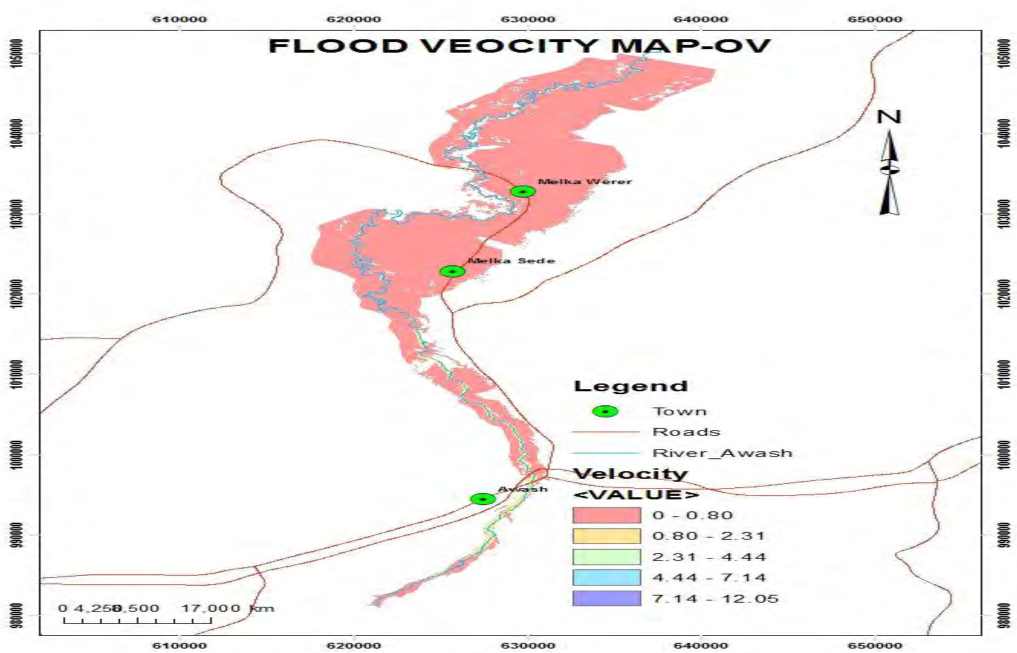
OVERTOPPING AND PIPING

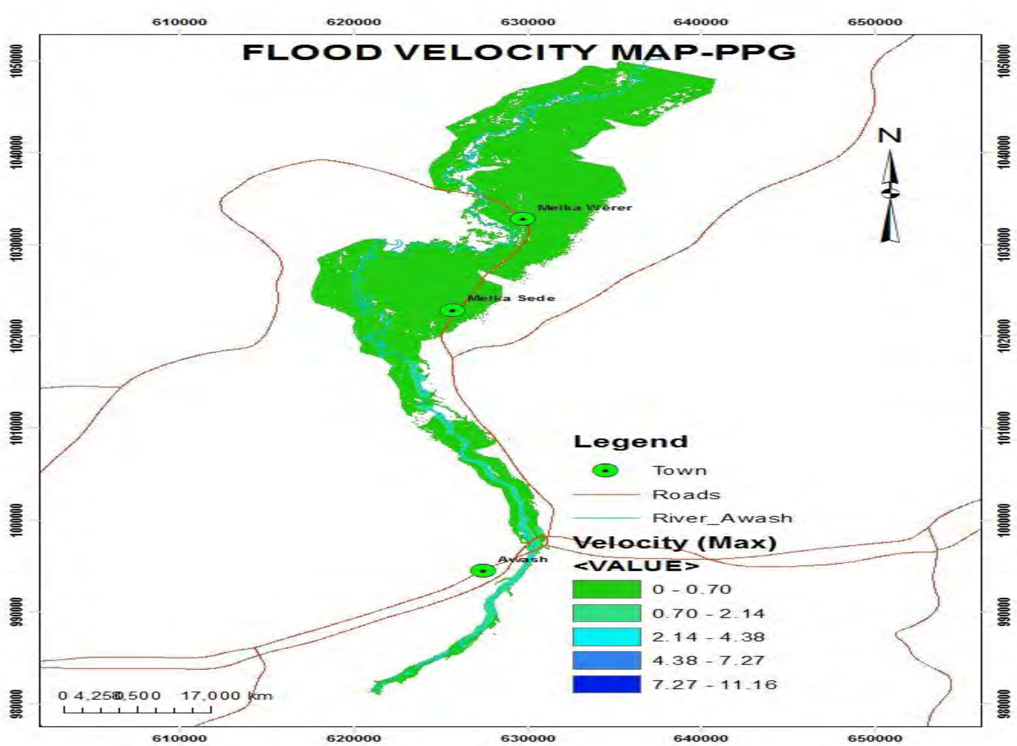


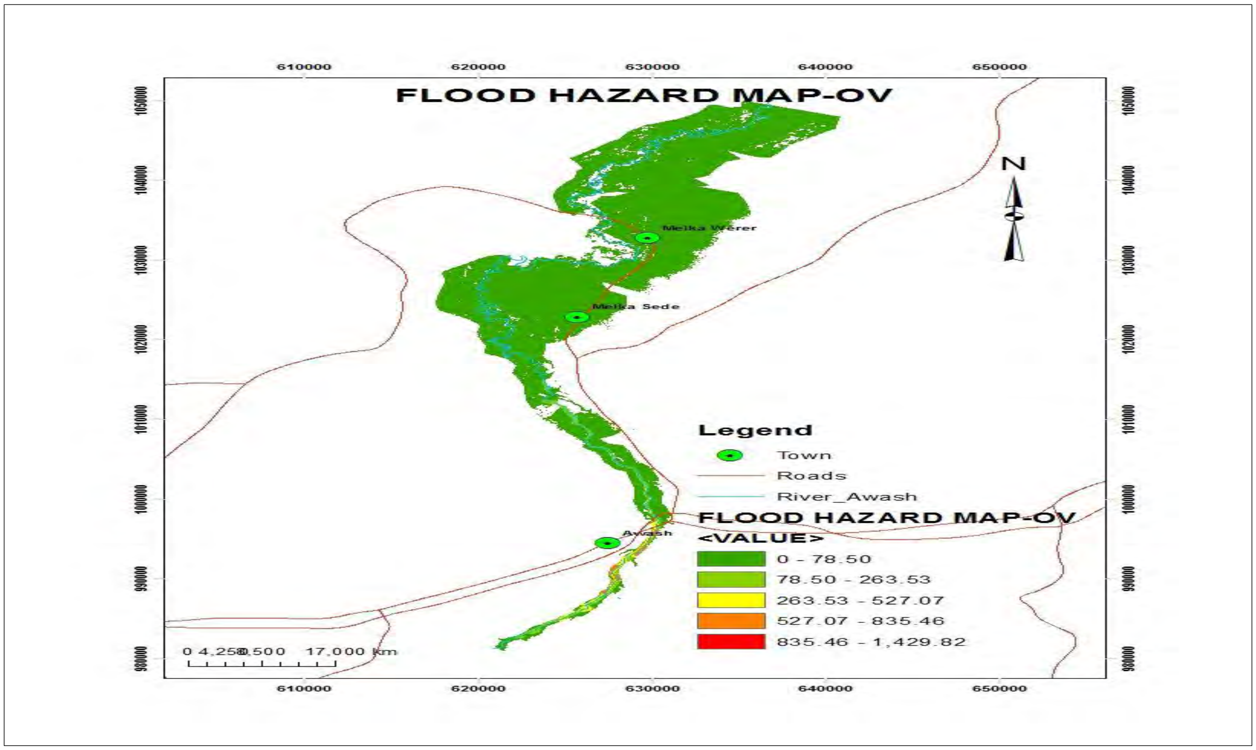


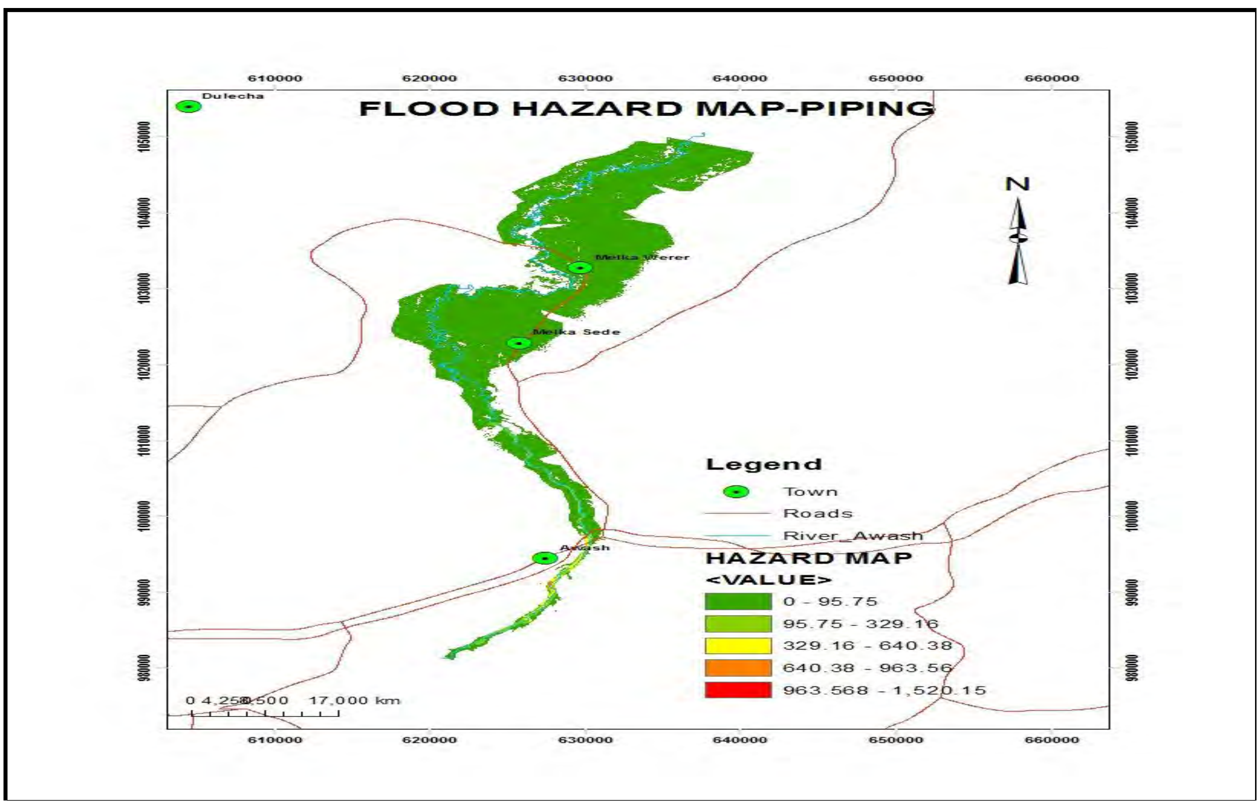

FLOOD DEPTH MAP-OV
610000
620000
630000
640000
650000
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Flood Depth Map - OV
0.0011 - 6.23
6.23 - 20.78
20.78 - 48.31
48.31 - 81.04
81.05 - 132.46
0 4,250 8,500 17,000 km

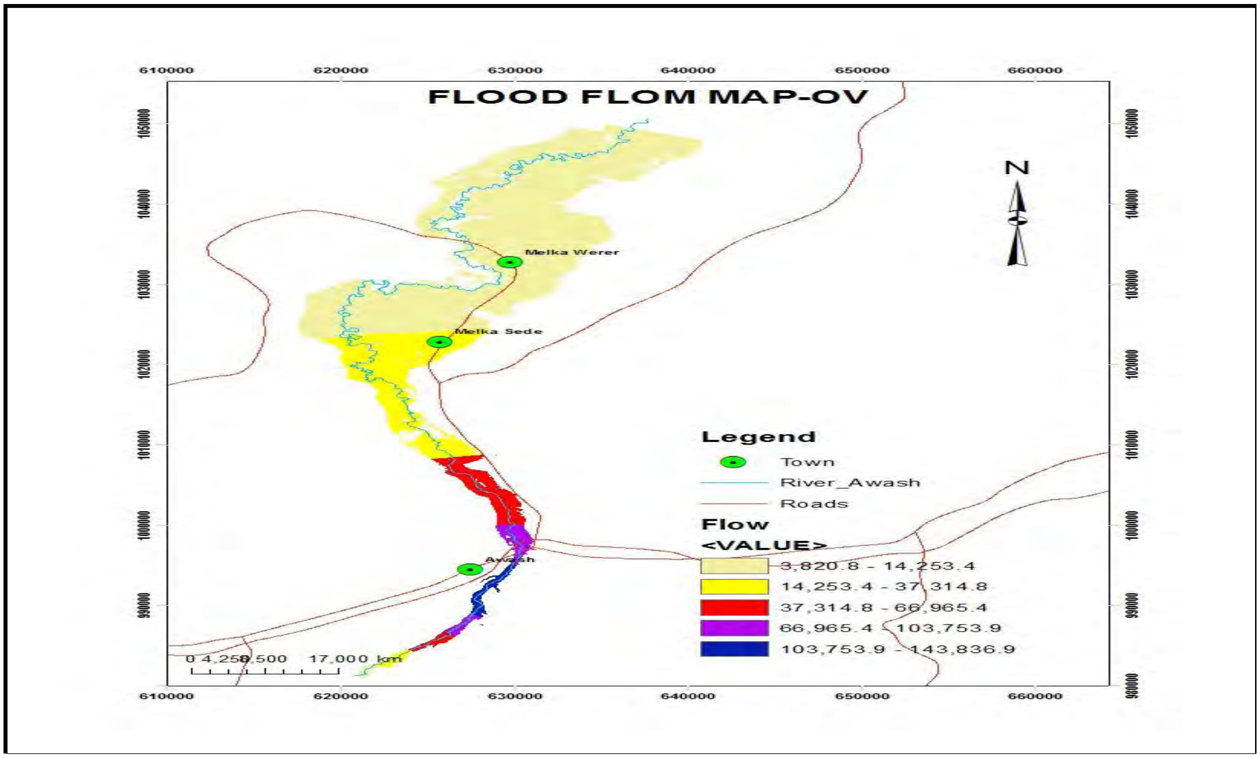

FLOOD DEPTH MAP-PIPING
610000
620000
630000
640000
650000
660000
Dulecha
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Flood Depth Map-Piping
<VALUE>
0.001 - 5.72
5.72 - 20.26
20.26 - 48.32
48.32 - 81.05
81.05 - 132.5
0 4,250 8,500 17,000 km

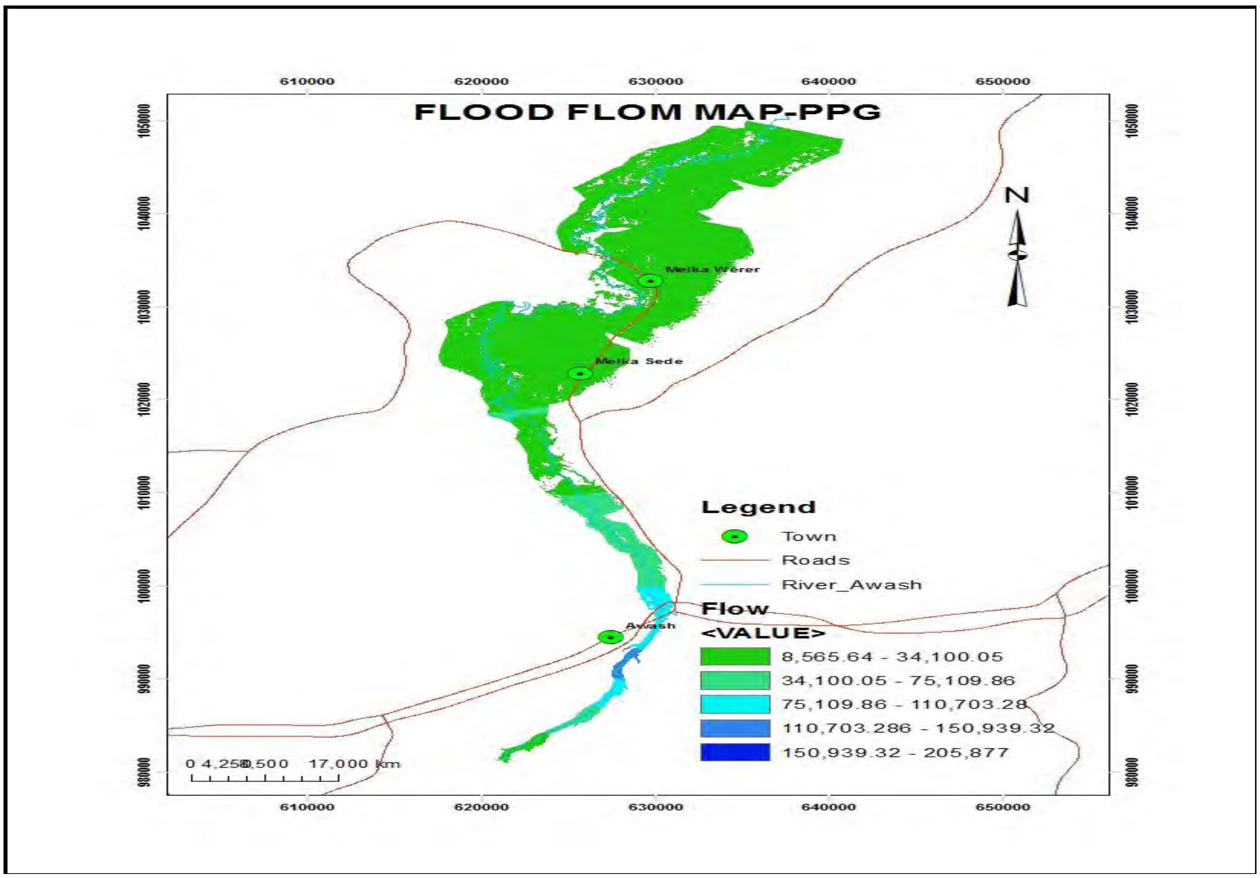

FLOOD VEOCITY MAP-OV
610000
620000
630000
640000
650000
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Velocity
<VALUE>
0 - 0.80
0.80 - 2.31
2.31 - 4.44
4.44 - 7.14
7.14 - 12.05
0 4,250 8,500 17,000 km

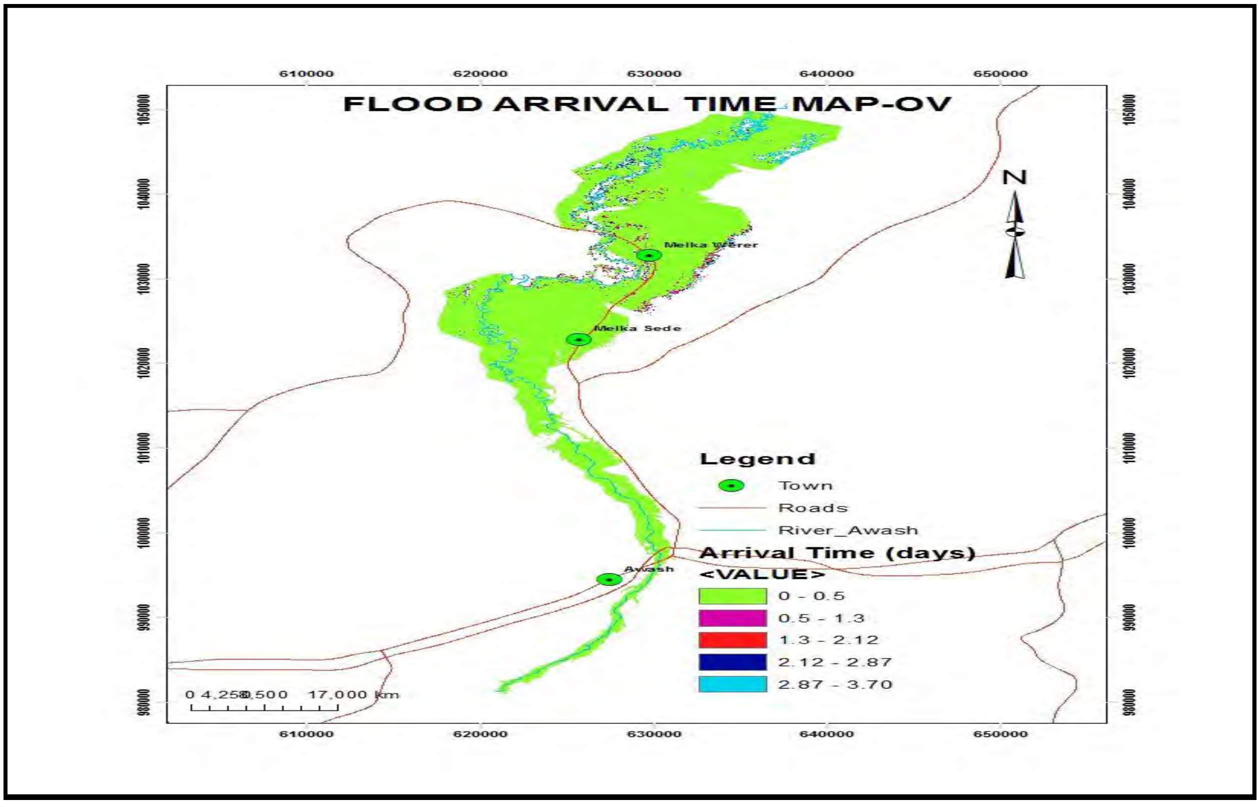

FLOOD VELOCITY MAP-PPG
610000
620000
630000
640000
650000
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Velocity (Max)
<VALUE>
0 - 0.70
0.70 - 2.14
2.14 - 4.38
4.38 - 7.27
7.27 - 11.16
0 4,250 8,500 17,000 km

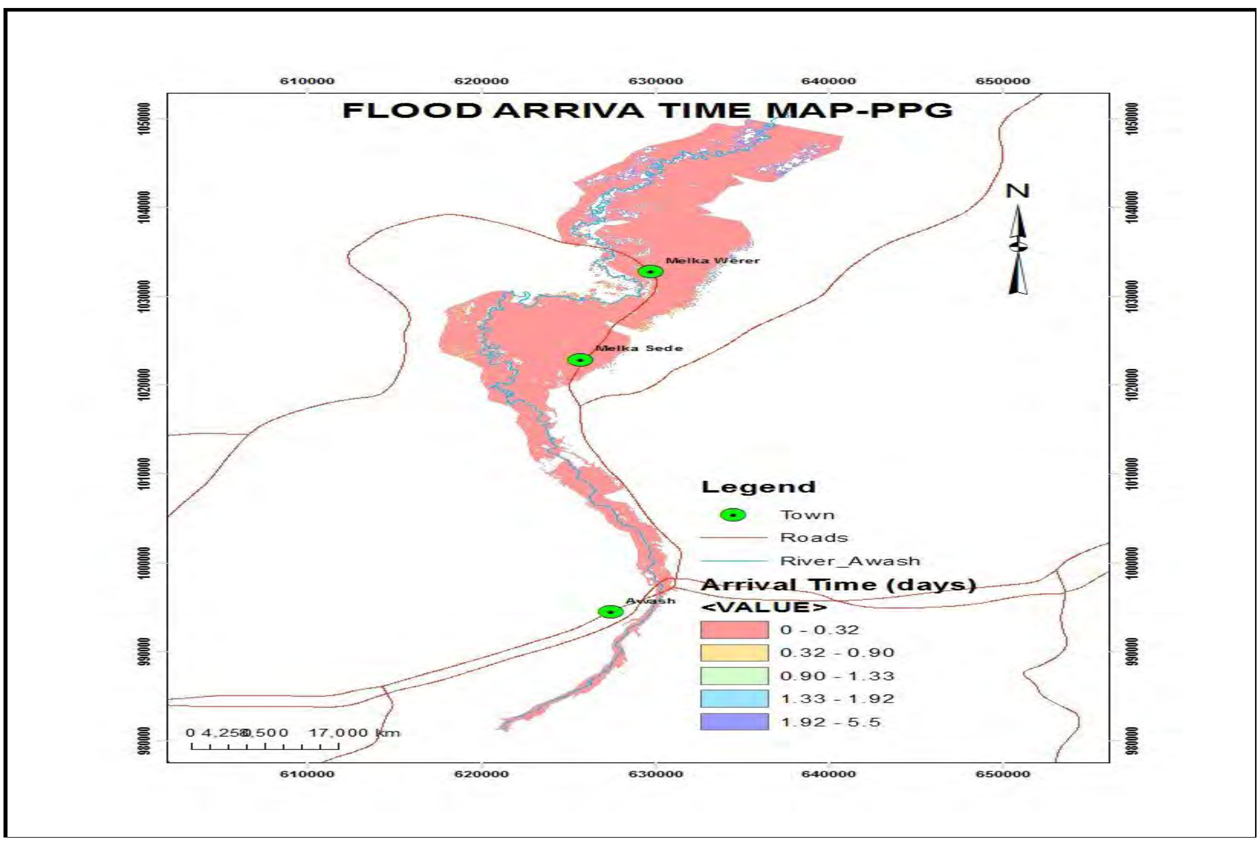

FLOOD HAZARD MAP-OV
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
FLOOD HAZARD MAP-OV
<VALUE>
0 - 78.50
78.50 - 263.53
263.53 - 527.07
527.07 - 835.46
835.46 - 1,429.82
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000

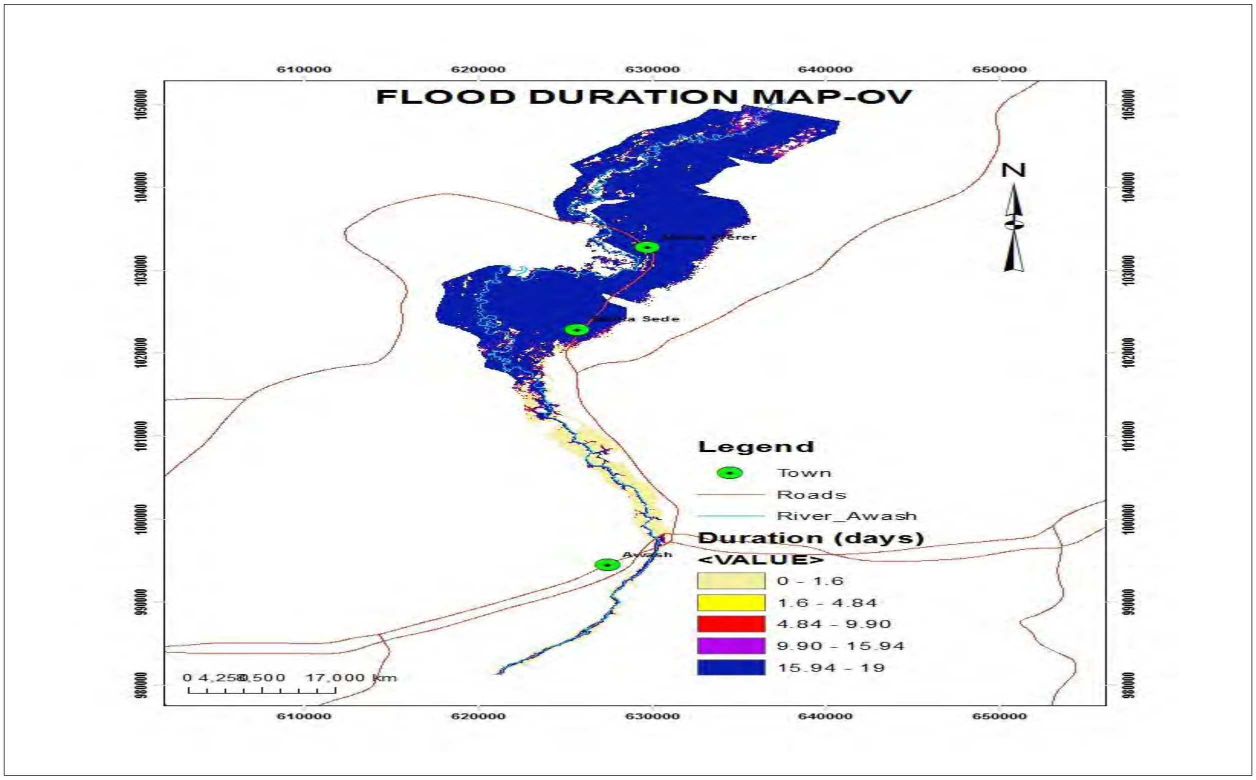

FLOOD HAZARD MAP-PIPING
Dulecha
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
HAZARD MAP
<VALUE>
0 - 95.75
95.75 - 329.16
329.16 - 640.38
640.38 - 963.56
963.568 - 1,520.15
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000
660000

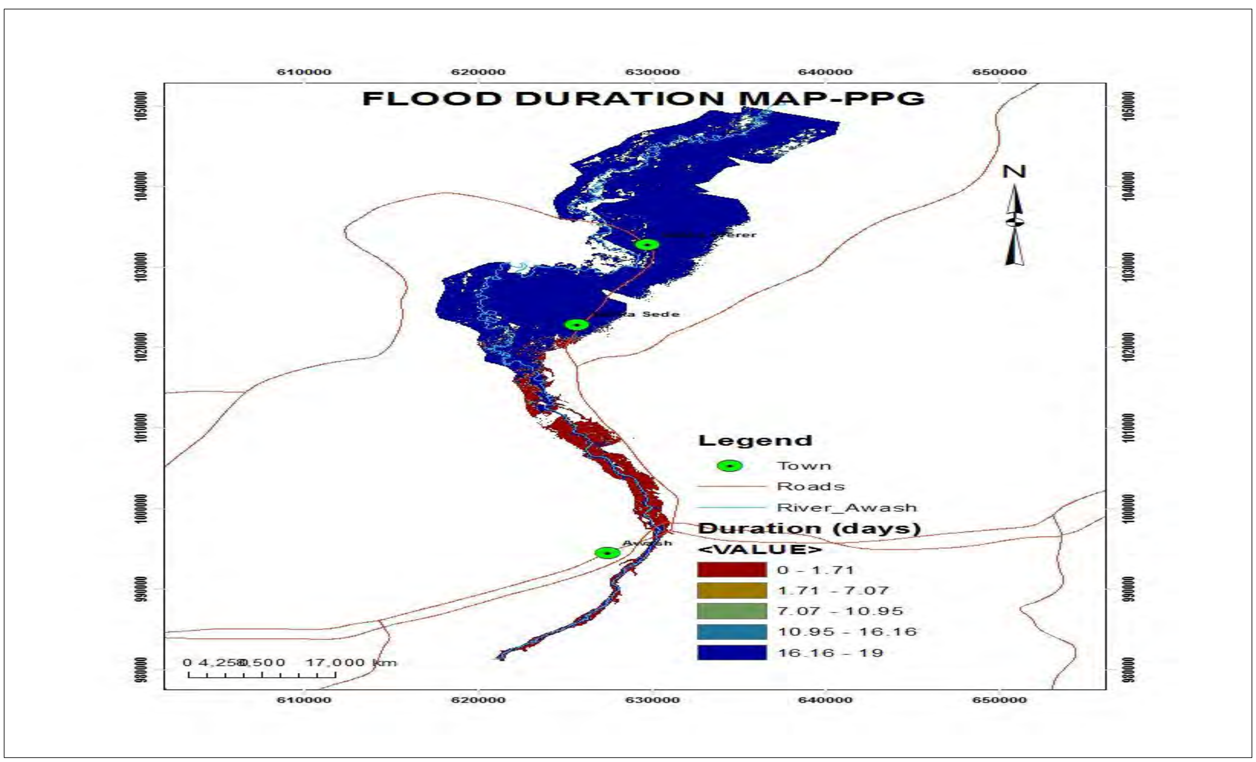

FLOOD FLOM MAP-OV
Melka Werer
Melka Sede
Awash
Legend
Town
River_Awash
Roads
Flow
<VALUE>
3,820.8 - 14,253.4
14,253.4 - 37,314.8
37,314.8 - 66,965.4
66,965.4 - 103,753.9
103,753.9 - 143,836.9
0 4,250 8,500 17,000 km
N

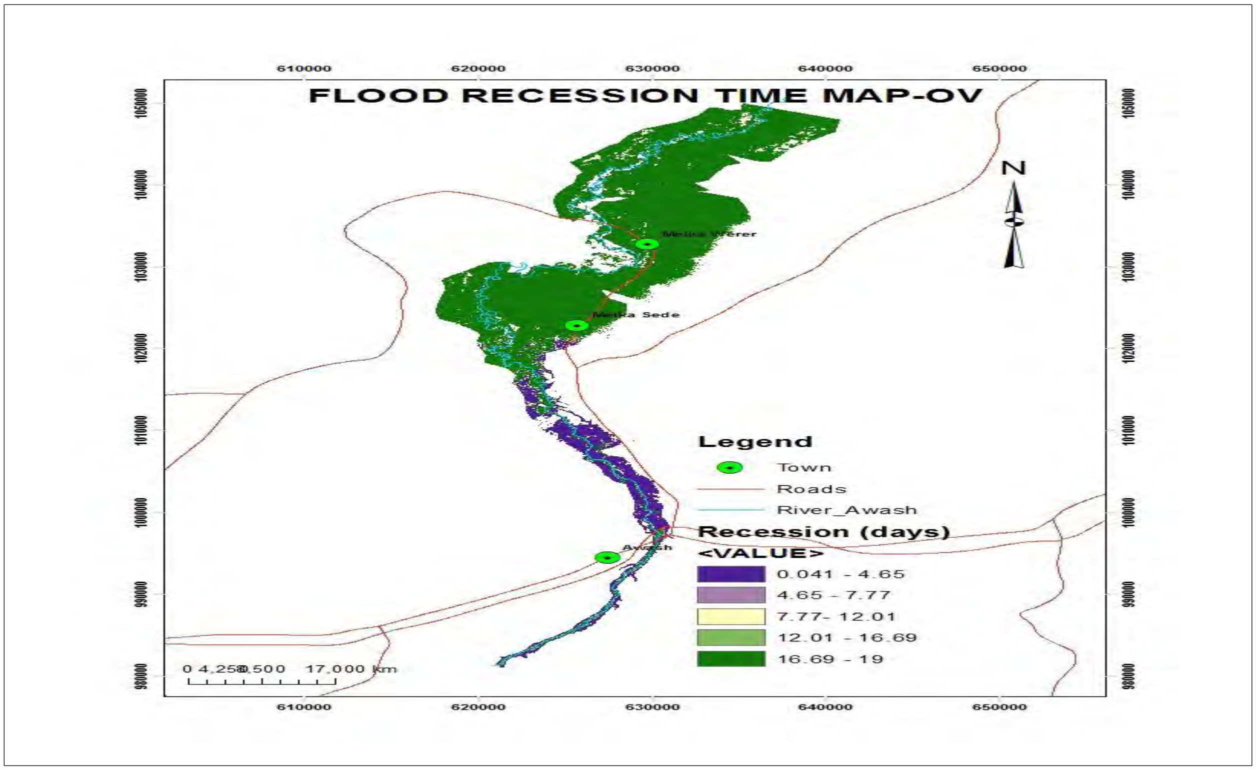

FLOOD FLOM MAP-PPG
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Flow
<VALUE>
8,565.64 - 34,100.05
34,100.05 - 75,109.86
75,109.86 - 110,703.28
110,703.286 - 150,939.32
150,939.32 - 205,877
0 4,250 8,500 17,000 km
N

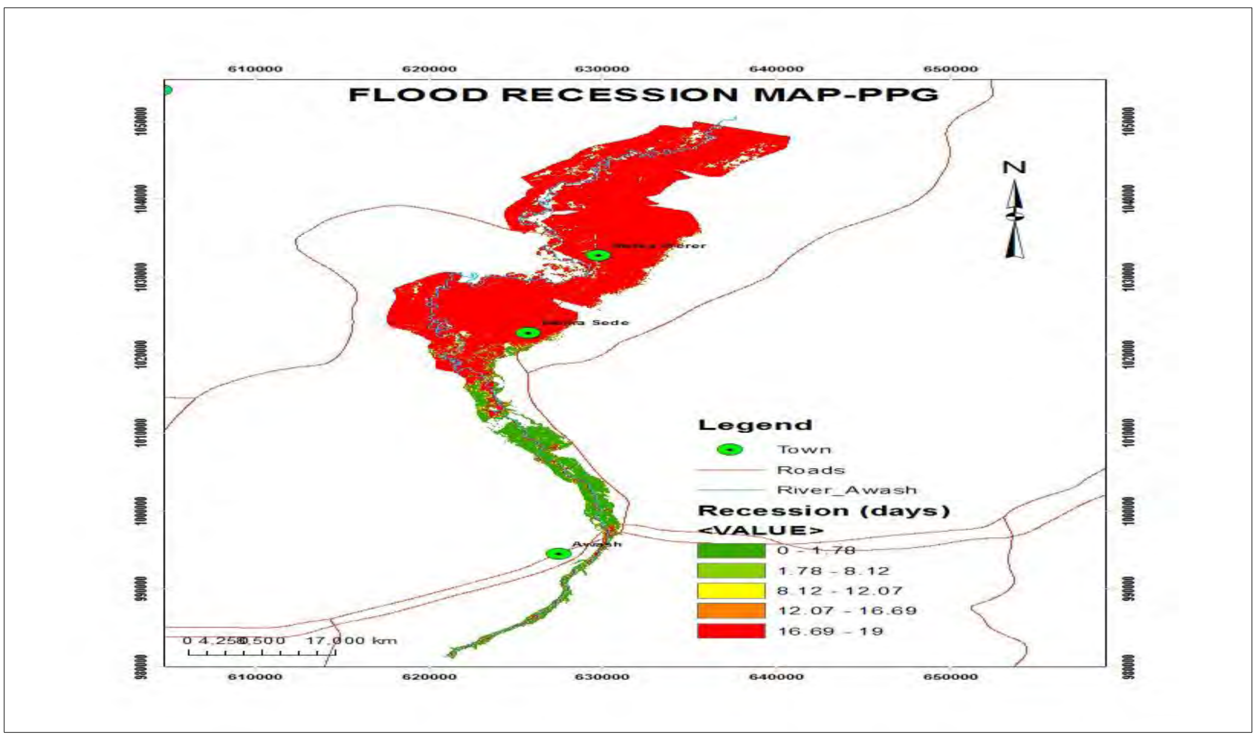

FLOOD ARRIVAL TIME MAP-OV
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Arrival Time (days)
<VALUE>
0 - 0.5
0.5 - 1.3
1.3 - 2.12
2.12 - 2.87
2.87 - 3.70
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000

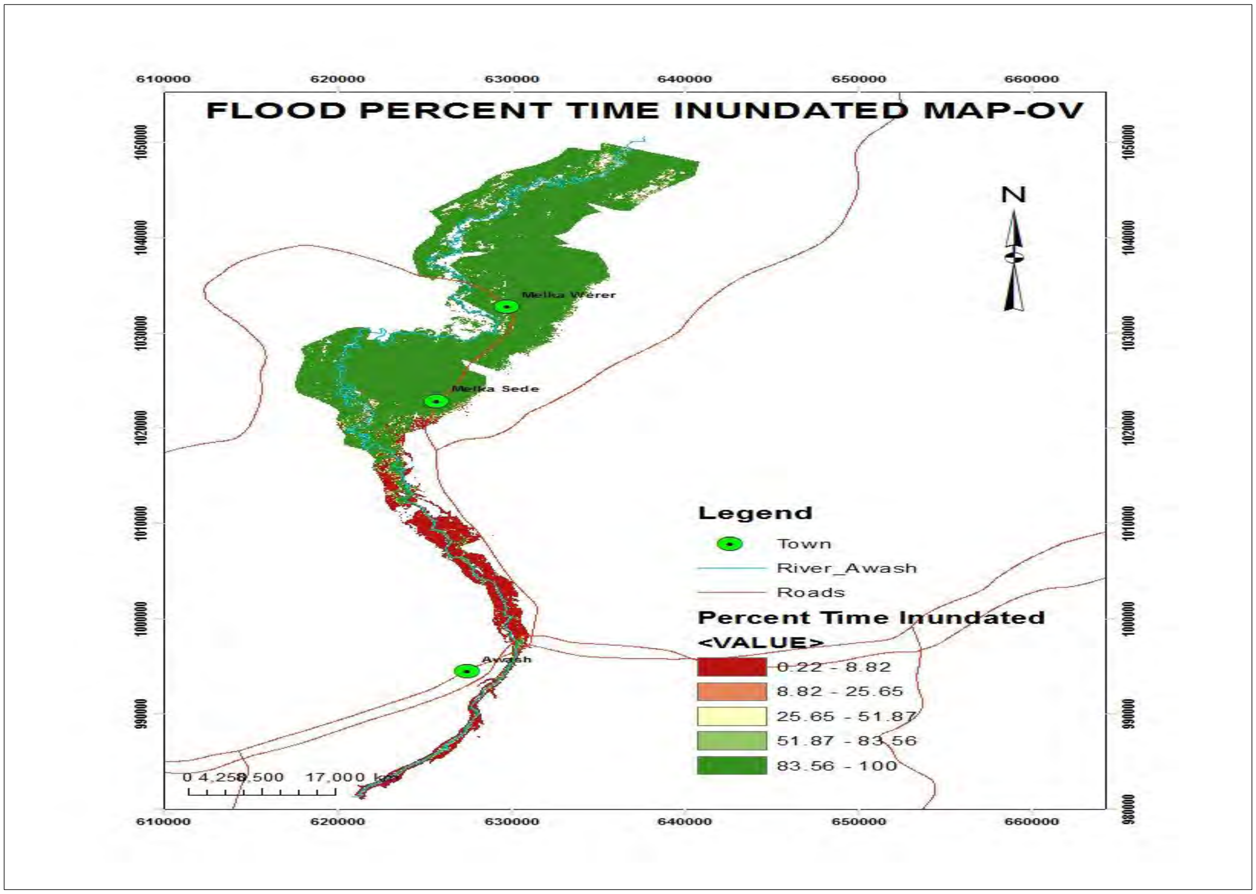

FLOOD ARRIVA TIME MAP-PPG
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Arrival Time (days)
<VALUE>
0 - 0.32
0.32 - 0.90
0.90 - 1.33
1.33 - 1.92
1.92 - 5.5
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000

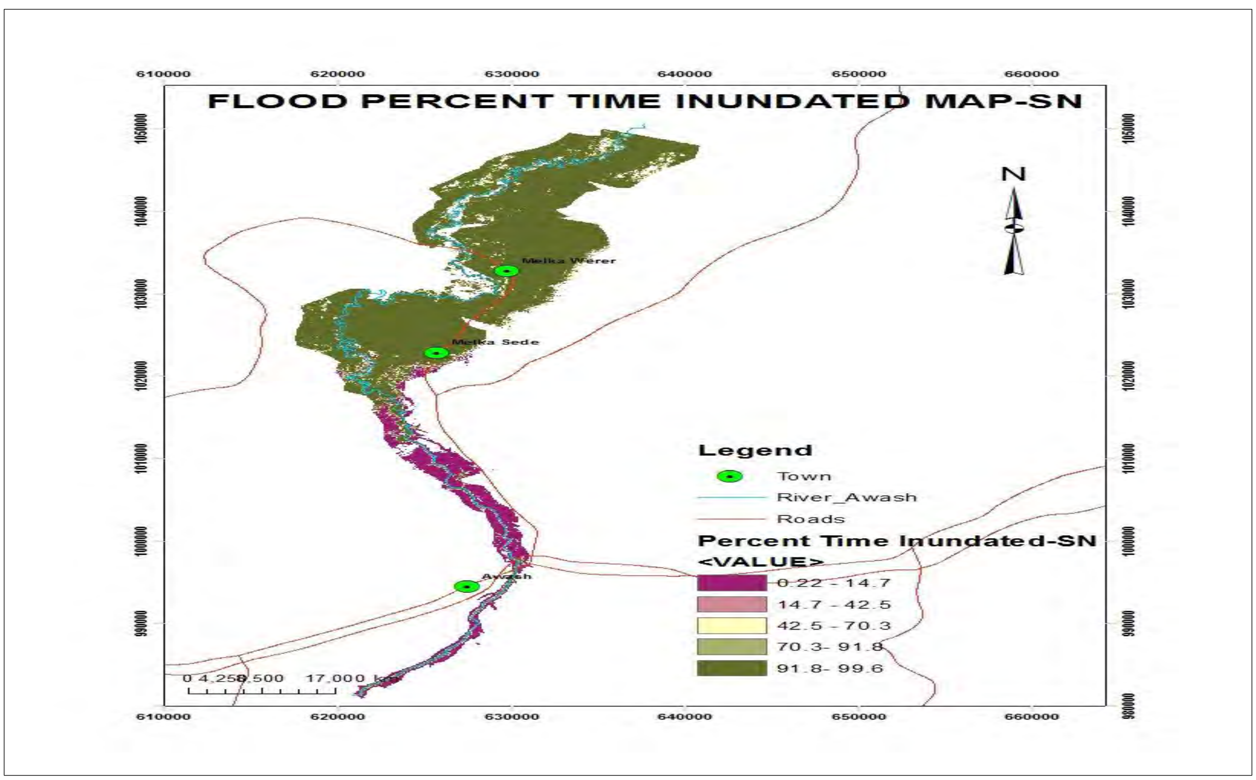

FLOOD DURATION MAP-OV
Legend
Town
Roads
River_Awash
Duration (days)
<VALUE>
0 - 1.6
1.6 - 4.84
4.84 - 9.90
9.90 - 15.94
15.94 - 19
Awash
N

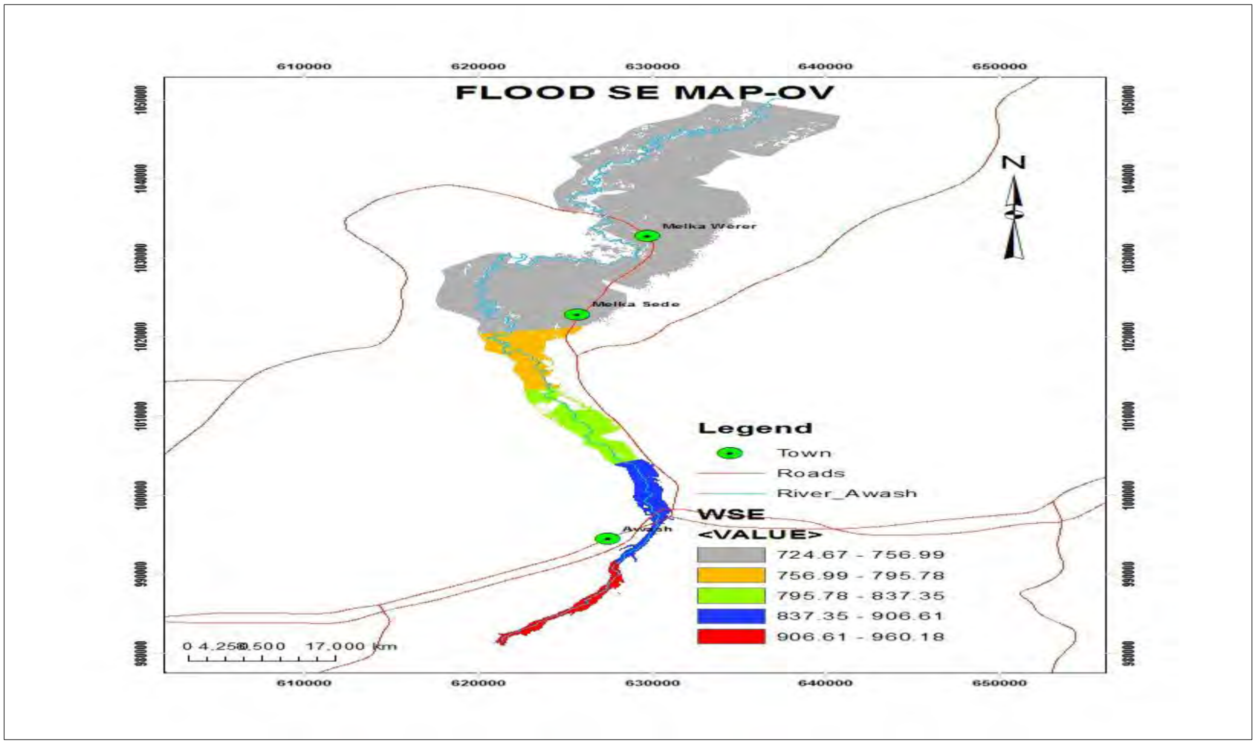

FLOOD DURATION MAP-PPG
Legend
Town
Roads
River_Awash
Duration (days)
<VALUE>
0 - 1.71
1.71 - 7.07
7.07 - 10.95
10.95 - 16.16
16.16 - 19
N

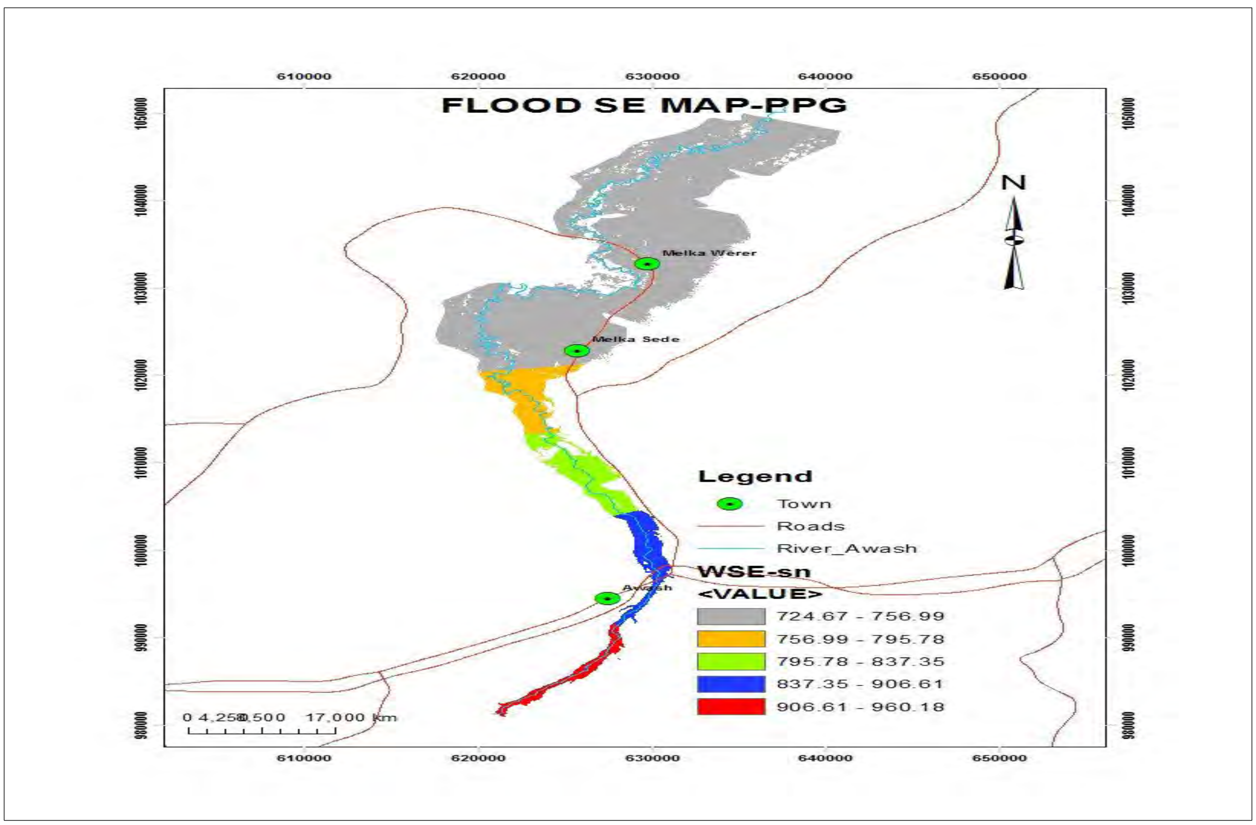

FLOOD RECESSION TIME MAP-OV
610000
620000
630000
640000
650000
N
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Recession (days)
<VALUE>
0.041 - 4.65
4.65 - 7.77
7.77- 12.01
12.01 - 16.69
16.69 - 19
0 4,250 8,500 17,000 km


FLOOD RECESSION MAP-PPG
610000
620000
630000
640000
650000
N
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Recession (days)
<VALUE>
0 - 1.78
1.78 - 8.12
8.12 - 12.07
12.07 - 16.69
16.69 - 19
0 4,250 8,500 17,000 km


FLOOD PERCENT TIME INUNDATED MAP-OV
610000
620000
630000
640000
650000
660000
N
Melka Werer
Melka Sede
Awash
Legend
Town
River_Awash
Roads
Percent Time Inundated
<VALUE>
0.22 - 8.82
8.82 - 25.65
25.65 - 51.87
51.87 - 83.56
83.56 - 100
0 4,250 8,500 17,000 km


FLOOD PERCENT TIME INUNDATED MAP-SN
610000
620000
630000
640000
650000
660000
N
Melka Werer
Melka Sede
Awash
Legend
Town
River_Awash
Roads
Percent Time Inundated-SN
<VALUE>
0.22 - 14.7
14.7 - 42.5
42.5 - 70.3
70.3- 91.8
91.8- 99.6
0 4,250 8,500 17,000 km


FLOOD SE MAP-OV
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
WSE
<VALUE>
724.67 - 756.99
756.99 - 795.78
795.78 - 837.35
837.35 - 906.61
906.61 - 960.18
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000


FLOOD SE MAP-PPG
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
WSE-sn
<VALUE>
724.67 - 756.99
756.99 - 795.78
795.78 - 837.35
837.35 - 906.61
906.61 - 960.18
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000

9.2. Middle Awash PMF

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
9:00	292.969	11:00	4267.296	15:00	3193.752	19:00	2103.543
10:00	293.064	12:00	4271.671	16:00	3180.491	20:00	2094.89
11:00	293.627	13:00	4275.209	17:00	3167.275	21:00	2086.278
12:00	295.293	14:00	4277.983	18:00	3154.102	22:00	2077.706
13:00	298.7595	15:00	4279.977	19:00	3140.973	23:00	2069.171
14:00	305.1095	16:00	4281.182	20:00	3127.891	0:00	2060.675
15:00	316.5975	17:00	4281.671	21:00	3114.857	1:00	2052.22
16:00	336.107	18:00	4281.512	22:00	3101.867	2:00	2043.803
17:00	365.429	19:00	4280.683	23:00	3088.922	3:00	2035.417
18:00	405.1955	20:00	4279.162	0:00	3076.027	4:00	2027.06
19:00	456.5335	21:00	4277.021	1:00	3063.181	5:00	2018.732
20:00	521.699	22:00	4274.311	2:00	3050.381	6:00	2010.435
21:00	602.9785	23:00	4271.006	3:00	3037.627	7:00	2002.172
22:00	701.155	0:00	4267.098	4:00	3024.922	8:00	1993.944
23:00	816.1735	1:00	4262.667	5:00	3012.268	9:00	1985.755
0:00	948.172	2:00	4257.763	6:00	2999.661	10:00	1977.604
1:00	1098.286	3:00	4252.366	7:00	2987.101	11:00	1969.49
2:00	1258.6	4:00	4246.47	8:00	2974.591	12:00	1961.412
3:00	1425.488	5:00	4240.129	9:00	2962.133	13:00	1953.378
4:00	1595.601	6:00	4233.382	10:00	2949.724	14:00	1945.404
5:00	1769.435	7:00	4226.214	11:00	2937.362	15:00	1937.497
6:00	1944.691	8:00	4218.618	12:00	2925.051	16:00	1929.626
7:00	2108.005	9:00	4210.638	13:00	2912.793	17:00	1921.764
8:00	2265.451	10:00	4202.313	14:00	2900.583	18:00	1913.916
9:00	2411.71	11:00	4193.644	15:00	2888.419	19:00	1906.092
10:00	2543.13	12:00	4184.626	16:00	2876.308	20:00	1898.299
11:00	2658.566	13:00	4175.292	17:00	2864.248	21:00	1890.542
12:00	2758.235	14:00	4165.675	18:00	2852.237	22:00	1882.821
13:00	2843.317	15:00	4155.773	19:00	2840.273	23:00	1875.133
14:00	2915.188	16:00	4145.58	20:00	2828.362	0:00	1867.481
15:00	2974.438	17:00	4135.121	21:00	2816.503	1:00	1859.864
16:00	3022.018	18:00	4124.425	22:00	2804.693	2:00	1852.284

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
17:00	3059.883	19:00	4113.492	23:00	2792.93	3:00	1844.736
18:00	3090.107	20:00	4102.32	0:00	2781.218	4:00	1837.223
19:00	3114.245	21:00	4090.937	1:00	2769.557	5:00	1829.745
20:00	3133.789	22:00	4079.366	2:00	2757.945	6:00	1822.302
21:00	3150.041	23:00	4067.608	3:00	2746.381	7:00	1814.892
22:00	3163.836	0:00	4055.658	4:00	2734.867	8:00	1807.516
23:00	3175.489	1:00	4043.538	5:00	2723.406	9:00	1800.175
0:00	3185.141	2:00	4031.265	6:00	2711.994	10:00	1792.867
1:00	3192.888	3:00	4018.838	7:00	2700.629	11:00	1785.593
2:00	3198.699	4:00	4006.255	8:00	2689.314	12:00	1778.35
3:00	3202.386	5:00	3993.535	9:00	2678.05	13:00	1771.141
4:00	3204.052	6:00	3980.695	10:00	2666.834	14:00	1763.966
5:00	3204.108	7:00	3967.738	11:00	2655.665	15:00	1756.823
6:00	3202.914	8:00	3954.662	12:00	2644.546	16:00	1749.712
7:00	3200.694	9:00	3941.481	13:00	2633.478	17:00	1742.635
8:00	3197.726	10:00	3928.208	14:00	2622.457	18:00	1735.591
9:00	3194.293	11:00	3914.846	15:00	2611.484	19:00	1728.578
10:00	3190.575	12:00	3901.392	16:00	2600.56	20:00	1721.596
11:00	3186.679	13:00	3887.857	17:00	2589.687	21:00	1714.647
12:00	3182.886	14:00	3874.251	18:00	2578.861	22:00	1707.73
13:00	3180.054	15:00	3860.58	19:00	2568.08	23:00	1700.844
14:00	3179.824	16:00	3846.841	20:00	2557.348	0:00	1693.988
15:00	3184.675	17:00	3833.045	21:00	2546.665	1:00	1687.164
16:00	3196.131	18:00	3819.2	22:00	2536.029	2:00	1680.373
17:00	3213.81	19:00	3805.311	23:00	2525.439	3:00	1673.613
18:00	3236.677	20:00	3791.375	0:00	2514.897	4:00	1666.882
19:00	3264.098	21:00	3777.401	1:00	2504.404	5:00	1660.182
20:00	3295.281	22:00	3763.394	2:00	2493.959	6:00	1653.514
21:00	3329.183	23:00	3749.358	3:00	2483.565	7:00	1646.876
22:00	3365.018	0:00	3735.293	4:00	2473.246	8:00	1640.267
23:00	3402.383	1:00	3721.204	5:00	2463.007	9:00	1633.688
0:00	3440.931	2:00	3707.099	6:00	2452.812	10:00	1627.141
1:00	3480.23	3:00	3692.979	7:00	2442.633	11:00	1620.623
2:00	3519.782	4:00	3678.845	8:00	2432.473	12:00	1614.134
3:00	3559.089	5:00	3664.702	9:00	2422.349	13:00	1607.675

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
4:00	3597.864	6:00	3650.553	10:00	2412.267	14:00	1601.247
5:00	3636.025	7:00	3636.401	11:00	2402.228	15:00	1594.847
6:00	3673.499	8:00	3622.248	12:00	2392.235	16:00	1588.476
7:00	3710.106	9:00	3608.096	13:00	2382.288	17:00	1582.134
8:00	3745.718	10:00	3593.95	14:00	2372.386	18:00	1576.236
9:00	3780.328	11:00	3579.811	15:00	2362.527	19:00	1569.974
10:00	3813.915	12:00	3565.681	16:00	2352.712	20:00	1563.715
11:00	3846.349	13:00	3551.563	17:00	2342.942	21:00	1557.485
12:00	3877.561	14:00	3537.459	18:00	2333.215	22:00	1551.285
13:00	3907.595	15:00	3523.37	19:00	2323.529	23:00	1545.112
14:00	3936.467	16:00	3509.299	20:00	2313.885	0:00	1538.966
15:00	3964.079	17:00	3495.247	21:00	2304.286	1:00	1532.848
16:00	3990.374	18:00	3481.215	22:00	2294.727	2:00	1526.759
17:00	4015.411	19:00	3467.206	23:00	2285.206	3:00	1520.698
18:00	4039.24	20:00	3453.222	0:00	2275.724	4:00	1514.663
19:00	4061.792	21:00	3439.263	1:00	2266.285	5:00	1508.656
20:00	4083.044	22:00	3425.332	2:00	2256.887	6:00	1502.678
21:00	4103.094	23:00	3411.428	3:00	2247.53	7:00	1496.727
22:00	4122.003	0:00	3397.554	4:00	2238.215	8:00	1490.802
23:00	4139.708	1:00	3383.713	5:00	2228.945	9:00	1484.905
0:00	4156.181	2:00	3369.902	6:00	2219.718	10:00	1479.037
1:00	4171.528	3:00	3356.124	7:00	2210.532	11:00	1473.194
2:00	4185.799	4:00	3342.382	8:00	2201.388	12:00	1467.377
3:00	4198.928	5:00	3328.677	9:00	2192.288	13:00	1461.588
4:00	4210.909	6:00	3315.007	10:00	2183.229	14:00	1455.825
5:00	4221.858	7:00	3301.373	11:00	2174.211	15:00	1450.089
6:00	4231.842	8:00	3287.779	12:00	2165.233	16:00	1444.378
7:00	4240.817	9:00	3274.224	13:00	2156.298	17:00	1438.694
8:00	4248.781	10:00	3260.709	14:00	2147.404	18:00	1433.037
9:00	4255.825	11:00	3247.232	15:00	2138.55	19:00	1427.406
10:00	4262.007	12:00	3233.798	16:00	2129.737	20:00	1421.799
	0	13:00	3220.408	17:00	2120.966	21:00	1416.219
	0	14:00	3207.059	18:00	2112.235	22:00	1410.665
	0		0		0	23:00	1405.137

9.3. Middle Awash Volume

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
821	591.0642	0	0	-	20.1043	0
822	3887.7328	2239.3985	2239.3985	2,239.40	28.7079	1
823	7051.1977	5469.46525	5469.46525	7,708.86	37.0005	2
824	12048.9865	9550.0921	9550.0921	17,258.96	43.5288	3
825	18486.6343	15267.8104	15267.8104	32,526.77	48.5649	4
826	24222.8807	21354.7575	21354.7575	53,881.52	53.5665	5
827	32127.3056	28175.0932	28175.09315	82,056.62	57.8792	6
828	40507.8955	36317.6006	36317.60055	118,374.22	62.1875	7
829	48005.9194	44256.9075	44256.90745	162,631.12	66.4957	8
830	54540.3721	51273.1458	51273.14575	213,904.27	70.2646	9
831	60514.1879	57527.28	57527.28	271,431.55	73.4954	10
832	67876.7093	64195.4486	64195.4486	335,627.00	76.7052	11
833	73801.0404	70838.8749	70838.87485	406,465.87	79.9143	12
834	97139.6831	85470.3618	85470.36175	491,936.24	83.1234	13
835	109932.857	103536.27	103536.2701	595,472.51	86.3324	14
836	123461.1768	116697.017	116697.0169	712,169.52	89.5129	15
837	137752.8201	130606.998	130606.9985	842,776.52	93.4736	16
838	152360.3035	145056.562	145056.5618	987,833.08	97.8895	17
839	167007.6403	159683.972	159683.9719	1,147,517.05	102.3025	18
840	200363.6649	183685.653	183685.6526	1,331,202.71	106.7113	19
841	225480.6699	212922.167	212922.1674	1,544,124.0	115.5007	20
842	252490.5225	238985.596	238985.5962	1,783,110.47	168.826	21

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
843	287151.0776	269820.8	269820.8001	2,052,931.27	181.6663	22
844	333898.9208	310524.999	310524.9992	2,363,456.27	193.1585	23
845	369292.288	351595.604	351595.6044	2,715,051.87	205.3197	24
846	406721.0278	388006.658	388006.6579	3,103,058.53	218.1153	25
847	446801.7165	426761.372	426761.3722	3,529,819.90	228.5584	26
848	554181.5199	500491.618	500491.6182	4,030,311.52	236.7833	27
849	626123.4892	590152.505	590152.5046	4,620,464.03	242.958	28
850	686284.1296	656203.809	656203.8094	5,276,667.84	248.52	29
851	750227.8482	718255.989	718255.9889	5,994,923.83	252.6816	30
852	820573.914	785400.881	785400.8811	6,780,324.71	256.2757	31
853	893678.9252	857126.42	857126.4196	7,637,451.13	259.8812	32
854	961983.1217	927831.023	927831.0235	8,565,282.15	262.9809	33
855	1032051.897	997017.509	997017.5092	9,562,299.66	265.4513	34
856	1104881.702	1068466.8	1068466.8	10,630,766.46	267.9032	35
857	1177252.222	1141066.96	1141066.962	11,771,833.42	270.3563	36
858	1250099.57	1213675.9	1213675.896	12,985,509.32	272.7852	37
859	1327214.506	1288657.04	1288657.038	14,274,166.35	275.0011	38
860	1405443.375	1366328.94	1366328.94	15,640,495.30	277.0965	39
861	2357579.477	1881511.43	1881511.426	17,522,006.72	279.168	40
862	2488163.764	2422871.62	2422871.621	19,944,878.34	281.2395	41
863	2623987.327	2556075.55	2556075.546	22,500,953.89	283.3111	42
864	2764880.02	2694433.67	2694433.673	25,195,387.56	285.3824	43
865	2946190.583	2855535.3	2855535.302	28,050,922	287.4537	44

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
866	3104947.871	3025569.23	3025569.227	31,076,492.09	289.5761	45
867	3265895.714	3185421.79	3185421.793	34,261,913.88	291.8011	46
868	3428110.993	3347003.35	3347003.354	37,608,917.24	294.0967	47
869	3585082.336	3506596.66	3506596.665	41,115,513.90	296.7493	48
870	3743959.358	3664520.85	3664520.847	44,780,034.75	299.6159	49
871	3896402.984	3820181.17	3820181.171	48,600,215.92	302.5569	50
872	4066266.247	3981334.62	3981334.615	52,581,550.53	305.7529	51
873	4227857.433	4147061.84	4147061.84	56,728,612.37	309.1325	52
874	4386212.56	4307035	4307034.996	61,035,647.37	312.5081	53
875	4542658.303	4464435.43	4464435.432	65,500,082.80	315.8807	54
876	4695026.088	4618842.2	4618842.196	70,118,925.00	319.2841	55
877	4847834.877	4771430.48	4771430.483	74,890,355.48	322.6998	56
878	5000455.073	4924144.98	4924144.975	79,814,500.46	326.7623	57
879	5184726.296	5092590.68	5092590.685	84,907,091.14	330.998	58
880	5348332.2	5266529.25	5266529.248	90,173,620.39	335.2569	59
881	5498008.099	5423170.15	5423170.15	95,596,790.54	339.3181	60
882	5635537.801	5566772.95	5566772.95	101,163,563.49	343.3681	61
883	5768271.988	5701904.89	5701904.895	106,865,468.38	347.1197	62
884	5901734.473	5835003.23	5835003.231	112,700,471.61	350.8517	63
885	6038913.858	5970324.17	5970324.166	118,670,795.78	354.5471	64
886	6169907.64	6104410.75	6104410.749	124,775,206.53	358.2091	65
887	6300579.218	6235243.43	6235243.429	131,010,449.96	361.8717	66
888	6436724.353	6368651.79	6368651.786	137,379,101	365.5341	67

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
889	6572571.447	6504647.9	6504647.9	143,883,749.64	369.1249	68
890	6720317.381	6646444.41	6646444.414	150,530,194.06	372.8904	69
891	6859463.731	6789890.56	6789890.556	157,320,084.61	376.5965	70
892	6996913.09	6928188.41	6928188.41	164,248,273.02	380.3057	71
893	7126006.469	7061459.78	7061459.779	171,309,732.80	384.0149	72
894	7250783.13	7188394.8	7188394.8	178,498,127.60	387.7213	73
895	7371608.709	7311195.92	7311195.92	185,809,323.52	391.048	74
896	7492988.546	7432298.63	7432298.627	193,241,622.15	393.7936	75
897	8216240.365	7854614.46	7854614.455	201,096,236.60	396.5095	76
898	8362147.348	8289193.86	8289193.856	209,385,430.46	399.2481	77
899	8511637.08	8436892.21	8436892.214	217,822,322.67	401.9762	78
900	8653054.323	8582345.7	8582345.702	226,404,668.38	404.7028	79
901	8791962.931	8722508.63	8722508.627	235,127,177.00	407.3564	80
902	8941094.047	8866528.49	8866528.489	243,993,705.49	409.9596	81
903	9085347.914	9013220.98	9013220.98	253,006,926.47	412.5627	82
904	9247779.737	9166563.83	9166563.825	262,173,490.30	415.1576	83
905	9389454.229	9318616.98	9318616.983	271,492,107.28	417.7826	84
906	9534390.867	9461922.55	9461922.548	280,954,029.83	420.5161	85
907	9669099.838	9601745.35	9601745.352	290,555,775.18	423.2212	86
908	9804422.625	9736761.23	9736761.231	300,292,536.41	425.9258	87
909	9942238.124	9873330.37	9873330.375	310,165,866.79	428.6309	88
910	10125427.65	10033832.9	10033832.89	320,199,699.67	431.3363	89
911	10258991.71	10192209.7	10192209.68	330,391,909	434.042	90

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
912	10395157.06	10327074.4	10327074.38	340,718,983.74	436.7477	91
913	10537767.87	10466462.5	10466462.46	351,185,446.20	439.441	92
914	10677170.97	10607469.4	10607469.42	361,792,915.62	442.8257	93
915	10838424.46	10757797.7	10757797.71	372,550,713.33	447.084	94
916	10986345.16	10912384.8	10912384.81	383,463,098.13	451.1967	95
917	11136516.14	11061430.6	11061430.65	394,524,528.78	455.2691	96
918	11281518.49	11209017.3	11209017.32	405,733,546.10	459.1173	97
919	11419535.11	11350526.8	11350526.8	417,084,072.90	462.5048	98
920	11609746.74	11514640.9	11514640.92	428,598,713.82	465.8482	99
921	11759671.07	11684708.9	11684708.9	440,283,422.72	469.199	100
922	11911085.71	11835378.4	11835378.39	452,118,801.11	472.5491	101
923	12052983.5	11982034.6	11982034.61	464,100,835.72	475.8849	102
924	12198196.9	12125590.2	12125590.2	476,226,425.92	479.1528	103
925	12344589.76	12271393.3	12271393.33	488,497,819.25	482.3581	104
926	12507769.81	12426179.8	12426179.79	500,923,999.04	485.5851	105
927	12666745.25	12587257.5	12587257.53	513,511,256.57	488.8752	106
928	12902254.66	12784500	12784499.96	526,295,756.53	492.2296	107
929	13095449.76	12998852.2	12998852.21	539,294,608.74	495.5168	108
930	13293573.07	13194511.4	13194511.41	552,489,120.16	498.7246	109
931	13486780.65	13390176.9	13390176.86	565,879,297.02	501.8982	110
932	13727144.3	13606962.5	13606962.47	579,486,259.49	505.0713	111
933	13933404.88	13830274.6	13830274.59	593,316,534.07	508.4633	112
934	14154450.97	14043927.9	14043927.92	607,360,462	511.9001	113

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
935	14333935.01	14244193	14244192.99	621,604,654.99	515.2738	114
936	14516463.36	14425199.2	14425199.18	636,029,854.17	518.5641	115
937	14740538.59	14628501	14628500.97	650,658,355.14	521.7788	116
938	14930100.62	14835319.6	14835319.6	665,493,674.74	524.98	117
939	15115696.14	15022898.4	15022898.38	680,516,573.12	528.4333	118
940	15297284.38	15206490.3	15206490.26	695,723,063.38	531.8448	119
941	15479319.88	15388302.1	15388302.13	711,111,365.51	535.1497	120
942	15662183.92	15570751.9	15570751.9	726,682,117.41	538.4751	121
943	15870450.97	15766317.4	15766317.45	742,448,434.85	541.7265	122
944	16086050.94	15978251	15978250.96	758,426,685.81	545.7146	123
945	16334615.03	16210333	16210332.99	774,637,018.80	549.9058	124
946	16589254.17	16461934.6	16461934.6	791,098,953.40	554.0968	125
947	19295800.35	17942527.3	17942527.26	809,041,480.66	558.1797	126
948	21913195.08	20604497.7	20604497.71	829,645,978.37	562.0447	127
949	27338642.14	24625918.6	24625918.61	854,271,896.98	566.0401	128
950	32508422.64	29923532.4	29923532.39	884,195,429.37	570.2715	129
951	47606165.62	40057294.1	40057294.13	924,252,723.50	574.7674	130
952	79942339.18	63774252.4	63774252.4	988,026,975.90	579.2107	131
953	94357889.79	87150114.5	87150114.49	1,075,177,090.38	583.5838	132
954	111503951	102930920	102930920.4	1,178,108,010.79	587.9541	133
955	125796552.6	118650252	118650251.8	1,296,758,262.62	593.1929	134
956	138748430.5	132272492	132272491.6	1,429,030,754.18	599.3797	135
957	148876413.1	143812422	143812421.8	1,572,843,175	605.8756	136

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
958	158479830.6	153678122	153678121.9	1,726,521,297.85	612.5823	137
959	169427955.1	163953893	163953892.9	1,890,475,190.73	618.755	138
960	179551726.2	174489841	174489840.7	2,064,965,031.41	625.8409	139
961	188274176.7	183912951	183912951.5	2,248,877,982.86	645.8167	140
962	196208067.1	192241122	192241121.9	2,441,119,104.75	676.7237	141
963	205458463.8	200833265	200833265.4	2,641,952,370.17	714.2359	142
964	215069047.4	210263756	210263755.6	2,852,216,125.77	773.7969	143

ACKNOWLEDGEMENT

First and foremost I would like to thank the almighty God for getting me through this.

I am grateful to express my deepest gratitude to my advisor Dr. Ing Dereje Hailu for his guidance and for all his constructive comments. I also would like to appreciate his continuous support and encouragement and to all my instructors in Addis Ababa University School of graduates for the valuable knowledge I have gained.

Great thanks to my families and relatives for their unlimited support .

To all my friends and colleagues who have been helping me in every direction specially WWDSE design and hydrology team greatly appreciated.

Table of Contents

List of Figures

List of Tables

ABBREVATIONS

HEC-RAS	Hydrologic Engineering Center River Analysis System
GIS	Geographic Information System
FEMA	Federal Emergency Management Agency
2D	2Dimensional
1D	1Dimensional
DSO	Dam Safety Organization
ICODS	Interagency Committee on Dam Safety
NWS	National Weather Service
US	United States
HEC	Hydrologic Engineering Center
SMPDBK	Simplified dam break flood forecasting model
DSGL	Dam Safety Guideline
DEM	Digital Elevation Model
Bavg	Average Breach Width
Tf	Time of failure
MAMP	Middle Awash Multipurpose
PMF	Probable Maximum Flood
WWDSE	Water Works Design and Supervision Enterprise
OV	Overtopping
PPG	Piping
WSE	Water Surface Elevation
MA	Middle Awash
BC	Boundary Condition

ABSTRACT

Analysis of dam breach events and the associated flooded area are helpful in the risk assessment of dams. Because sudden failure of dam causes risk of flooding hazard to downstream area. The objective of this study is to model the dam break and map flood inundation areas to be used for estimating the potential consequence of dam breach and emergency planning purpose.

To meet these objectives materials used are HEC-RAS model in conjunction with HECGEORAS an extension of ARCGIS for the case of Middle Awash Multipurpose Dam which is located in Middle Awash sub-basin and DEM 30m of the area was used because of its availability for the extraction of river geometries in HECGEORAS then exported to HECRAS. In HEC RAS dam feature is entered as inline structure, and breach parameters were estimated using the built-in parameter calculator in HEC-RAS and the PMF flow data is entered and unsteady flow simulation is run. The flood map is done with RASMAPPER in HECRAS and Exported to ARCGIS for further work.

Accordingly the Middle Awash dam has been checked for both overtopping and piping mode of failure in HECRAS. The out flow hydrograph result was different for each breach method and breach parameters which in turn affects the inundation area and hazard. It is also different for both mode of failure in HECRAS.

The Middle Awash Dam was checked for overtopping failure with the PMF inflow and for piping starting elevation at the center of dam which occur due to internal erosion of dam material. The peak outflow hydrograph was routed down stream and flood inundation mapping was produced. Failure with overtopping inundate 31000ha area and the failure of the dam with piping inundate 30840 ha. The inundation extent, depth and hazard map of dam breach flood shows that the towns at downstream Melkasedi, MelkaWerer and the cultivated area, infrastructures, the animal and human life in the downstream can be affected with inundation caused due to flood from dam failure.

The flood inundation map helps in estimation of the severity and extent of dam break flood, warning time, consequence classification for emergency action planning and to alert the government body. Therefore, damages that could occur in the surrounding settlements, agricultural areas, on both lives and infrastructure can be minimized and even controlled

1. INTRODUCTION

1.1. Background

From the early period of civilization, construction of dams has been a long established practice. Dams are one of the infrastructures that play a vital role in meeting water demand of the socitiety by providing many benefits like flood control, water supply, irrigation, hydropower, navigation, and recreation. These benefits provided by dams come at a risk due to their potential to fail and cause catastrophic flooding. For this reason, mitigation of this risk is essential by identifying potential failure modes, simulate the potential failure and protect against them for the benefit of the society as floods resulting from the failure of dams produce devastating disasters(Wahl, 2010). This dam failure caused for a number of reasons and its consequence leads to requirement for preparation of dam breach inundation modeling and mapping to identify the flood risk and mitigate the consequences. As the report of FEMA(2013) Dam breach inundation mapping received attention following two significant failure incidents in California (Baldwin Hills Dam in 1964 and Lover Van Norman Dam in 1971).This prompted the state of California to prepare dam failure inundation maps.

So while planning and implementing dams, taking a good care of their safety is currently becoming an important issue. Potential consequences of a dam failure should be understood for planned dams and for those which are already built since the failure phenomena is unexpected an immediate mitigation measures cannot be taken to hinder the breaching process dam For dams under planning stage, one can use dam breach inundation information for classifying the dam (and its hazard class) and this classification can then be used to estimate spillway discharge capacity, seismic parameters, and others. Moreover, for both planned and existing dams, this information is essential for preparedness and emergency action planning related to dam failure (FEMA, July 2013). To put this into practice Different organizations and researchers have contributed their findings in the analysis of dam break and its consequences. They have derived regression equations based on data from historical dam failure events that are used in predicting the breach geometry. These include; the United States Bureau of Reclamation (USBR) and MacDonald and Langridge equations and Development of analytical models (Colorado, 2010).

In our country Ethiopia Construction of dams has been commenced since the first dam was built in 1939 and the dam was constructed on Akaki River to generate hydro-electric power. The dams built so far are being used in order to alleviate the water related problems of the population.

Nowadays there are several number of dams constructed and others are under construction for irrigation, water supply and hydroelectric power mainly. The dams Tendaho and Beles are constructed mainly for irrigation, koka and Gilgel gibe dams are constructed for hydroelectric power demand including the Great Ethiopian Renaissance Dam to boost the nation"s economy (WIKIPEDIA, n.d.).

Middle Awash Multipurpose dam is also one of the dams planned for flood control and irrigation development which is located in Middle Awash sub basin Afar region Ethiopia. The dam is a rock fill embankment type .

In this study, a dam breach analysis and mapping the downstream area inundated by the resulting catastrophic flood shall be investigated for Middle Awash Multipurpose Dam.

1.2. Problem Statement

Understanding Potential consequences of a dam failure for planned dams and for those which are already built are essential because due to various reasons dam failure may occur that can cause devastating disaster to the downstream ecosystem. During construction failure may occur due to unpredictable inflow of extreme hydrologic event to the reservoir, during operation due to malfunctioning of spillway gates and operation errors overtopping may occur, seismic activity, excessive seepage and other hydraulic activities.

Nowadays in Ethiopia many large dams are under construction and some of them have been built in recent years for the purpose of development of irrigation, hydropower or water supply or the combination of these. Following the establishment of dams, the downstream ecosystem is highly changed that huge area is covered with irrigation farms, agro processing plants, new settlements and residence areas of inhabitants living on the farms and factories are formed. All these investments and newly settled inhabitants are highly exposed to flooding and are at risk for damage and death respectively and Middle Awash Dam faiure can cause loss of human life and property of the downstream. The flood from the damage of the dam can also damage towns, villages and its inrastructues on the downstream areas.

In addition causes ruining out of the embankment and its appurtenant structures which constructed with high investment, loss of impounded water that has been accumulated for year and that could irrigate an enormous area of land and flooding of the irrigation farms which is the main purpose of constructing the dam are consequences of Middle Awash Dam Break.

Apart from constructing dams in it is essential for every dam to have dam break analysis and its faiure consequence study. Hence, the intention of this study is to fill this gap.

So this study focuses on the dam break analysis aimed to investigate the p.ossible breaching of the proposed Middle Awash Multipurpose dam and to delineate the area that would be flooded out due to the hazardous wave front.

1.3. Research Questions

The following questions can be raised to initiate this study

- What are the different failure modes that would cause the dam to break and which of these is the most catastrophic?
- What are the breach parameters defining the cross section of the breach and how sensitive are they?
- How much area will be inundated?
- What are the parameter uncertainties that affect the analysis result?

1.4. Objective of the Study

1.4.1. General Objective

The general objective of this study is to model the dam breach , map the resulting inundation, identify and analyze the breach parameters that affect the result of the analysis for the iddle Awash Multipurpose Dam.

1.4.2. Specific Objectives

The specific objectives are:

- Mapping flood inundation areas to be used for estimating the potential consequence of dam breach , confirming the classification and emergency planning purpose
- Simulate the breach process or development applying different breach parameters and breach methods
- To determine dam breach parameters to make realistic estimate of the outflow hydrograph and downstream inundation extent
- To identify the possible failure mode

2. LITERATURE REVIEW

2.1. Dam Failure Overview

Dam is one of the infrastructures that provide several benefits for society but floods resulting from failure produce the most devastating disaster on property and life. The loss of life varies with the extent of the inundation area, the size of population at risk and the amount of warning time available (Tony L, 1998).

Modern dam-safety analysis has been an evolving science since the 1970s. After four notable dam failures occurred in the United between 1972 and 1977, President Carter issued a memorandum directing the review of federal dam-safety activities by a committee of recognized experts. According to this the analysis of dam breaching and the resulting floods became an essential. This helps in reducing loss of life and damage in the downstream flood plain (FEMA, 2013).

The research report of US Dam safety (Tony L, 1998) concludes that the Simulation of embankment dam breach events and their resulting floods are crucial in characterizing and identifying threats due to potential dam failures. This Characterization of the threat to public safety that a dam poses establishes the Hazard Classification of the dam and the associated standard of care to which the dam is held. The Hazard Classification of a dam determines the inflow design flood (IDF), which is the basis for spillway sizing. The Hazard Classification also triggers the requirement to prepare an Emergency Action Plan, requiring preparation of inundation maps which accurately predict dam breach flood depths and arrival times at critical locations.

As the report of (FEMA, July 2013) breaching in embankment dams may occur for a variety of reasons but breaches in embankment dams often modeled as overtopping or piping failures. Flow over an embankment dam (earth or rockfill) usually leads to erosion of material on the downstream slope and failure of the dam. Depending on the composition of the dam overtopping failures can occur very differently.According to a study by Ralston (1987), a small headcut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as the breach 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. 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 downstream main channel. 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 reached bedrock (Gee, 2010) .

FEMA(2013)summarizes that piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse.

There are several possible options to identify the breach initiation time. 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. For breaches associated with a hydrologic event, the initiation can be considered to begin when the reservoir water level reaches a certain elevation or after the water level has exceeded a certain elevation for a specified duration. For fair weather breach analysis, an initiation time should be specified regardless of pool elevation (Gee, 2010). 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, is uncontrolled, and will result in the failure of the dam (FEMA, 2013).

The failure consequence classification is to determine the design requirements for a particular dam. Dams with higher failure consequence are required to be designed to higher standards. Regulatory requirements such as maintenance, operation and surveillance are also based on failure consequence classification (CDA, 2013). In addition (Tony L, 1998) reported that flood inundation mapping is an important tool for municipal and urban growth planning, emergency action planning, flood insurance rates and ecological studies. By understanding the extents of flooding and flood water inundation decision makers are able to make choices about how to best allocate resources to prepare for emergency and to generally improve the quality of life. The resulting flood inundation maps are useful for municipal planning purposes, emergency action plans, flood insurance rate and ecological study (Goodell, 2006).

2.2. Dam Failures

2.2.1. Causes of Dam Failures

Depending on the type of dam and site-specific conditions, a dam may be susceptible to failure from multiple causes. The breach shape and timing of a dam failure varies depending on the type of dam. Concrete gravity dams tend to have a partial breach, as one or more monolith sections formed during dam construction fail, whereas concrete arch dams tend to fail suddenly and. In contrast, embankment dams do not usually have a complete or sudden failure, but rather tend to breach to the point where the reservoir is depleted or to where the breached materials resist erosion, such as at the dam foundation (FEMA, 2013).

Breach analysis for rigid structures is generally straightforward. It typically involves the instantaneous removal of a portion of the structure, or, in some cases, the entire structure. (Colorado, 2010)

Embankment dam failure or breaching may occur for a variety of reasons, breaches in embankment dams are most often modeled as overtopping or piping failures (FEMA, 2013).

2.2.2. Failure Modes

The many causes of dam failures are commonly summarized using five types of failures modes: hydrologic, geologic, structural, seismic, and human-influenced.

Table 2:1. Possible Failure Modes for Various Dam Types

Failure Mode	Earthen/ Embankment	Concrete Gravity	Concrete Arch	Concrete Buttress	Concrete Multi-Arch
Overtopping	X	X	X	X	X
Piping/Seepage	X	X	X	X	X
Foundation Defects	X	X	X	X	X
Sliding		X	X	X	
Overturning		X		X	
Cracking	X	X	X	X	X
Equipment failure	X	X	X	X	X

Source: (Brunner, 2006)

2.2.3. Hydrologic Failure Modes

Hydrologic dam failures are induced by extreme rainfall or snowmelt events that can lead to natural floods of variable magnitude. The main causes of hydrologic dam failure include overtopping, structural overstressing, and surface erosion due to high velocity flow and wave action (FEMA, 2013)

2.2.3.1. Overtopping

Overtopping occurs when the water surface elevation in the reservoir exceeds the height of the dam; water can then flow over the top crest of the dam, an abutment, or a low point in the reservoir rim. Overtopping usually results from a design inadequacy of the dam/spillway system and reservoir storage capacity to handle the resulting flooding event. A failure may also occur when a reservoir‟s outlet system is not functioning properly, thereby raising the water surface elevation of the dam. Overtopping of a dam as a result of flooding is the most common failure mode for embankment dams. During a severe overtopping event, the foundation and abutments of concrete dams may also be eroded, leading to a loss of support and failure from sliding or overturning (FEMA, 2004).

Dam failure begins when appreciable amounts of water begin flowing over or around the dam face and begin to erode the face of the dam. For embankment dams, the failure typically begins at a point on the top of the dam and expands in a generally trapezoidal shape. The water flow through the expanding breach acts as a weir; however, depending on conditions such as headwater and tail water, various flow characteristics can be observed during a breach development including weir flow, converging flow, and channel flow (FEMA, 2013).

|As (Colorado, 2010) Overtopping failures of earthen dams typically begin with head cutting at the downstream toe and advance upstream until the erosion reaches the dam crest and reservoir surface. A dam failure resulting from an embankment slide can also lead to an overtopping type of failure when the slide encroaches upon the high water line. Once the reservoir is connected to the progressing breach, down cutting of the embankment and lateral erosion occur until the breach expands to its final dimensions. The above process assumes a level dam crest. Uneven dam crest surfaces can result in concentration of flow and erosion of the crest itself, accelerating the process of connecting the reservoir to a progressing breach.

2.2.3.2. Structural Overstressing of Dam Components

Higher loading conditions are typically found in dams where the reservoir elevation is increased due to a hydrologic event. While the dam itself may not be overtopped, the surcharge may be increased, overstressing the dam‟s structural components. This overstressing may then result in an overturning failure, sliding failure, or failure of specific components of the dam (FEMA, 2004).

2.2.3.3. Surface Erosion From High Velocity And Wave Action

Surface erosion can occur along earthen spillways, the upstream or downstream embankment slopes, or along other appurtenant structure inlet and outlet channels. Surface erosion is primarily caused by high velocity runoff, reservoir wave action, and ice action. High flow velocities may cause head cutting along spillway sides that can progress towards the spillway crest, eventually leading to a full dam beach (FEMA, 2004).

2.2.3.4. Geologic Failure Modes

Geologic failure modes include piping and internal erosion as well as slope instability and hydraulic fracturing. For embankment dams, geologic failures are typically caused by long-term seepage of water stored in the reservoir; the water seeps through the dam or the foundation and abutments, weakening the embankment over time. If seepage is uncontrolled it may lead to internal erosion or piping of the embankment materials within the dam. A geologic failure may also result from inadequate geotechnical design of the embankment and foundation, inadequate seepage controls, or increased load situations such as the rapid increase or drawdown of water level due to a hydrologic event, landslide, earthquake, or wave action (FEMA, 2013).

2.2.3.5. Piping and Internal Erosion

Piping: Piping occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam embankment or foundation leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Once the erosion reaches the reservoir, it can enlarge and cause catastrophic dam failure (FEMA, 2013).

Internal Erosion: similar to piping, internal erosion is the occurrence of erosion where two adjacent zones interface within the embankment or at the contact between the embankment and foundation. Internal erosion is differentiated from piping in that internal erosion originates internally, whereas piping originates externally when voids of the material into which seepage is flowing are larger than a critical size required to retain the particles, the particles of the up-gradient material can be transported into or through the adjacent material, thereby resulting in internal erosion (CDA, 2013).

2.2.3.6. Structural Failure Modes

Structural failures can occur when there is a failure of a critical dam component. Structural failures may be related to an inadequate initial design, poor construction, poor construction materials, inadequate maintenance and repair, or gradual degradation and weakening over time Additionally, structural failure may be inter-related with other modes of failure for example, structural failure of the main embankment may be related to internal piping, or a critical dam component could fail due to overstressing during a flood event(FEMA, 2013).

2.2.3.7. Seismic Failure Modes

Earthquakes are another important cause of dam failures, especially in seismic zones of the United States. Seismic failures are generally related to either ground movement or liquefaction. Ground movements may cause a dam to shift, settle, or crack into an undesirable configuration that prevents the dam from performing as designed (FEMA, 2013).

Failure Mechanisms Due To Seismic Activities Include:

- Slope instability
- Permanent deformations
- Fissures or cracking
- Differential settling
- Rupture of principal spillway outlet pipeline
- Liquefaction

2.2.3.8. Human-Influenced Failure Modes

Human-influenced dam failure incidents can be related to improper design or maintenance, mis operation including scheduled volume releases, or terrorist acts.

Maintenance:

Misoperation: Misoperation is the sudden or accidental and/or non-scheduled operation of a water retaining element of a dam that releases stored water to the downstream channel in an uncontrolled manner. Mis-operation also includes the deliberate release of floodwater because of an emergency situation, but without the issuance of a timely evacuation warning to the downstream interests. It also includes the inability to operate a gate in an emergency, a condition that could lead to overtopping of the dam and potential breach (FEMA, 2013).

Scheduled volume releases: The release of reservoir volume is a common practice for maintenance purposes, and to provide additional flood storage volume in a reservoir in anticipation of an extreme flooding event. The rapid release of reservoir volume in an upstream dam may result in dam overtopping at a downstream dam, resulting in dam failure. A rapid release of storage volume in a reservoir may also result in a rapid drawdown and a geologic failure. Improper releases of storage volume may result in a dam failure (FEMA, 2013).

Terrorist incidents: Terrorist activities can range from purposeful mis-operation of the dam to physical attacks on the structure itself. Two common scenarios are typically considered when analyzing human-influenced dam failure: rapid failure of spillway gates, and a lowering of the dam crest. For an embankment dam, the rapid lowering of the dam crest could subjugate the dam to overtopping and subsequent erosion (FEMA, 2013).

Table 2:2 : Typical Dam Failure Modes

Hydrologic	Overtopping due to: • Inadequate spillway design • Blocked spillway • Loss of freeboard* due to embankment settlement or erosion • Structural overstressing of dam components Surface erosion due to: • High velocity water • Wave action
Geologic	Piping and internal erosion caused by: • Internal cracking, hydraulic fracture, or differential settlement • Inadequate filters • Outlet pipeline failure • Pipes through the embankment formed by roots or animal/insect burrows Slope instability and hydraulic fracturing: • Load exceeds sliding resistance at base or at joints of structure
Structural	Concrete dam: Failure of critical structural components Embankment dam: Failure of the upstream or downstream face
Seismic	Earthquakes/ground movement; also liquefiable foundations or embankment materials
Human influenced or caused	Misoperation: • Sudden rise in reservoir level causes flow through transverse cracks in embankment • Incidents including gate failures, power interruption
Terrorist activities: • Purposeful MisOperation of the dam • Impact of object that removes part of the dam crest	

(FEMA, 2013)

2.2.3.9. Consequences Of Dam Failure

Despite efforts to improve the safety of the dams, we must still address the concerns of what may happen should a dam fail. The potential loss of life and property damage likely to occur during a catastrophic dam failure may be mitigated through an understanding of the resulting flood wave characteristics and inundated area. The flood results may then be applied to develop emergency response plans and future land use planning (Cameroon T. Ackreman, n.d.)

2.2.3.10. Hazard Potential Classification

In most situations, the investigation of the impacts of failure on downstream life and property is sufficient to determine the appropriate hazard potential rating; however, there may be circumstances where further evaluation is appropriate. For example, the reservoir of a dam that would normally be considered to have a low-hazard potential based on insignificant flooding due to failure may be known to contain toxic sediments, such as may exist in a tailings pond. Therefore, a low-hazard potential rating may not be appropriate and instead a higher standard may be more appropriate to classify the hazard potential (FEMA, 2013). FEMA guidance recommends that the hazard potential rating be based on consideration of the effects of a failure or mis operation during both normal and flood flow conditions. FEMA further recommends that the hazard potential should be based on the worst-case probable scenario of failure or mis operation of the dam.

Table 2:3 FEMA Hazard Potential Classification System Hazard Potential

Hazard Potential	Loss of Human Life	Economic, Environmental and Lifeline Losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable. One or more expected	Yes

Source: (FEMA, 2013)

2.3. Dam Breach Analysis

2.3.1. History of Dam Breach Analysis

In the early 1980‟s, computer programs were developed to analyze the dam breaching process. As indicated by MacDonald & Langridge-Monopolis 1984 that those programs were limited by the accuracy of the breach geometry and failure timing information that was typically used as input. After that MacDonald & Langridge-Monopolis 1984 performed the first systematic analysis of a database of 42 existing dam failures in order to establish empirical relationships relating reservoir/dam dimensions to breach width, timing and peak discharge (Colorado, 2010). Similar statistical (regression) analyses were performed by the USBR 1988, Von Thun and Gillette 1990, Dewey and Gillette 1993 and Froehlich 1995a, 1995b to create their own empirical methods. A few empirical methods were also developed to predict breach peak discharge like the equation developed for the National Weather Service (NWS) Simplified Dam Break Model (SMPDBK) (Wermore, 1984).

2.4. Breach Models

2.4.1. Dam Breach Analysis Tools and Methods

The two primary tasks in the analysis of dam breach are the prediction of the reservoir out flow hydrograph and the routing of that hydrograph through d/s valley, predicting the breach characteristics such as shape, depth, width, rate of breach formation and routing the reservoir storage and inflow through the breach (Wahl, 2010).

There are four critical elements of any breach analysis:

1) Breach parameter estimation (breach size/shape and time of failure),

2) Breach peak discharge and breach hydrograph estimation,

3) Breach flood routing and

4) Estimation of the hydraulic conditions at critical locations. The most commonly used approaches for the required elements of the analysis are described briefly as follows (Colorado, 2010).

2.4.1.1. Comparative Analysis

Comparative analysis 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 (Colorado, 2010).

2.4.1.2. Empirical Methods

Empirical methods are used to predict time to failure and breach geometry, as well as to predict peak breach discharges. The empirical approach relies on statistical analysis of data obtained from documented failures. The four most widely used and accepted empirically derived enveloping curves and/or equations for predicting breach parameters are: MacDonald & Langridge – Monopolis 1984, USBR 1988, Von Thun and Gillette 1990, and Froehlich 1995a, 1995b, 2008. These methods have reasonably good correlation when comparing predicted values to actual observed values (Colorado, 2010).

2.4.1.3. Physically-Based Models

A physically-based 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. When the input is changing with time it may become complex. 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. The National Weather Service‟s BREACH program (NWS BREACH or BREACH) is 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. 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 (Tony L, 1998).

2.4.1.4. Parametric Models

HEC-1, HEC-HMS and HEC-RAS are parametric computer models that estimate the peak discharge and breach hydrographs from dam breaches based on parameters (breach geometry and breach development time) provided by the user. They can also be used to calculate the flood routing of the hydrograph downstream, and, in the case of HEC-RAS, can be used to estimate the hydraulic conditions at critical downstream locations (Colorado, 2010).

2.4.1.5. Hydrologic Models

Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end as indicated in USACE, 1994. Hydrologic routing models provide attenuated flow hydrographs at locations of interest, but do not provide useful information on water surface elevations or flow velocities. HEC-1 and HEC-HMS are the most widely used hydrologic models for dam safety analysis, and both contain a parametric dam breach routine that calculates the breach hydrograph (Colorado, 2010).

2.4.1.6. Hydraulic Models

Hydraulic models are more physically based than hydrologic models since they only have one parameter the roughness coefficient to calibrate. The basic data requirements for hydraulic routing techniques include: flow data, channel geometry, roughness coefficients, and internal boundary conditions. Hydraulic modeling is further subdivided into steady flow analysis and unsteady flow analysis.

HEC-RAS is the most widely used hydraulic model for dam safety analyses in the United States and can be utilized for steady and unsteady flow analyses. The latest versions of HEC-RAS have a parametric dam breach routine that can calculate a breach outflow hydrograph within an unsteady flow simulation (Colorado, 2010).

Another hydraulic model that has been widely used for unsteady flow analyses is the NWS DAMBRK model. The model is based upon the same basic unsteady routing hydraulic principles as HEC-RAS, but DAMBRK was specifically developed for modeling dam failures. The cross-section input requirements for routing dam break floods require the same number of points to represent every cross section, which limits its usefulness (Colorado, 2010).

2.5. Description Of The Model

2.5.1. HEC-RAS

The model package "River Analysis System" (RAS) by the US Army Corps of Engineers – Hydrologic Engineering Center (HEC) includes: a steady flow model and unsteady flow model the consideration of a wide range of hydraulic works, bridges, storage areas and facilities for hydraulic design such as computation of localized scour at the piles of a bridge (USACE, 2010). Due to its capability of describing that wide range of physical processes it has proven very helpful in supporting all phases of river management planning (Pistocchi, n.d.).

HEC-RAS supports both overtopping and piping failure modes with the failure trigger being a target water surface, water surface and duration, or specific time. To model a dam failure in RAS, enter the failure mode, breach size, and breach time. The breach size is defined by a trapezoid and the duration over which the breach occurs. Lastly, RAS allows the user to customize the progression of the breach over the full formation time(Cameroon T. Ackreman).

HEC-RAS has a very easy to use graphical users‟ interface (GUI) and this provides a highly efficient file management, data entry and editing, hydraulic analyses, and tabulation and graphical displays of input and output data (USACE, 2010).

Basically, the software has four 1D river analysis components: steady flow water surface computations; unsteady flow simulation; sediment transport computations; and water quality analysis. HEC-RAS also has a quite a number of options, such as mixed flow regime analysis ,allowing analysis of both sub- and supercritical flow regimes in a single computer run, culvert and bridge routines allowing for multiple openings of different types and sizes, quasi 2-D velocity distributions, and xyz graphs of the river channel system. The stream flow profile follows the basic physical laws: principle of conservation of mass and principle of conservation of momentum. These laws are expressed mathematically and referred as continuity and momentum equations. In unsteady flow, time dependent changes in flow rate are analyzed explicitly as a variable, while steady flow analysis models neglect time all together(USACE, 2010).

Steady flow water surface computation: Steady flow analysis can determine a water surface elevation and flow velocity at a given cross section for a given flow using Manning‟s equation under the assumption of gradually varied flow conditions. This component computes water-surface profiles and energy grade lines in 1-D for a steady and gradually-varied flow. It can handle a single reach, and full network of channels or streams. This component can model subcritical, supercritical, and mixed flow regimes‟ water surface profiles. The flow computations are based on basic physical laws: principle of conservation of mass (continuity equation) and principle of conservation of momentum (momentum equation).Energy losses are evaluated by Manning's equation and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied (USACE, 2010).

Unsteady Flow Simulation: Unsteady flow analysis can be used to evaluate the downstream attenuation of the flood wave, providing a more accurate estimate of flood magnitude and velocity at critical locations this component models 1D unsteady flow through a single or full network of channels or streams. The unsteady flow component was developed primarily for subcritical flow regime calculations (USACE, 2010).

2.5.2. HEC-GEORAS

HEC-Geo RAS is an ArcGIS extension developed by the HEC. This model contains a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. It assists in creating data sets in GIS to extract information essential for hydraulic modeling (Cameron T. Ackerman, n.d.).

After steady or unsteady flow simulation, HEC-RAS results can be exported for processing in the GIS by Geo RAS. The user can read the HEC-RAS results into the HEC-Geo RAS and perform the flood inundation mapping(Brunner, 2006).

HEC-Geo RAS is a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. The HEC-Geo RAS assists in creating datasets (referred to collectively as RAS Layers) in Arc-GIS to extract information essential for hydraulic modeling. The latest release of HEC-Geo RAS supports the extraction of elevation data from Digital Elevation Models (DEMs) in either TIN or GRID format. In this thesis, the GRID format is used (USACE, 2012).

Table 2:4 Summary of Hec_Geo RAS Layers and Corresponding output for HEC-RAS

RAS Layer	Description
Stream centerline	Used to identify the connectivity of the river network and assign river stations to computation points.
Cross-sectional cut lines	Used to extract elevation transects from the DEM at specified locations and other cross-sectional properties.
Bank lines	Used in conjunction with the cut lines to identify the main channel from overbank areas.
Flow path centerlines	Used to identify the center of mass of flow in the main channel and overbanks to compute the downstream reach lengths between cross sections.
Land use	Used to assign flow roughness factors (Manning‟s n values) to the cross sections.
Ineffective flow areas	Used to identify the location of non-conveyance areas.
Blocked obstructions	Used to identify obstructions to flow
Bridges	Used to extract the top-of-road data from the DEM at specified locations

RAS Layer	Description
Inline Structures	Used to extract the weir profile from the DEM for inline structures (i.e. dams).
Lateral structures	Used to extract the weir profile from the DEM for structures the pass flow perpendicular from the main channel.
Storage areas	Used to define the extent of detention areas and develop the elevation-volume relationship from the DEM.
Storage area connections	Used to extract the weir profile from the DEM for connections between storage areas.

(Tariku, 2015)

2.5.3. ARC GIS

The geographical information system, GIS is a system capable of capturing, storing, analyzing and displaying geographically referenced information. Hence, in our context, this model with its HEC-Geo RAS extension shall be used to study the areal distribution of the flood on downstream reach and to delineate the boundary of inundation

2.6. Dam Breach Parameter

In the analysis of dam breach the breach parameter have significant importance b/c accurate prediction of breach parameters is necessary to make reliable estimation of peak outflow and resulting downstream inundation in close proximity to the dam. For realistic modeling of inundation map for a dam breach event, the breach shape, size, location, and timing must be estimated for the considered type of dam and its modes of failures. A key element for calculating a dam breach hydrograph for a specific dam involves estimating the dam breach parameters for dam breach modeling related to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation (Tony L, 1998).

The following definitions are commonly accepted for use in evaluating and selecting dam breach parameters.

• **Breach formation time (also time-to-failure)** – The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached it full geometry.

• **Breach depth (also breach height)** – The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.

• **Breach width** – The breach width is the average of the final breach width, typically measured at the vertical center of the breach.

Breach side slope factor – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

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, is uncontrolled, and will result in the failure of the dam (FEMA, 2013).

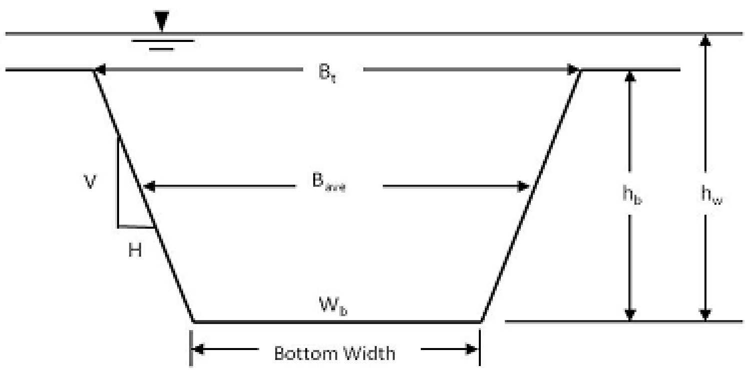


Figure 2:1 Breach Dimension

Table 2:5 Suggested Breach Parameters for Earth Dams

Source	Average Breach Width (ft)	Breach Side Slope (1V:ZH)	Breach Failure Time (hrs)
NWS (1988)	1H to 5H	Z = 0 to 1	0.1 to 2.0
COE (1980)	0.5H to 4H	Z = 0 to 1	0.5 to 4
FERC (1991)	1H to 5H	Z = 0 to 1	0.1 to 1.0
USBR (1982)	3H	N/A	0.00333b
BossDAMBRK (1988)	0.5 to 4H	Z= 0 to 1	0.5 to 4
Harrington (1999)	1H to 8H	Z= 0 to 1	H/120 to H/180
Note: H = Height of water against dam above breach bottom elevation in feet.			

(FEMA, 2013)

2.7. Dam Breach Parameter Estimation

2.7.1. Breach Parameter Estimation

There are various empirical equations available for estimating breach parameters on the basis of dam and reservoir characteristics i.e. dam height, and reservoir‟s volume and other physical characteristics. The most important component of a dam break analysis is the definition of reasonable breach parameters, which are highly difficult to be accurately predicted (Wahl, 1997). When population centers and associated critical sections are located well downstream of a dam, details of the breaching process and the calculated peak discharge may have little effect on the results. In this case, travel time, attenuation, and other routing effects tend to predominate. However, in a growing number of cases, the location of population centers near a dam makes accurate prediction of breach parameters (e.g. breach width, depth, and rate of development) crucial to the analysis. If breach parameters cannot be predicted with reasonable accuracy, more conservative assumptions and associated increased costs may be required(Wahl, 1997). A number of approaches and empirical equations were put to test for this analysis and the following equations were used widely used for various dam breach modeling studies: Froehlich (1995), Froehlich (2008), MacDonald and Langridge-Monopolis (1984), and Von Thun and Gillette (1990) (USACE, 2016) .

Froehlich (1995): The Frohelic 1995 utilized 63 earthen, zoned earthen, earthen with core and rockfill data sets to develop a set of equations to predict average breach width, sideslpes & failure time.

BREACH DEVELOPMENT TIME : The expression developed for the breaching time (t, hr.) is (Froehlich 1995)

$t = 0.00254V^{0.53}\, h^{-0.9}$

BREACH WIDTH :An expression for the breach width (B, m) was also developed (Froehlich 1995)

$B = 0.1803V^{0.32}\, h^{0.19}$. (USACE, 2016)

Froehlich (2008): Froehlich analyzed a total of 74 earthen, zoned earthen, earthen with a clay core wall, and rockfill dams‟ data sets and undertook regression analysis. Following these,the derived equations for the average breach width and breach formation time (USACE, 2016).

$$B_{avg} = 0.27K_o V_w^{0.32} h_b^{0.04}$$

$t_f = 63.2\sqrt{\frac{V_w}{gh_b^2}}$

Where:

B_{avg}= Average breach width (meters)

K_o= Cnstant (1.3 for overtopping failures, 1.0 for piping)

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

h_b= Height of the final breach or dam height from river bed level (i.e. which is taken as dam height) (meters)

g= Gravitational acceleration

t_f= Breach formation time (seconds)

Froehlich states the average side slopes for overtopping failures as 1.0 H: 1V. (USACE, 2016).

MacDonald and Langridge–Monopolis (1984): MacDonald and Langridge-Monopolis analyzed a total of 42 earthen, zoned earthen, earthen with a clay core wall, and rockfill dams" data sets to develop "Breach Formation Factor". Breach Formation Factor is basically a product of the height of water above the dam crest and the volume of water leaving the dam. Then, they related this factor to the volume of embankment material eroded (USACE, 2016).

Following are equations for volume of eroded material and breach formation time:

For earthfill dams:

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

$t_f = 0.0179(V_{erroded})^{0.364}$

For earthfill with clay core or rockfill dams:

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

Where:

V_{eroded} = volume of eroded material from the dam embankment (m^3)

V_{out} = volume of water leaving through the breach (m^3) i.e. storage volume at time of breach plus volume of inflow after breach begins, minus any spillway and gate flow after breach begins.

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

t_f = breach formation time (hours).

V_{out} is not accurately known prior to performing the breach analysis; thus, the first estimate is to set it as the volume of water in the reservoir when breach starts. With this first estimate analysis can be run and one can re-evaluate the initial estimate then make better estimates of the actual volume of water passing through the breach and redo the breach analysis. Basically, this is an iterative process where calculations continue till the estimated volume at the start of each calculation and the end of the analysis converge to some degree.

Here, breach shape is assumed trapezoidal with side slopes of 0.5H: 1V. The bottom width of the breach is then estimated with the following equation (State of Washington, 1992):

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

Where:

W_b = bottom width of the breach (meters)

h_b= height of the final breach or dam height from river bed level (i.e. which is taken as dam height) (meters)

C = crest width of the top of dam (meters)

$Z_3 = Z_1 + Z_2$

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

Z_2 = average slope (Z_2:1) of the downstream face of dam

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

Von Thun and Gillette (1990): Von Thun and Gillette utilized 57 dams from the Froehlich (1987) paper and the MacDonald and Langridge-Monopolis (1984) papers to derive their methodology and equations. With this, the equation for average breach width is presented as (USACE, 2016):

$B_{avg} = 2.5h_w + C_b$

Where:

B_{avg}= average breach width (meters)

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

C_b= coefficient which is a function of reservoir size

Reservoir size (Mm3)	Cb
< 1.23	6.1
1.23-6.17	18.3
6.17-12.3	42.7
>12.3	54.9

Here, breach shape is assumed trapezoidal with side slopes of 1H:1V, except for dams with cohesive soils, where the method stated side slops here should be between 0.5H:1V to 0.33H:1V. As for breach formation time, Von Thun and Gillette used two sets of equations. The first set is only a function of water depth above the breach bottom, and the second set is a function of water depth above breach bottom and average width of the breach.

The first set of equations shows breach development time as a function of water depth above the breach bottom:

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

$t_f = 0.015h_w$ (Easily erodible)

Where: t_f= breach formation time (hours)

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

The second set shows t_f as a function of water depth (h_w) above the bottom of the breach and average breach width (B_{avg}).

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

$t_f = \frac{B_{avg}}{4h_w+61}$ (Easily erodible) (USACE, 2016)

2.8. Dam Classification Systems

The hazard potential classification of a dam, along with its size (height and capacity) classification, is used by State agencies to regulate dam design and dam breach modeling. In USA Common practice among Federal and State Dam Safety agencies is to classify a dam according to the potential consequences of a dam failure on areas located downstream of the dam. FEMA guidance recommends a three-step rating system that defines low-, significant-, and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure. In addition, guidance developed by the USACE incorporates size classification determined by the dam‟s height and storage volume. Some States have additional hazard potential categories such as "extreme hazard" and "very low hazard" and/or have added additional classifications to account for the size of the dam (height and capacity) (FEMA, 2013). The selection of the inflow design flood (IDF), also referred to as the spillway design flood (SDF), according to the assigned hazard potential rating and size classification. The assigned ratings establish the flood events (dam breach scenarios) used in dam breach modeling for design purposes and for use in EAPs (FEMA, 2013).

2.8.1. Dam Hazard Classification

Hazard Potential Classification is a system that categorizes dams according to the degree of adverse incremental consequences of a failure or mis-operation of a dam. The hazard potential classification does not reflect in any way on the current condition of the dam (e.g., safety, structural integrity, flood routing capacity (FEMA, 2004). Downstream Hazard Classification does not correspond to the condition of the dam or appurtenant works, nor the anticipated performance or operation of the dam. Rather, it is descriptive of the setting in areas downstream of the dam and is an index of the relative magnitude of the potential consequences to human life and development should a particular dam fail (Damsafety, 2007). In most situations, the investigation of the impacts of failure on downstream life and property is sufficient to determine the appropriate hazard potential rating; however, there may be circumstances where further evaluation is appropriate. For example, the reservoir of a dam that would normally be considered to have a low-hazard potential based on insignificant flooding due to failure may be known to contain toxic sediments, such as may exist in a tailings pond. Therefore, a low-hazard potential rating may not be appropriate and instead a higher standard may be more appropriate to classify the hazard potential (FEMA, 2013). FEMA guidance recommends that the hazard potential rating be based on consideration of the effects of a failure or mis operation during both normal and flood flow conditions. FEMA further recommends that the hazard potential should be based on the worst-case probable scenario of failure or mis operation of the dam.

Dams are classified according FEMA, 2004

• *High-hazard potential* – Dams assigned the as high-hazard potential classification are those where failure or mis operation will probably cause loss of human life.

• *Significant-hazard potential* – Dams assigned the significant-hazard potential classification are those dams where failure or mis operation are not likely to result in loss of human life but may cause economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns. Significant-hazard potential classification dams are often located in predominantly rural or agricultural areas but may be located in areas with population and significant infrastructure.

• *Low-hazard potential* – Dams assigned the low-hazard potential classification are those where failure or mis- operation are not likely to result in loss of human life and only low economic and/or environmental losses. Losses experienced are likely limited principally to the owner‟s property.

Table 2:6 USACE Dam Hazard Potential Classification system

Category	Hazard Potential Classification		
	Low	Significant	High
Direct Loss of Life	None expected (due to rural location with no permanent structures for human habitation)	Uncertain (rural location with few residences and only transient or industrial development)	Certain (one or more extensive residential, commercial or industrial development)
Lifeline Losses	No disruption of services – repairs are cosmetic or rapidly repairable damage	Disruption of essential facilities and access	Disruption of critical facilities and access
Property Losses	Private agricultural lands, equipment and isolated buildings	Major public and private facilities	Extensive public and private facilities
Environmental Losses	Minimal incremental damage	Major mitigation required	Extensive mitigation cost or impossible to mitigate

Source (FEMA, 2004)

2.8.2. Population at Risk

The potential for loss of life is the primary factor in determining the downstream hazard classification. For purposes of classification, the Population at Risk (PAR) is used to represent the potential for loss of life. This essentially corresponds to the number of people who would have to be evacuated from downstream areas in the event of a dam failure. Population at risk is defined as the number of people who may be present in areas downstream of a dam and could be in danger in the event of a dam failure (Damsafety, 2007).

2.8.3. Property Damage and Economic Losses

Property damages would include damage to inhabited dwellings, commercial and production buildings, agricultural lands and crops, livestock, roads, highways and utilities and the associated economic losses both permanent and temporary. The intent, in considering the potential property damage and economic loss, is to identify the relative magnitude of losses against a broad scale of values (Damsafety, 2007).

2.8.4. Environmental Damages

Consideration of environmental damages would address situations where the reservoir contains materials which may be deleterious to human or aquatic life or stream habitat. This applies to projects such as: domestic and agricultural waste lagoons; industrial waste lagoons; and mine tailings dams where the reservoir may contain trace amounts of heavy metals, chemical residues from ore processing, or large volumes of sediment in a loose or slurry condition. This would apply to streams with fisheries of regional significance where large scale channel scour and sediment deposition are likely to result from a dam break flood (Damsafety, 2007).

2.8.5. Current/Future Development

The downstream hazard classification should reflect the current downstream development and the associated consequences of dam failure. When using the classification it is advisable to investigate the effect that future downstream development may have in increasing the classification and increasing the minimum design standards/criteria at a given dam (Damsafety, 2007).

3. STUDY AREA DESCRIPTION

3.1. Location

The Awash basin is part of the Great Rift Valley in Ethiopia located from 8.50 N to 12N.

It covers the central and northern part of the rift valley and is bounded to the west, southeast and south by the Blue Nile, the Rift Lakes and the Wabi- Shebele Basins respectively. (Behailu, 2004)

The Awash Basin has been traditionally divided into four distinct zones. These are; Upper Basin, Upper Valley, Middle Valley and Lower Valley. (Behailu, 2004)

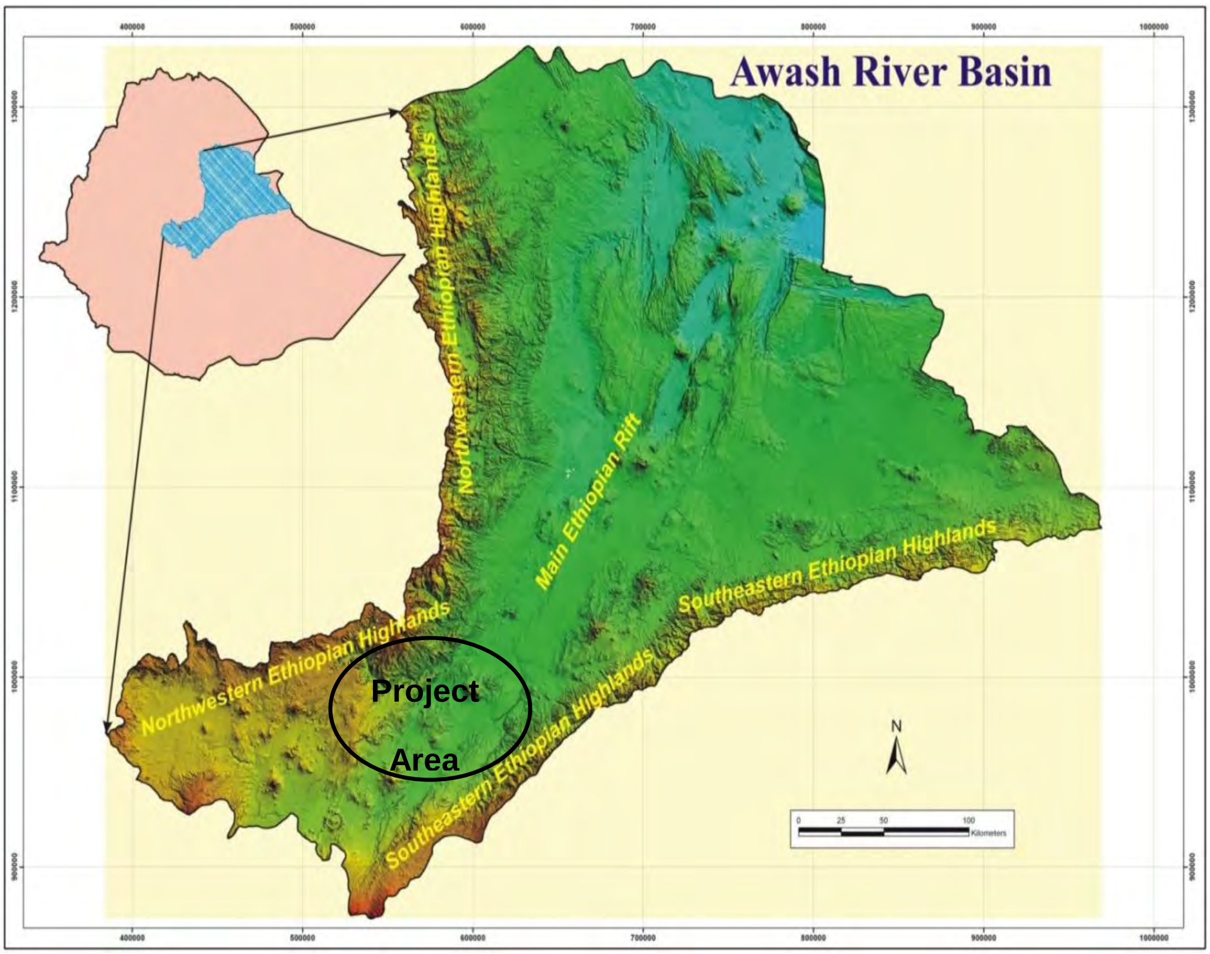


Figure 3:1. Awash River Basin and Middle Awash Multipurpose Dam Project Area(Source: WWDSE dam design feasibility report

3.1.1. Study Project Location

The Middle Awash Multipurpose dam project is found in the Middle Awash valley which is in Afar National Regional State and Oromia National Regional State. As WWDSE(2015) the dam site is located close to Awash town about 200kmfrom the capital, Addis Ababa on Main Awash River just 7km upstream of the Addis Ababa-Djibouti road main bridge on Awash River. The proposed height of the dam is 120m..

The purpose of the middle awash multi-purpose project is for flood control to protect the downstream community and infrastructure and serve to harness the water resources of Awash River for irrigation. (WWDSE, 2015)

Figure 3:2 Study Area

3.2. Topography

The area is prone to flooding by river Awash, which has a tendency to change its course very often (WWDSE, 2007). The altitude of the study area varies from 1598m upstr.a.m.s.l. to 709m.a.m.s.l. at downstream. For reach and floodplain cross-sectional geometry generation and for downstream inundation mapping Study domain DEM is used.(Figure 3:3. Study Area DEM)

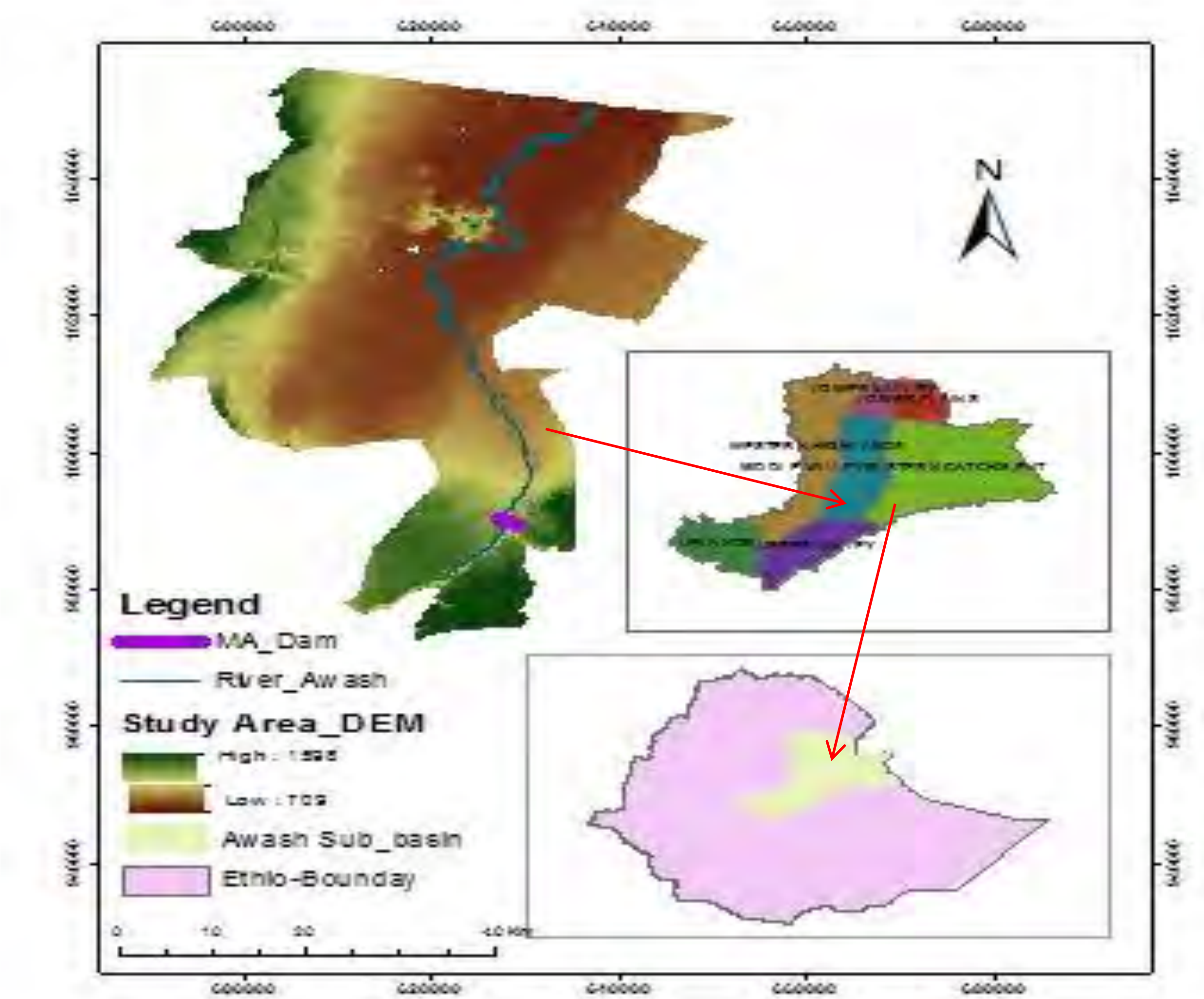


Figure 3:3. Study Area DEM (Source: ETHIO-GIS)

3.3. Climate

The climate of the Awash basin is influenced the Inter-Tropical convergence (ITCZ). This zone of lower pressure makes the convergence of tropical easterlies and the moist equatorial westerlies. The seasonal rainfall distribution within the basin results from the south, bringing the small or spring rains. In June & July it gets its most rains, and then returns south wards during August to October restoring the drier easterly air stream which prevails until the cycle repeated itself in March. .The mean annual rainfall varies from about 1600mm at Ankober in highlands north east of Addis Ababa observatory to 160mm at Asayita on the Northern limit of basin (WWDSE, 2015).

3.4. Catchment characteristics.

The Awash river drains to northerly part of the Rift Valley in Ethiopia from approximately 8.5 ºN to 12 ºN with total drainage area of 112. 211 km^2 (Halcro-, 1989).

3.5. Land Use /Cover

At downstream of Middle Awash dam site Amibara Woreda is found. Based on the population and housing census of 2007, Amibara Wereda has total population of 78,105.Livestock husbandry. state irrigated mechanized farm producing cotton and sesame , Cultivation at , maize, sorghum, Teff, wheat, cotton, sugarcane, sweet potato, mango, onion, tomato, banana, crops .Others are shrubs and some wood land around river bank (WWDSE, 2014).

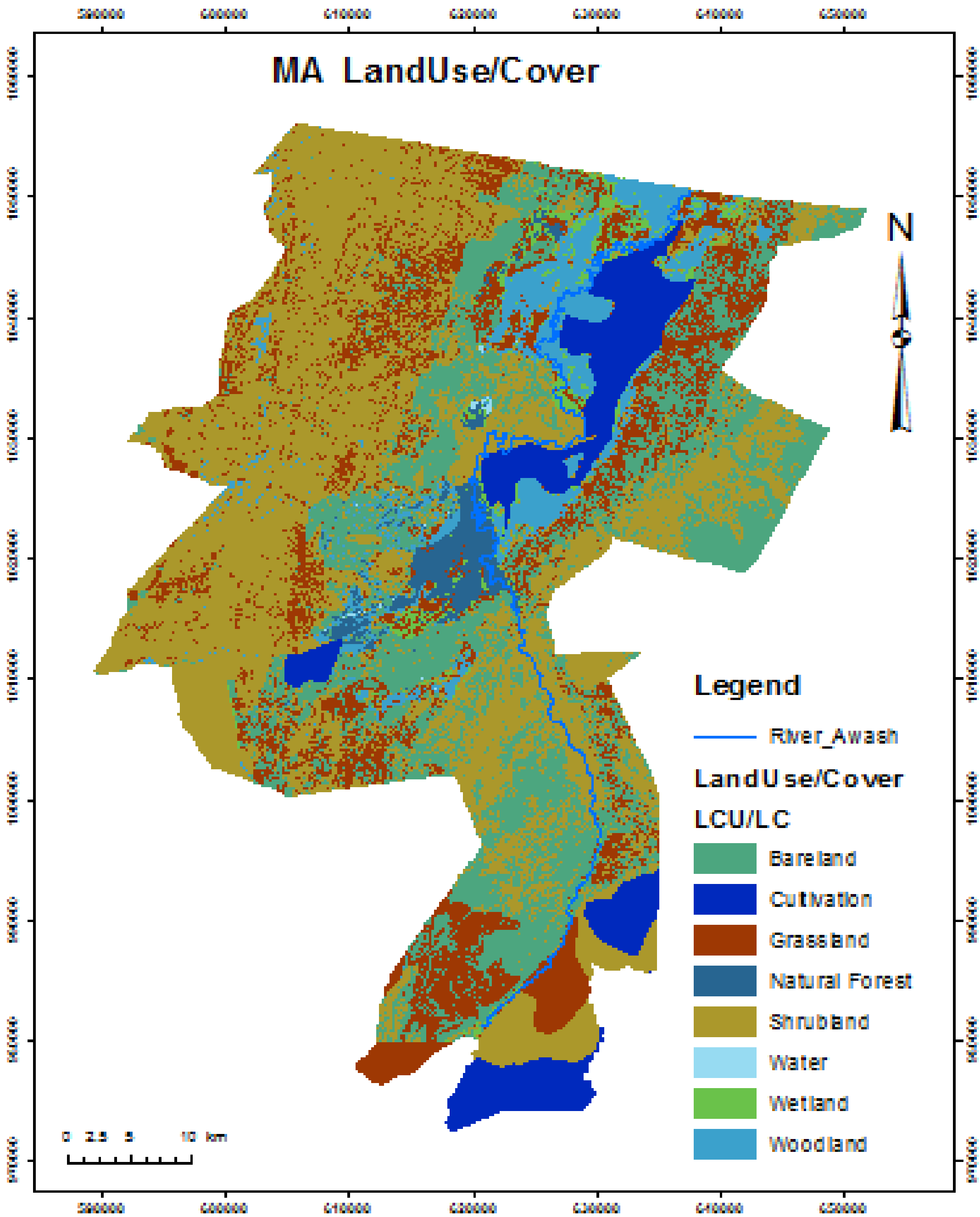


Figure 3:4 Land Use/Cover(Source:WWDSE S.-E. R., 2014)

3.6. The Dam

Midde Awash Muti purpose dam is constructed on Awash river. The dam is a zoned (clay core rock fill) embankment type dam, Its height is 120m at river bed section having the dam crest level at 943 m. The main dam body is designed Rock fill with central clay core, downstream and upstream filter, drainage zone and transitions are provided for protection to drain seepage water and smooth transition from fine to coarse textured dam fill materials (WWDSE, 2015). As WWDSE(2015) the dominant lithologic units around the foundation area are rocks (Rhyolite, Ignimbrite, and Basalt), old alluvial, colluvial deposits, fault breccia and tuff with layers of ignimbrite material. In order to minimize seepage through the foundation, cutoff trench and cement grouting provisions of seepage curtailing measures are considered.

The reservoir is intended to accommodate a total of 501 million cubic meter of water.The maximum water level adapted for Middle Awash Dam design is fixed as the water head over the spillway corresponding to the maximum water level is approximated to be 938.0m (WWDSE, 2015). The reservoir area volume elveation is avaiabe(Annex9).

Table 3:1: Middle Awash Dam Characteristics

Parameters	**Middle Awash Dam**
Dam height	120m
Dam crest elevation	943m
Spillway crest elevation(NPL)	926m
Capacity	501Mm^3
Dam Type	Earth Dam: clay core rock fill dam

(Source WWDSE Design Report)

4. METHODOLOGY

4.1. Data Collection

The Geometrical and other data required to simulate the study dam failure in Hec-Ras were used. Most of the data are generated and most are gathered from Water Works Design and Supervision Enterprise.

A 30 meter resolution DEM map of the study area were available. Due to the difficulty of collecting land surveying because of the fact that the study area is very wide to cover in a short period of time, inaccessibility of river channel and flood plain to some extent because it is covered with bushes and shrubs and in general the financial constraint hinders not to overcome. Hence, a DEM with 30x30m resolution has been used for the study.

These data were imported to ArcGIS for preprocessing of the necessary inputs to HEC-RAS model. The Geometric data are extracted from DEM in ArcGIS with HEC-Geo RAS which ultimately be fed to HEC-RAS for dam break modeling. In addition topographical data, land use map and hydrological data of the area (i.e. Probable Maximum Flood (PMF) inflow hydrograph (Annex 9).

Dam characteristics includes data about name of dam, dam type, dam size, location of the dam, design water storage pool elevation, spillway crest elevation, crest of dam elevation, and height of the dam measured from downstream toe to the crest, and category of the dam and general information of the infrastructure such as city, town, and country area, geographic information, watershed boundary, and others.

Geometric data such as river route, banks, cross sections, and flow paths were extracted from DEM using HEC-GeoRAS and then these data were imported into HEC-RAS and reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported. Data gathered under Downstream includes bank stations, reach stations, downstream developments, cross section plots.

These data were extracted from study area DEM and others are collected from the Water Works Design and Supervision Enterprise (WWDSE), which is undertaking the design of MAMP dam and information from the books, Papers, journals and internet.

4.1.1. Identifying Physical Descriptions of Dam

This step includes the identification of dam height, dam crest width, spillway elevation and width and coefficient of discharge. In this study, data about the physical characteristic and cross section of the Middle Awash Dam is used as an input in HEC-RAS modeling.*(Table 3:1)&Figure(4:1)*

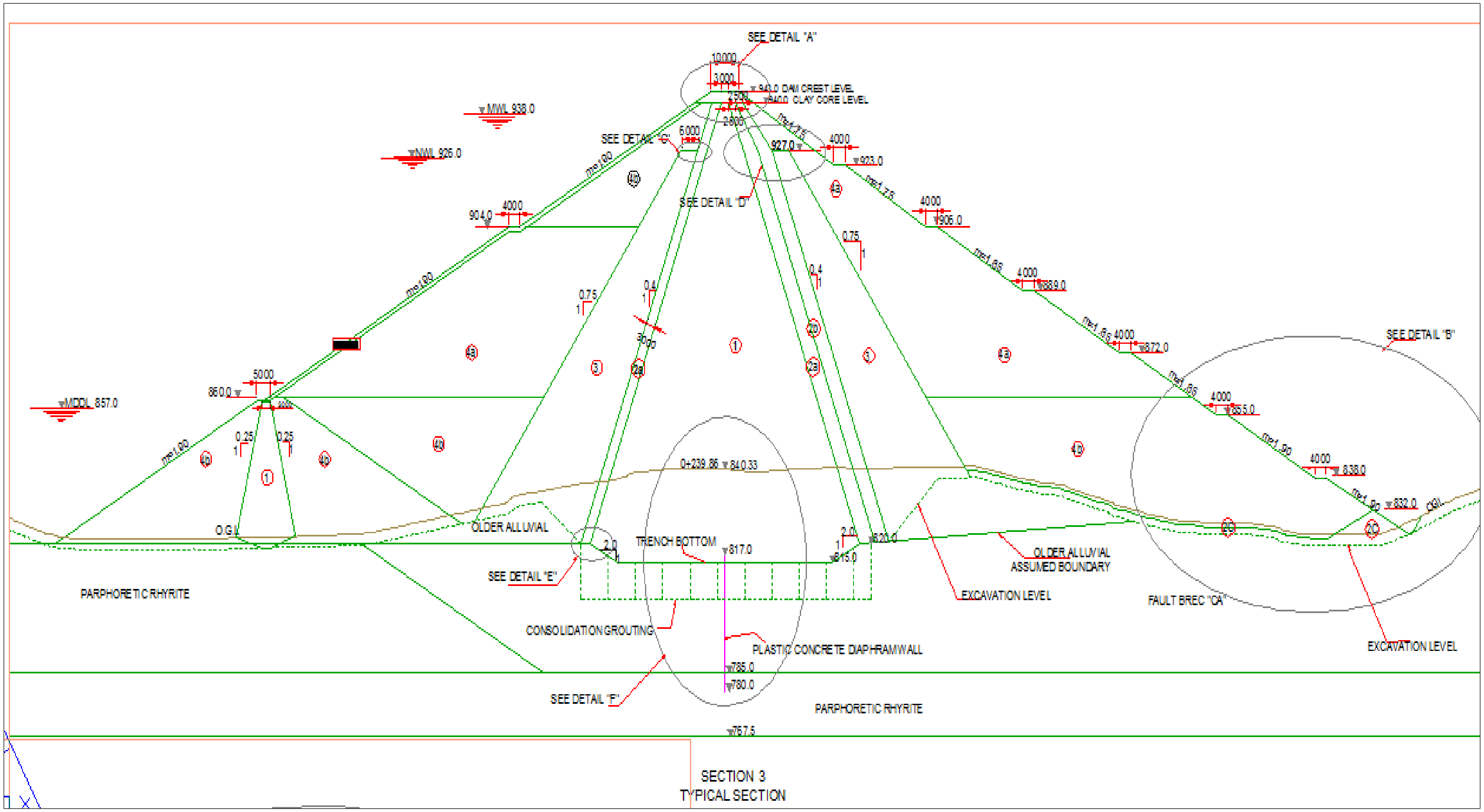


Figure 4:1 Dam Cross Section(Source:WWDSE, Design Report)

ZONING OF DAM

ZONE 1 - IMPERVIOUS CLAY CORE
ZONE 2A - FINE FILTER F1 SEE Detail "E" and "C"
ZONE 2B - COURSE FILTER F2 SEE Detail "E"
ZONE ZONE 2C - HORIZONTAL BLANKET AND TOE DRAIN
ZONE 3 - GRANULAR INNER SHELL
ZONE 4A - ROCK FILL EXCAVATED FROM QUARRY
ZONE 4B - ROCK FILL EXCAVATED FROM SPILLWAY
ZONE 5 - RIPRAP

4.1.2. Inflow Hydrograph to the Reservoir

In order to analyze the worst case of dam breach analysis the inflow design flood is used. The dam to breach inflow hydrographs generated from PMF was used for the breach analysis.

PMF is estimated from Probable Maximum Precipitation (PMP) which is a theoretical maximum precipitation that a given watershed can experience. HEC HMS model was used to estimate the PMF. PMF inflow hydrograph for the catchment at upstream end of of Middle Awash is taken from the Middle Awash hydrological study report of WWDSE(2015) (Annex…9.). As WWDSE(2015) the 10,000 year return period flood peak estimated is 4483.6 m3/s.

4.2. Materials Used

The dam break analysis is done using HECRAS model which is widely used for river analysis with HEC_GEORAS an extension of ARCGIS. The model was selected for the study because of its availability, universality and usability.

To obtain River network and cross sectional geometry and to produce inundation mapping Arc Map GIS 10.0 and HEC-GeoRAS 10.0 tools were used, HEC-RAS 5 Beta Version model for dam break simulation and unsteady flood routing at downstream, Arc GIS study area infrastructure shape files and Google earth for remote sensing of the developed infrastructures like hydraulic structures, towns and current land use information..

4.3. Data Analysis

After getting relevant information and data of the study area the next step is analysis of the collected data. In this thesis study, only secondary data sets were collected and used as input data to HEC-Geo RAS model to generate reach and floodplain cross-section in the study area for input to HEC-RAS Model.

4.3.1. HECGEORAS Model Setup

HEC-Geo RAS is an extension developed by the USACE for use with ArcGIS to process geospatial data to support hydraulic model development and analysis of water surface profile results. The HEC-Geo RAS assists in creating datasets (referred to collectively as RAS Layers) in Arc-GIS to extract information essential for hydraulic modeling. HEC-Geo RAS supports the extraction of elevation data from Digital Elevation Models (DEMs) in either TIN or GRID format. (USACE, 2012)

In this thesis HEC-Geo RAS is used for the development of RAS Layers for extracting cross section data like stream center line, bank lines, flow path and cross section cut lines from Digital Elevation Models (DEMs) in GRID format. In the pre-processor stage, the river centerline was defined using the GIS -Hydro‟ results of the watershed delineated using GIS. Both left and right banks were determined off setting the river centerline to a distance measured on „Google Earth‟ and cross-checked with data collected from site office. Flow path lines, which are centerlines of the flood flowing through the channel, left flood plain and right flood plain, are arbitrarily provided and then adjusted after observing the flood extent.

4.3.1.1. Defining the River Geometry

Using HECGEORAS Ras Geometry Geometric data such as river route, banks, cross sections, and flow paths were extracted from Digital Elevation Model (DEM of 30mx30m resolution) with Ras Geometry in HECGEORAS of the study area Middle Awash and then these data were imported into HEC-RAS. In addition to these data, reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported. (Figure 4.1) The cross sections were located to adequately describe geometric features roughness changes, grade breaks, expansions and contractions, and the numerical requirements for the solution scheme used by HEC-RAS. The cross sections were drawn to remain perpendicular to the expected maximum flood wave flow lines.

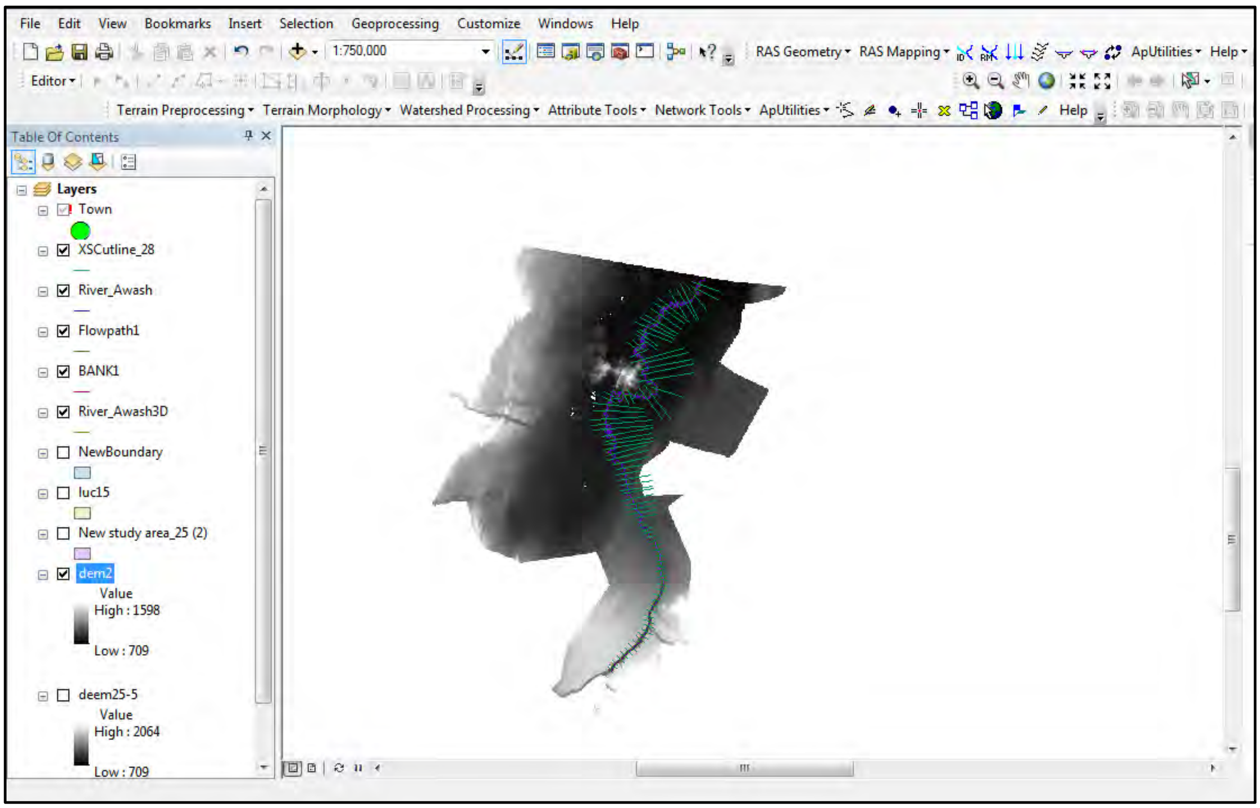


Figure 4:2 . ArcGIS HECGEORAS Window extracted cross sectional data from DEM

4.3.2. HECRAS Model Setup/Data in HECRAS

HEC-RAS has the ability to model flood events and produce water surface profiles over the length of the modeled stream..

4.3.2.1. Geometric Data

After River Geometry is imported from HECGEORAS to HEC-RAS in GIS format by adjusting the unit system to SI unit system the HECRAS project is saved.

Geometric data such as river route, banks, cross sections, and flow paths were imported into HEC-RAS form HECGEORAS. In addition to these data, reach lengths between cross-sections, contraction and expansion coefficients, and other relevant data were also imported.

These exported Cross sections are used to define the shape of the stream and its characteristics, i.e. roughness, expansion and contraction losses, and ineffective flow areas. Cross sections were extracted from the DEM to define the terrain of the expected flood path. They were located in such a way that geometric features such as roughness changes, elevation breaks, expansion and contractions, and numerical requirements for model stability were to be adequately met. The geometry of the Awash River generated from ArcGIS was exported to HEC-RAS. With respect to this, cross sections were spaced at 100m intervals (here interpolations were done in cross sections where the interval is greater than 100m). *(Figure4.2)*

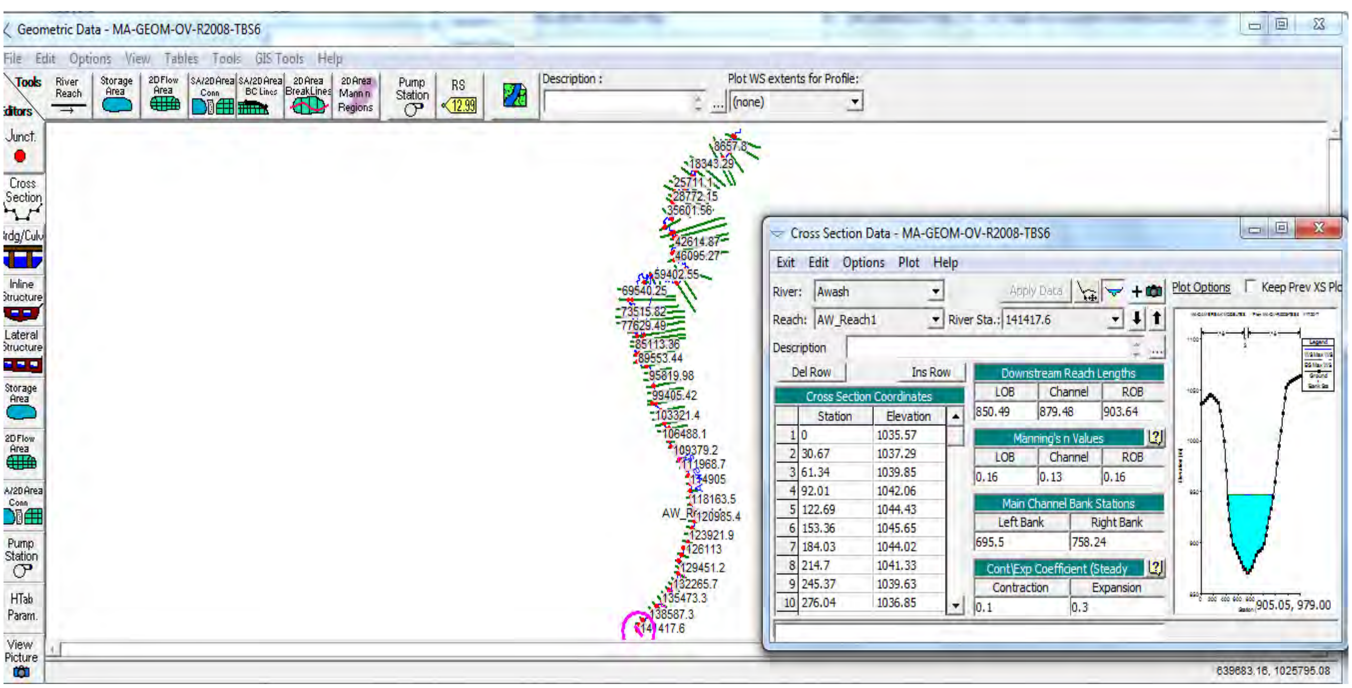


Figure 4:3 Imported Geometric data in HEC-RAS: (a) River and cross section cutline; (b) cross section data editor

4.3.2.2. Breach Model in HECRAS

In Dam Break analysis the dam feature is entered to HECRAS as Inline Structure Either extracted in HECGEORAS and exported to HECRAS or entered in HECRAS with geometry HECRAS window using Inline structure data editor. Then, the dam embankment and spillway crest data were provided using inline structure weir station elevation editor, and breach parameters were estimated using the built-in parameter calculator in HEC-RAS. For this study the Middle Awash Dam geometry is entered with weir/embankment data editor in inline structure editor window.

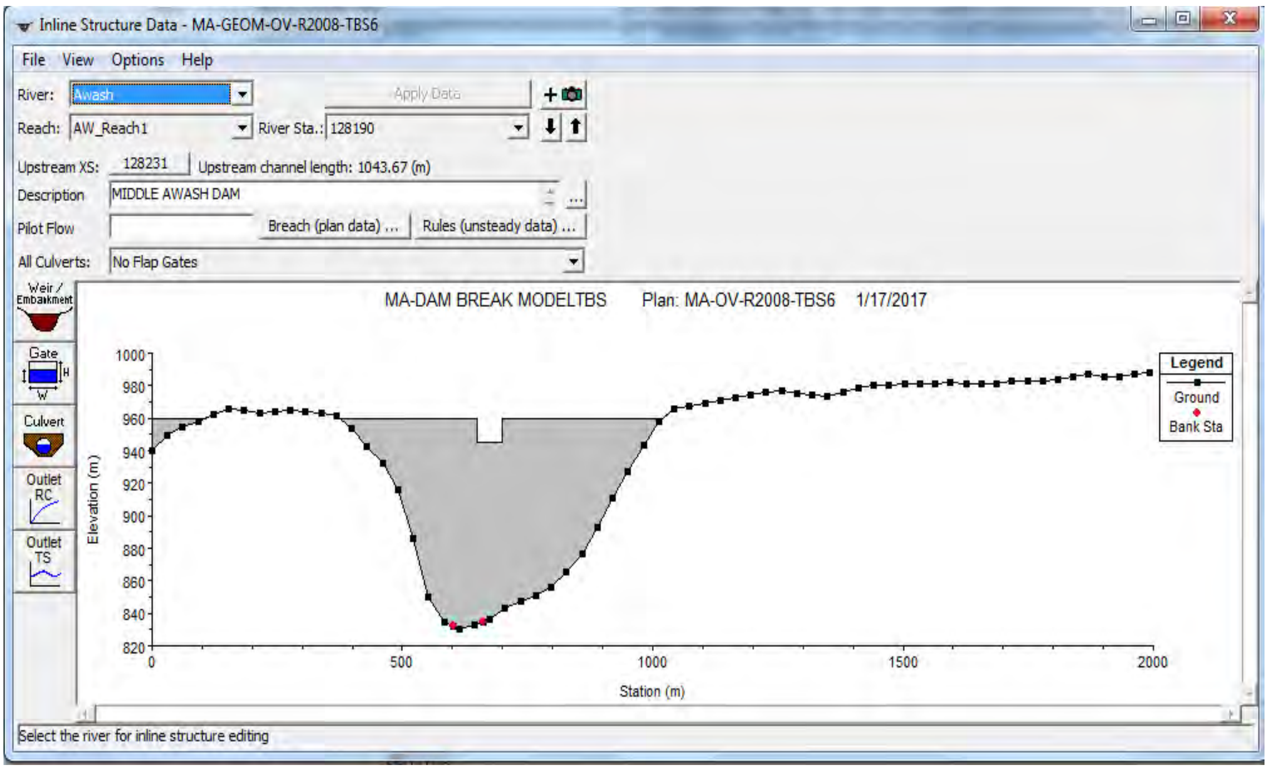


Figure 4:4 Inline Structure (Middle Awash Dam)

In the inline structure data window, breach parameters can be added using the breach plan data editor. The breach parameters were done for each failure mode and breach method. Here, the trigger breach elevation was set to 943.1 m. which is 0.1m above the MWL i.e. 943m. The pool volume at failure was taken from the dam design capacity.

Moreover, the breach progression with time was also defined using the same editor under breach progression menu.

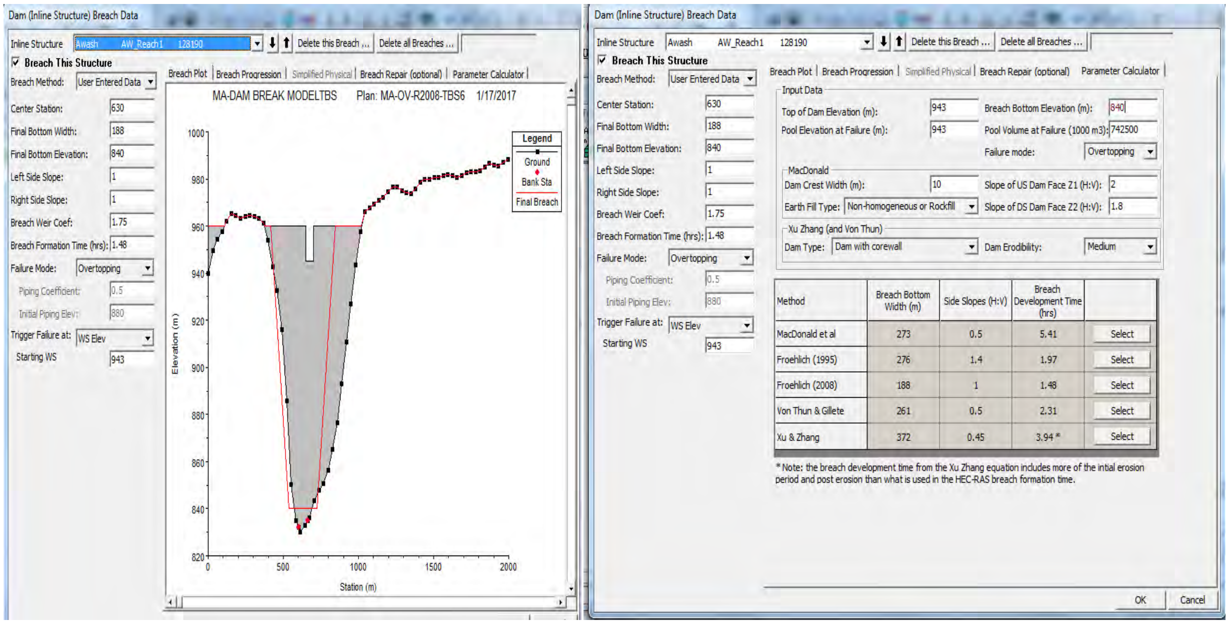


Figure 4:5 Breach Plan Data Editor

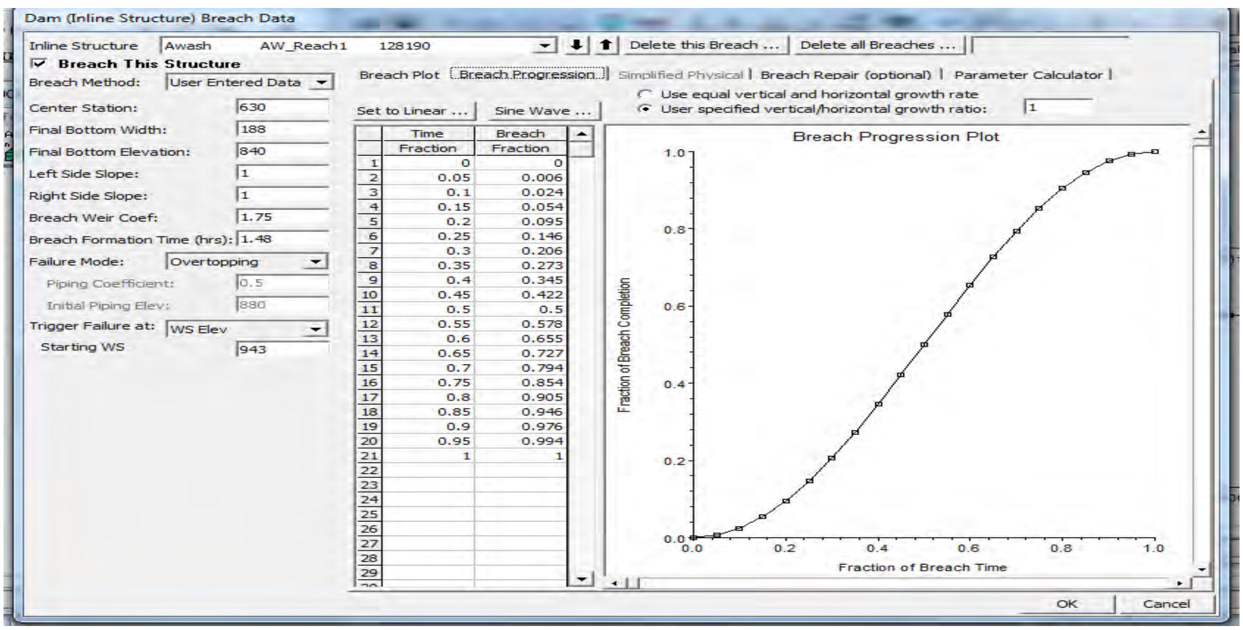


Figure 4:6. Breach Progression with Time

4.3.2.3. Estimating Dam Breach Characteristics

For realistic modeling of inundation map for a dam breach event, the breach shape, size, location, and timing must be estimated for the considered type of dam to determine a dam breach hydrograph, its modes of failures and flood exent.

4.3.2.3.1. Breach parameters

The most important component of a dam break analysis is the definition of reasonable breach parameters, which are highly difficult to be accurately predicted. There are various empirical equations available for estimating breach parameters on the basis of dam and reservoir characteristics i.e. dam height, and reservoir‟s volume and other physical characteristics.

The breach development and breach outflow hydrograph modeling in HEC-RAS demands parameters which define breach geometry such as breach shape, average breach width (B_{avg}), time to failure (t_f), pool elevation at time of failure, weir and orifice coefficients, and breach side slope

A trapezoidal breach which progresses with time was assumed, and the location of the breach was assumed at the centerline station of the dam. The breach width and breach time have a great influence on the forecast of the outflow and the flooded area downstream the dam. The developing time for a breach is defined as the point where dam failure is imminent and end when the breach has reached its maximum size.

The breach development and breach outflow hydrograph modeling in HEC-RAS demands parameters which define breach geometry such as breach shape, average breach width (Bavg), time to failure (tf), pool elevation at time of failure, weir and orifice coefficients, and breach side slope

In this study breach parameters are estimated in HECRAS. Each breach method considers different approach for the estimation of the outflow done in HECRAS because gives different result of the same dam characteristics input. The dimensions of the dam are known from the plan for the breach methods Macdonald, Frohelic (1995) and Frohelic (2008). The breach discharge is the out flow produced from the HECRAS simulation.

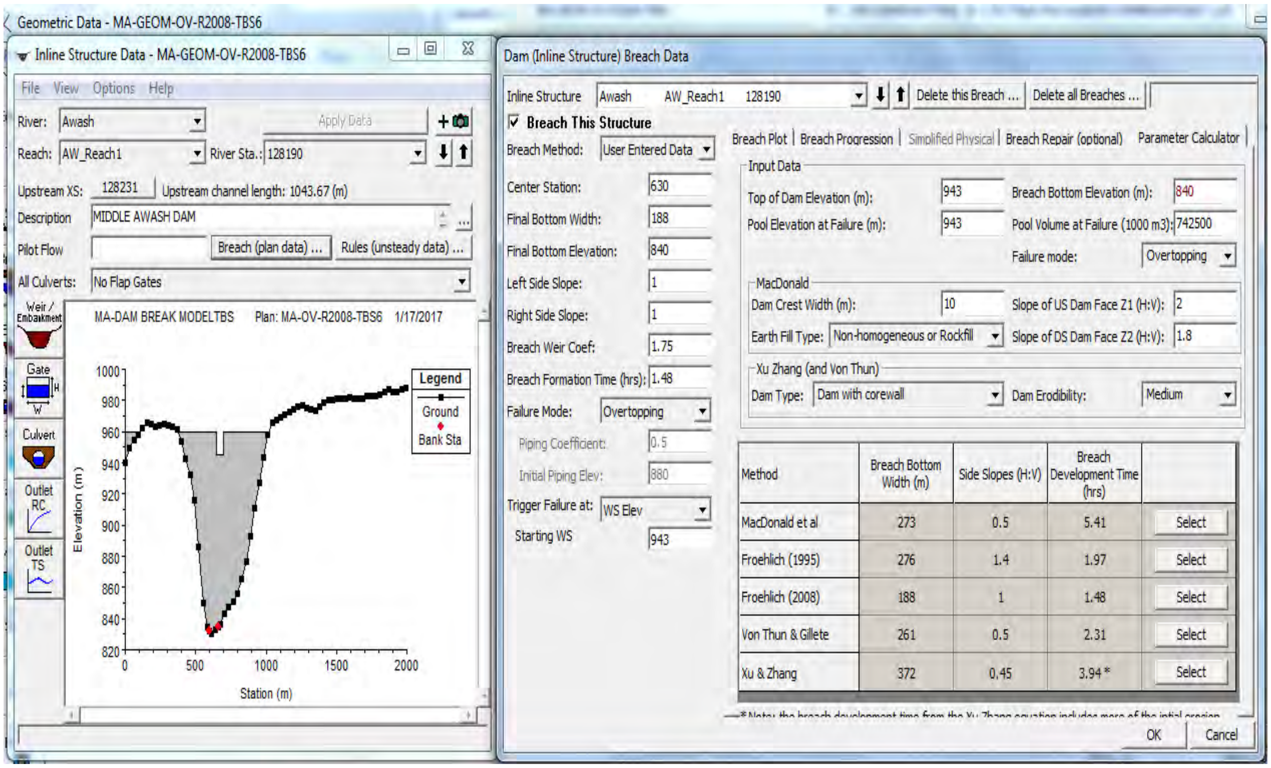


Figure 4:7 Breach Plan Window

With these, the breach parameters for MAMP dam were estimated and the parameters are presented.

4.3.2.4. Unsteady flow data

To model the dam breach process in HEC-RAS, an unsteady flow calculation is performed for 20 days period of simulation selected by the modeler were used.

Boundary condition data must be entered for the unsteady analysis. For the purposes of unsteady flow analysis, upstream boundary conditions are typically flow hydrographs; PMF hydrograph (for hydrological induced failures), or a constant inflow (for sunny day failure analyses).

Downstream boundary conditions can be set to normal depth, a rating curve, a known water surface elevation, or critical depth.

The upstream boundary condition for unsteady flow analysis is taken for PMF hydrograph of Awash River upstream of the proposed Middle Awash Multi-purpose Dam was estimated through rainfall-runoff modeling of the basin.

Here, the downstream boundary conditions are input as a normal depth with friction slope of 0.0001.

Table 4:1: Boundary condition data for MA Dam breach model

Boundary Condition	
Upstream BC	Downstream BC
Flow Hydrograph(PMF)	Normal Depth

4.3.2.5. Running unsteady flow simulation

Once the geometries and flow data entered, HEC-RAS simulates unsteady flow through the channel and flood plain in unsteady flow simulation window. The program is run for Geometry processor, unsteady flow simulation and post processor. Simulation date and time duration is set by the modeler. The simulation time window is run for 20 days and computed for 1 minute interval and mapping, hydrograph and detailed output interval of 1 hour for mixed flow regime.

4.3.3. Flood Mapping

After running the model, the in-built RAS Mapper tool in HEC-RAS used for flood mapping, and other computations related with the flood such as depth, velocity, flood arrival time and duration.

5. RESULTS AND DISCUSSIONS

The dam break analysis was done for Middle Awash Dam and mapping of flood inundation for downstream area to meet the objectives of the study by using HECRAS MODEL. The major inputs for dam breach analysis are data such as geometry and properties of the dam, reservoir information, inflow hydrograph and breach characterstics. There are different dam failure modes (Table2:2). Middle Awash dam is of rock fill dam with central clay-core material, and the failure modes considered for analysis in HEC-RAS are piping (i.e. under sunny day condition) and overtopping failure(i.e. under hydrological induced condition). After simulation it is reaveld that Middle Awash Dam can fail for both overtopping for event of peak PMF and piping with erosion of embankment material.. During dam breach with overtopping the unsteady flow profile is observed through the downstream channel so the unsteady flow analyses were used for dam breach simulation. Dam Breach development is simulated for different breach methods in HEC-RAS Macdonald, Frohelic(1995) and Frohelic(2008) and their breach outflow depending on the breach parameter routed through its downstream channel is and the results of the HEC-RAS model were used to map the inundation limits.

5.1. River Cross Section

This river cross section is the result of river geometry analysis done in Hec-Geo Ras and Exported to HecRas model for further work.

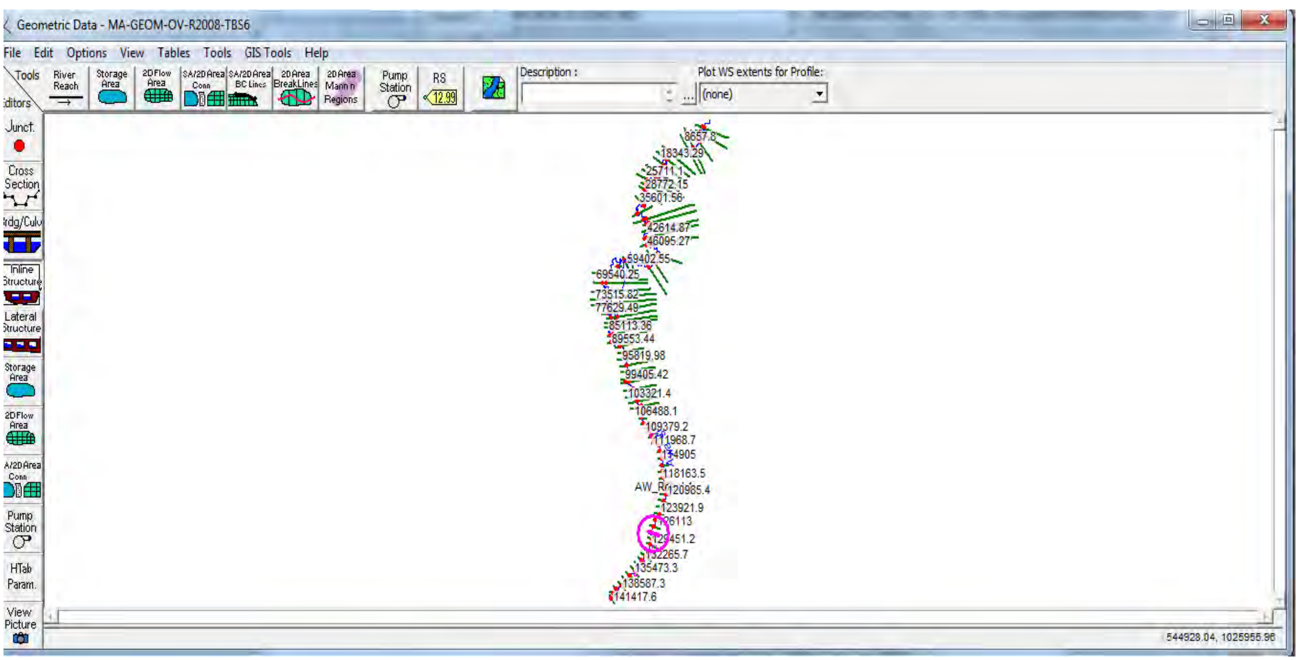


Figure 5:1 Middle Awash River cross Section exported to HecRas

5.2. Dam breach parameter estimation

HEC-RAS allows the modeling of the breach development by entering key data and assumptions regarding the dam, the reservoir and the breach characteristics. The breach parameters breach formation time, bottom width of the breach and breach side slope.

Breach parameters and related dam break peak outflow have been defined, using the three methods Macdonald, Frohelic(1995) and Frohelic(2008) done in HECRAS. The simulation for dam break model was done in Breach plan data window in which the dam characteristics and dam breach parameters are entered. (*Figure5.2*)

The Dam Breach Model is simulated for each breach method, breach parameters and mode of failure for the given dam characteristics *(Table 3.1).*

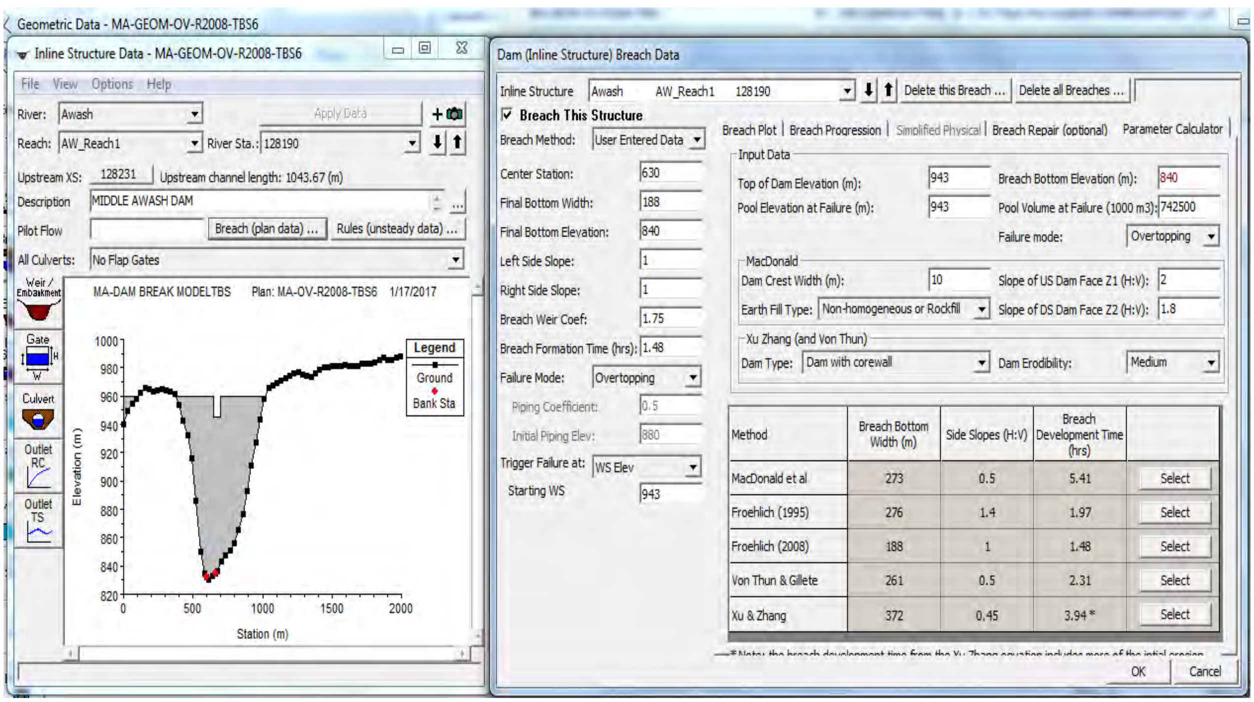


Figure 5:2 Breach Plan window

In dam break analysis first estimate the characteristics of the breach. Once the breach characteristic is estimated then HECRAS used to compute the outflow hydrograph of the breach and perform the downstream routing. Here are the breach parameters estimated in HECRAS for both overtopping piping mode of failure mode (Table 5.1&5.2).

The dam breach parameters Breach Bottom Width, Breach side slope and Breach formation time are different for each breach method during both overtopping and piping mode of failure.

Breach parameters are different for each breach method. Froehlich (1995) have a greater breach bottom width, side slope and breach time as compared with the other methods for both overtopping and piping mode of failure. The breach parameters estimated with Macdonald are equal for both overtopping and piping modes. For Froehlich (1995) and Frohelic2008) parameters with overtopping mode are greater (Table 5.1&5.2).

Table 5:1 Estimated Dam breach parameters for overtopping

Breach Parameters	Macdonald	Froehlich (1995)	Froehlich(2008)
Breach Bottom Width(m)	273	276	188
Side slope (H:1V)	0.5	1.4	1
Breach Development Time (hrs.)	5.41	1.97	1.48

Table 5:2 Estimated Dam breach parameters for Piping

Breach Parameters	Macdonald	Froehlich(1995)	Froehlich(2008)
Breach Bottom Width(m)	185	172	126
Side slope (H:1V)	0.5	0.9	0.7
Breach Development Time (hrs.)	4.79	1.60	1.22

5.3. Dam Breach Out Flow Hydrograph and Downstream Routing

The two primary tasks in the hydraulic analysis of a dam breach are the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley The flood hydrograph is a plot of discharge versus time. Flood routing is the term used to describe the movement of a flood wave as it traverses a reach of channel.

The dam breach outflow hydrograph for the Middle Awash embankment dam resulted from the HEC-RAS dam break modeling , The peak breach outflow from the hydrograph for each breach method MacDonald, Froehlich(1995) and Froehlich (2008) estimated geometries for both Overtopping and Piping modes of failure for the simulation period of 20 days to complete*(Fig 5.3-5.8).*

MacDonald: - For Mac Donald dam breach parameter peak outflow of 62888.9m3/s and 4.63Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of72688m3/s and 4.66 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

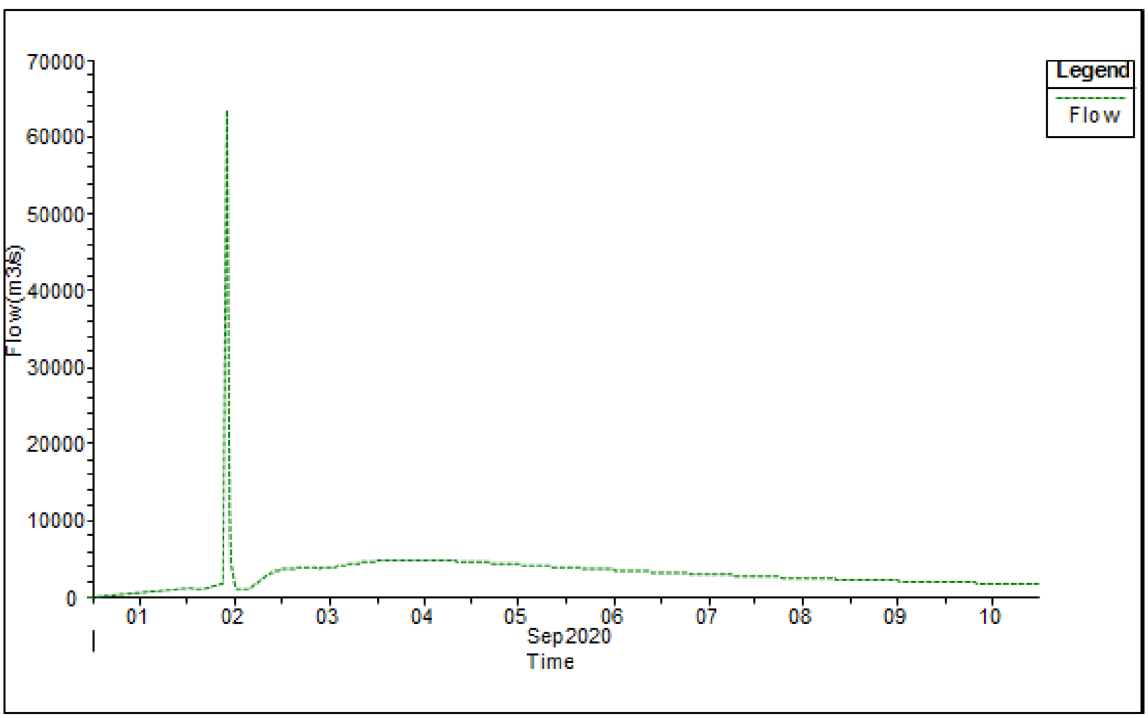


Figure 5:3-Breach Outflow hydrograph Middle Awash Dam MacDonald Dam Breach Parameters (Overtopping)

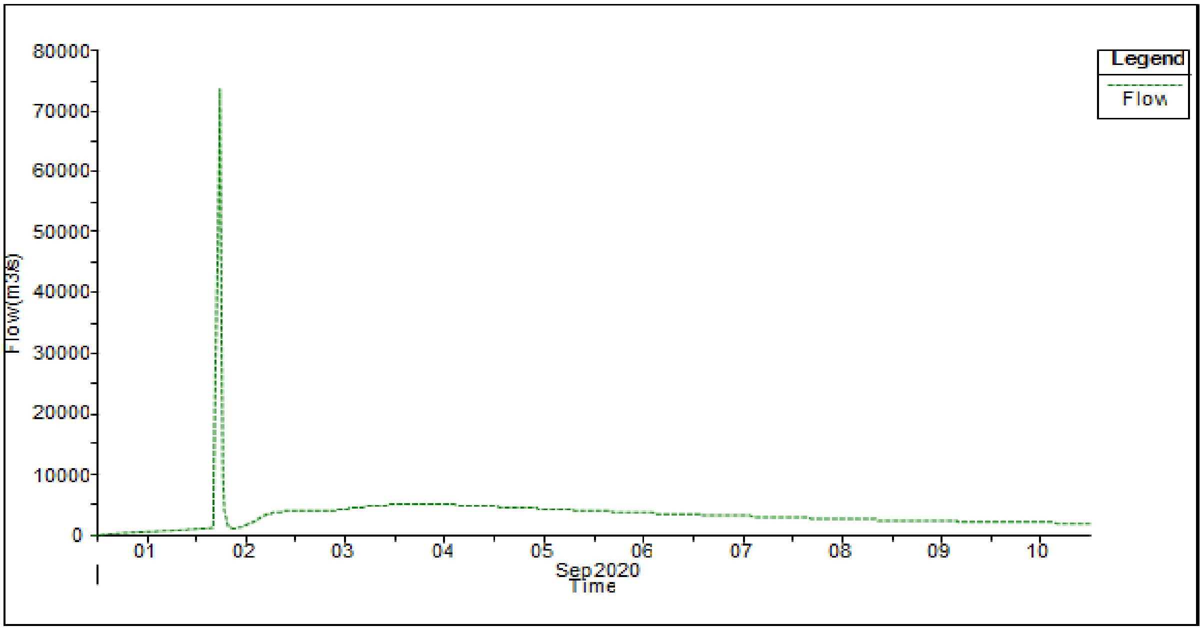


Figure 5:4 Breach outflow hydrograph Middle Awash dam MacDonald Dam breach parameters (PIPING)

Froehlich (1995): - For Froehlich (1995) dam breach parameter peak outflow of 111663m3/s and 4.74Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of 118039m3/s and 4.76 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

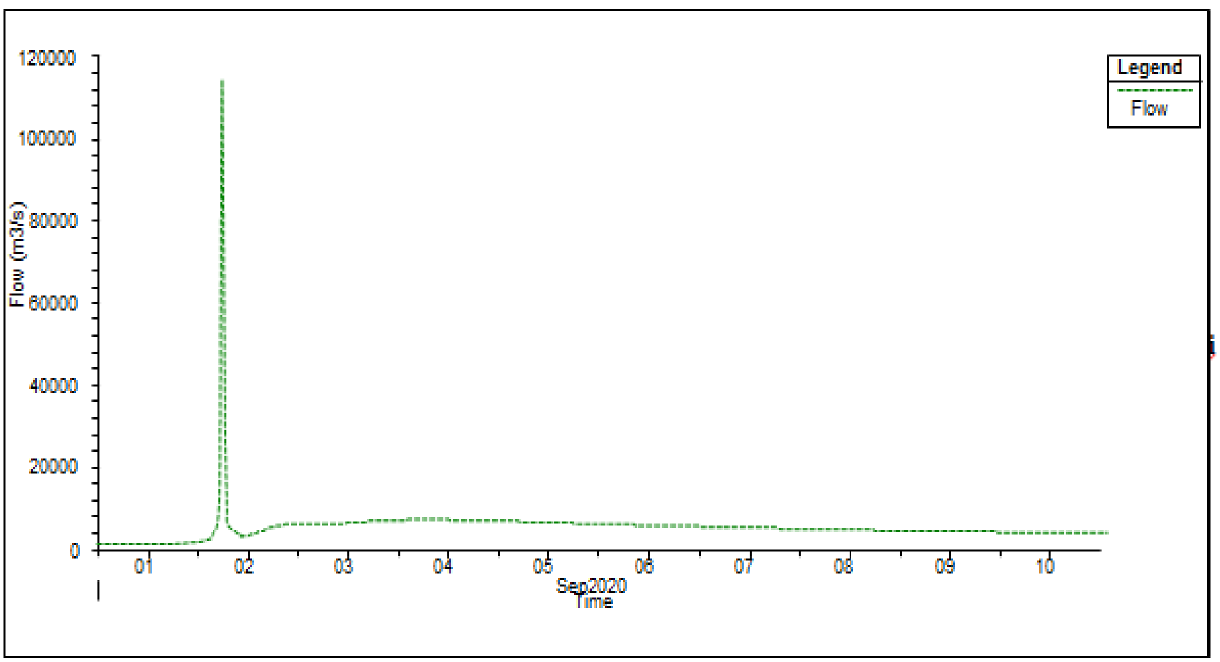


Figure 5:5-Breach Outflow hydrograph at Middle Awash Dam Froehlich (1995) Dam Breach Parameters (OverTopping)

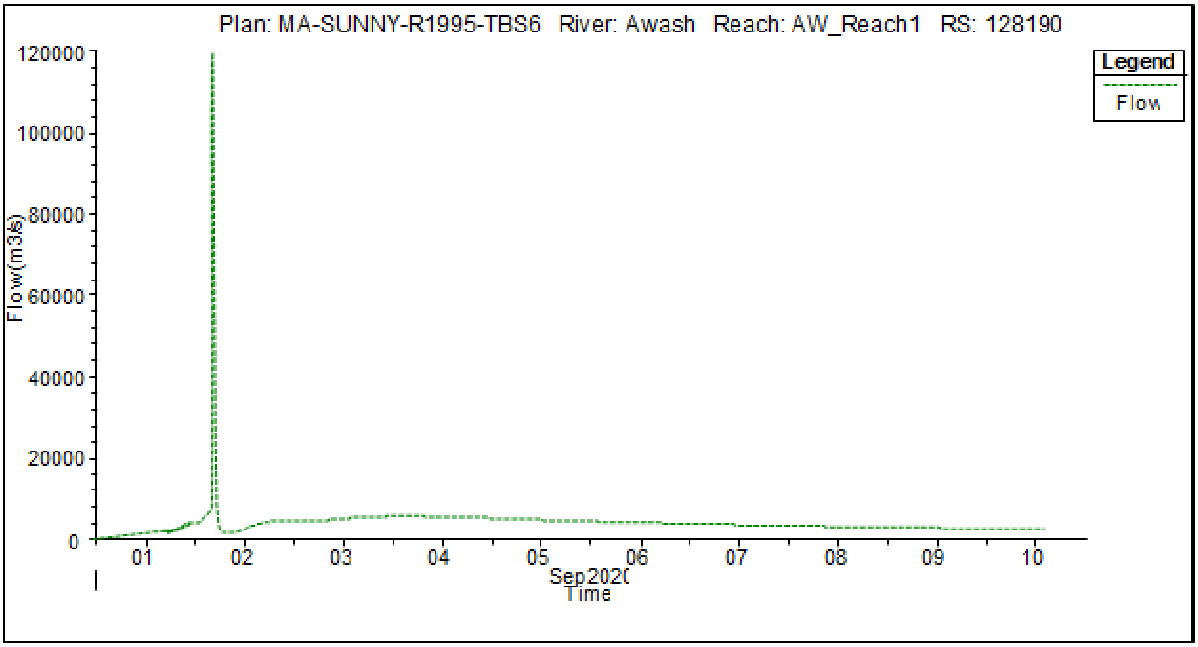


Figure 5:6-Breach Outflow hydrograph at Middle Awash Dam Froehlich (1995) Dam Breach Parameters (Pipng)

Froehlich (2008): - For Froehlich (2008) dam breach parameter peak outflow of 104814m3/s and 4.71Bm3 of water were estimated to be released under overtopping failure of MAMP Dam and for piping breach peak outflow of 100045m3/s and 4.7 Bm3 of were estimated to be released for the breach parameters done in HEC RAS.

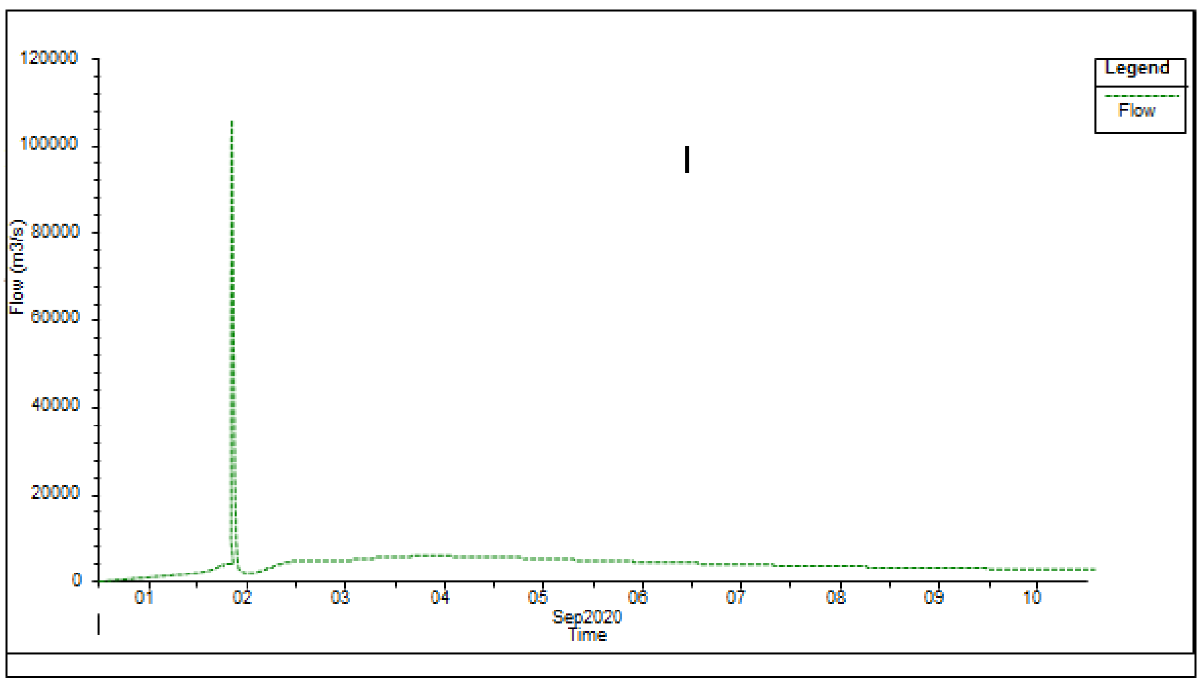


Figure 5:7-Breach Outflow hydrograph at Middle Awash Dam Froehlich (2008) Dam Breach Parameters (Overtopping)

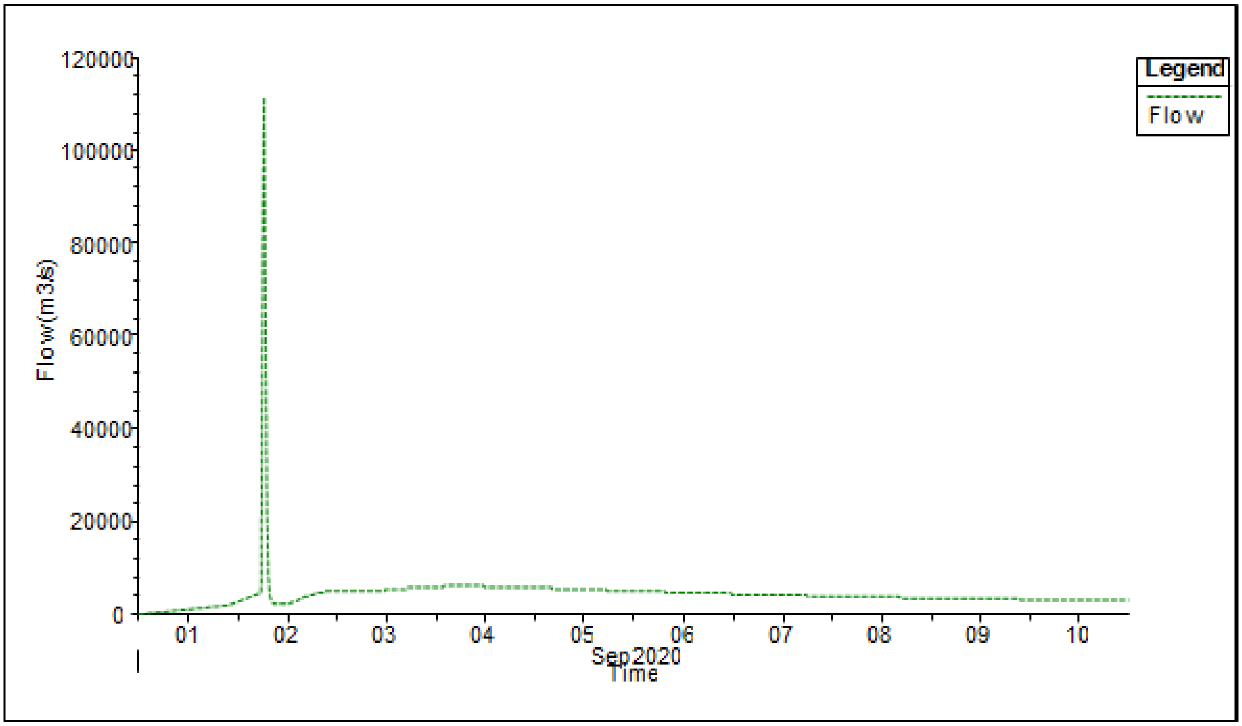


Figure 5:8-Breach Outflow hydrograph at Middle Awash Dam Froehlich (2008) Dam Breach Parameters (Piping)

The dam failure occurs for both Overtopping and Piping mode simulation under the three methods. The hydrograph is routed downstream at selected cross sections (Table 5.3) and the flow is different through downstream reach.

Table 5:3 List of Outflow value from MAMP Dam at selected towns and cross sections

	MacDonald				Froehlich(1995)				Froehlich(2008)			
	Overtopping		Piping		Overtopping		Piping		Overtopping		Piping	
	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume B(m^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)	Flow (m^3/s)	Volume (Bm^3)
0m from the Dam(at the dam)	62888	4.62	72688	4.65	111663	4.74	118039	4.76	104814	4.71	100045	4.7
10km from Dam	38030	4.61	46607	4.64	49259	4.64	55973	4.64	49490	4.64	25195	4.63
MelkaSedi(44km from Dam)	19141	4.6	21667	4.62	18480	4.6	21245	4.61	18414	4.6	21172	4.61
MelkaWerer(80km from Dam)	3892	4.3	3957	4.38	3888	4.3	3954	4.4	3888	4.3	3954	4.4
120 km from Dam	3830	4	3905	4.1	3827	4.0	3901	4.1	3826	4	3902	4.1
End of the XS(126km)	3825	3.98	3899	4	3821	3.97	3896	4.0	3820	3.97	3896	4.0

Overtopping flow simulation of the three breach methods for indicates that the flow for Frohelic 1995 reaches greater as going down stream but shows some little difference with frohelic 2008 *(Figure 5:9)*. Macdonald shows low flow through downstream reach as compared to others.

Piping flow simulation of the three breach methods for indicates that the flow for Frohelic 2008 reaches greater as going down stream but shows some difference with frohelic 1995*(Figure 5:10)*. The same as overtopping macdonald shows low flow through the downstream reach as compared to others.

For both overtopping and piping Overtopping Shows greater flow than piping through downstream reach for the three methods *(Figure 5:11)*.

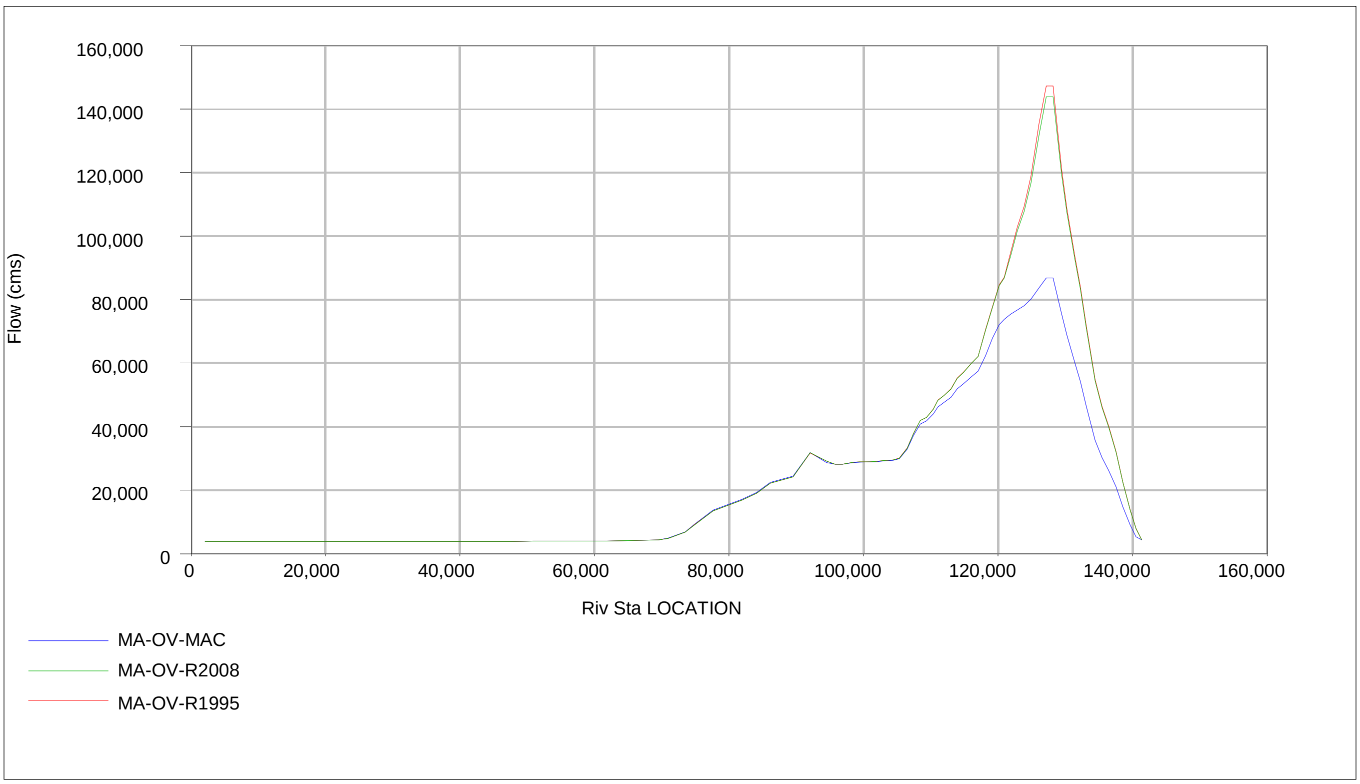


Figure 5:9 Flow of the three breach methods throughthe river station- overtopping

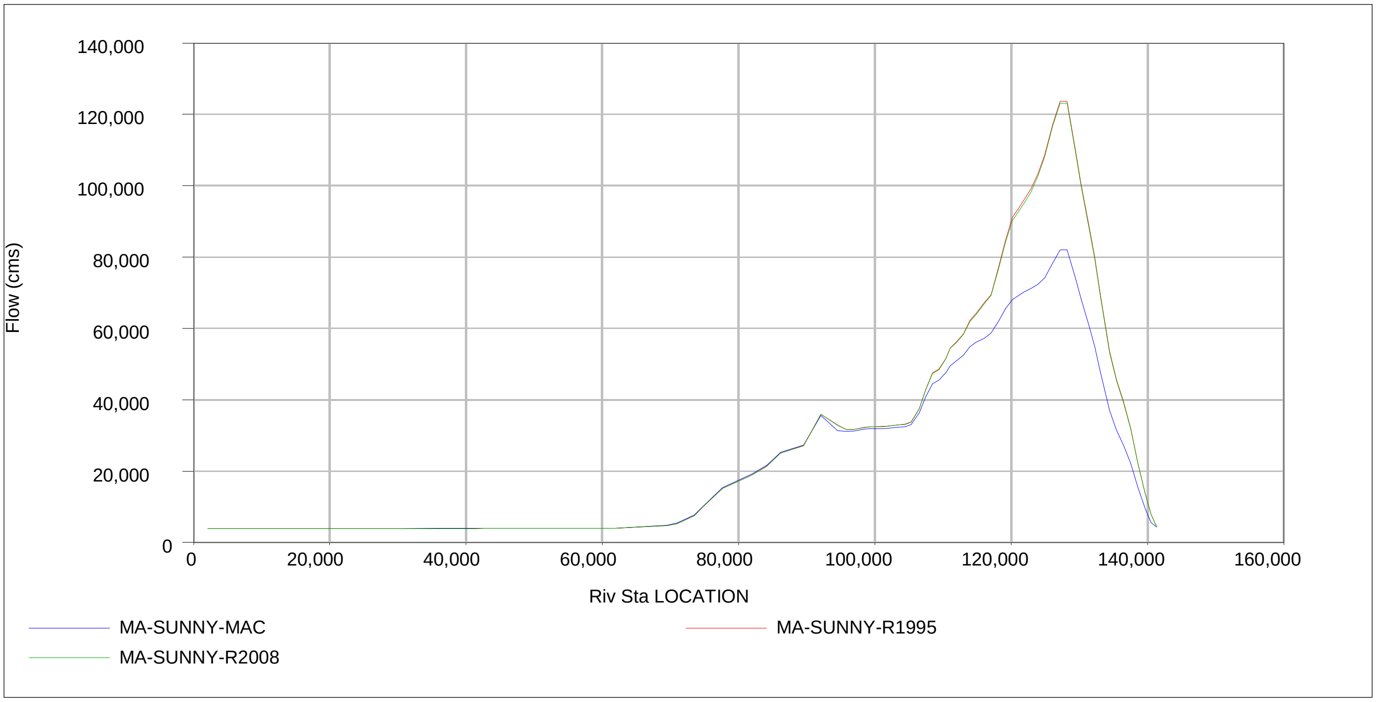


Figure 5:10 Flow of the three breach methods throughthe river station- piping

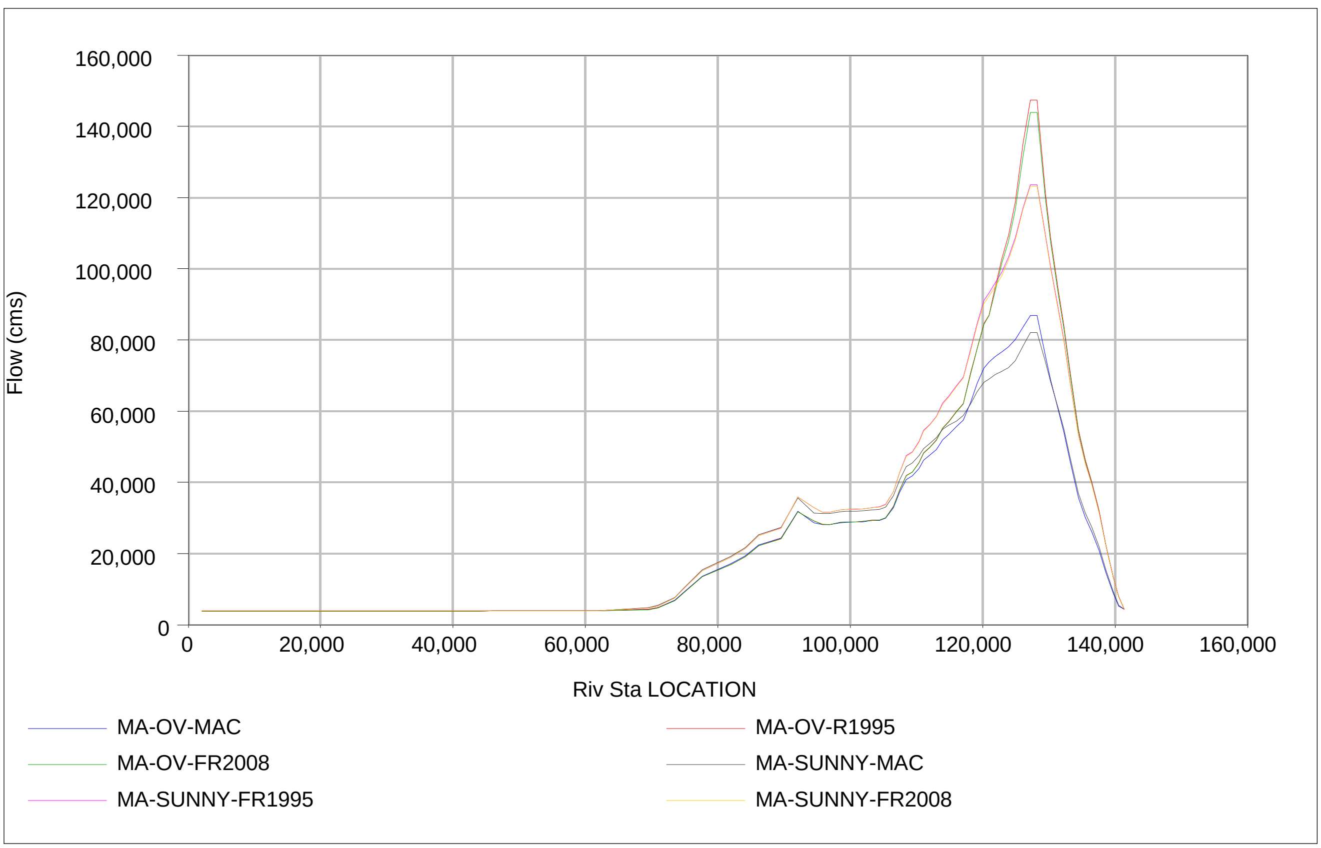


Figure 5:11 Flow of the three breach methods throughthe river station- overtopping&Piping

- After the dam failure simulation of unsteady flow routing the dam break cross section view and the water surface profile for both overtopping and piping (Figure5:12,5:13, 5:14 &5:15).

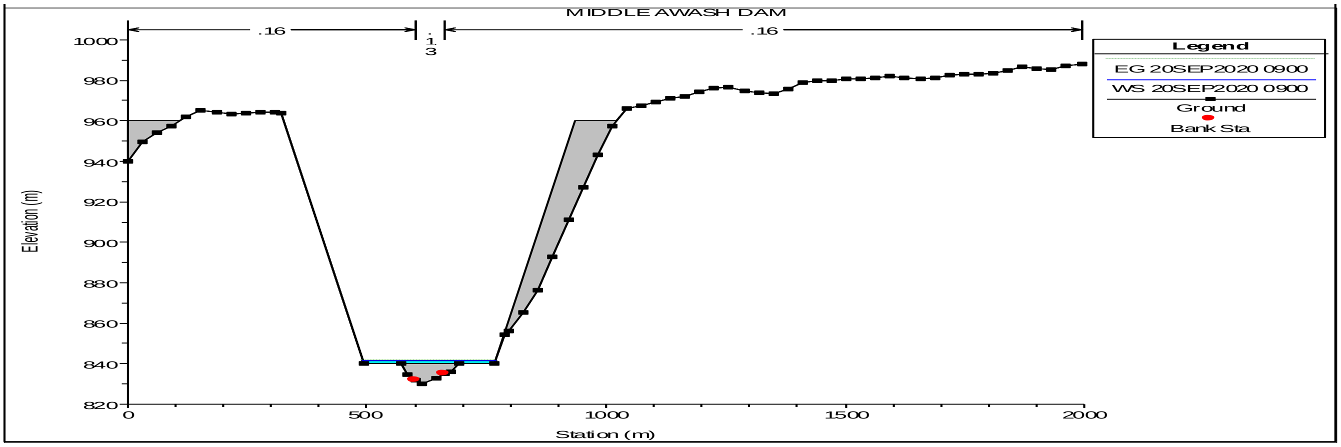


Figure 5:12 Sample Dam Break Cross Section View for Overtopping (Froheilic1995)

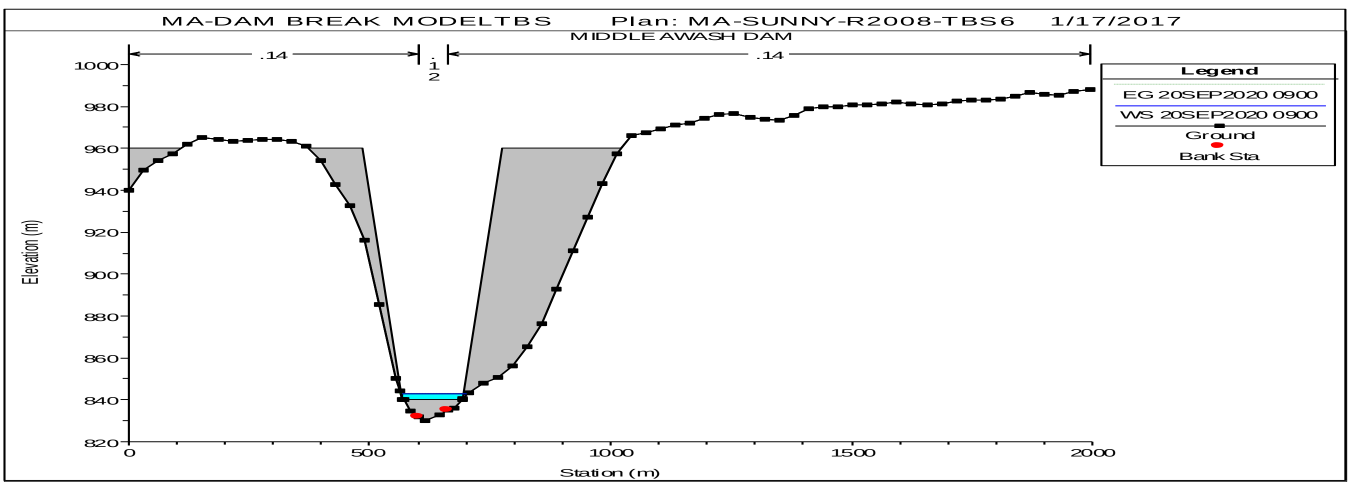


Figure 5:13 Sample Dam Break Cross Section View for Piping (Frohelic 2008)

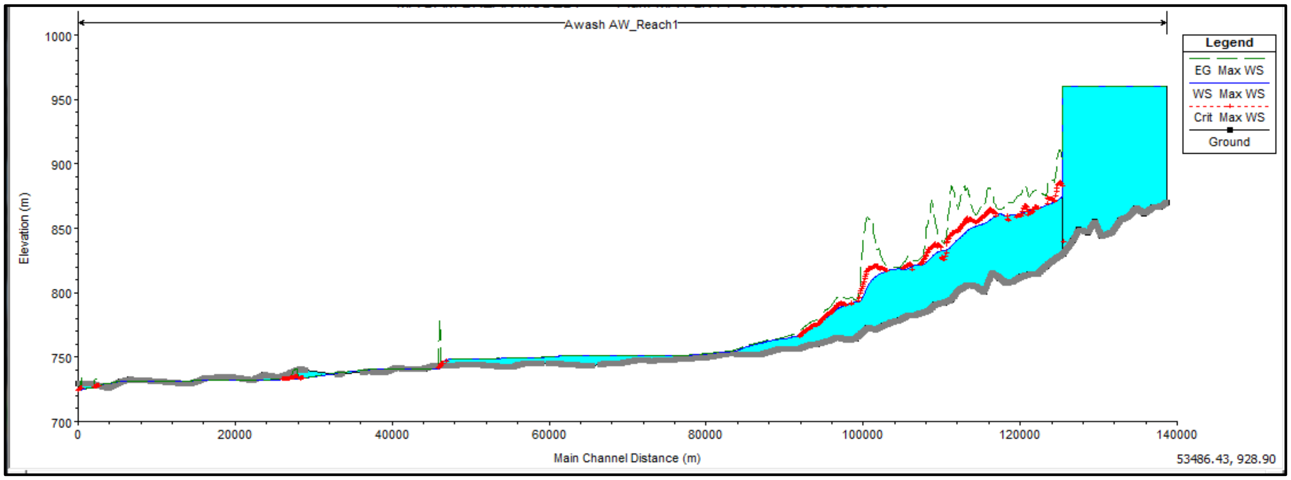
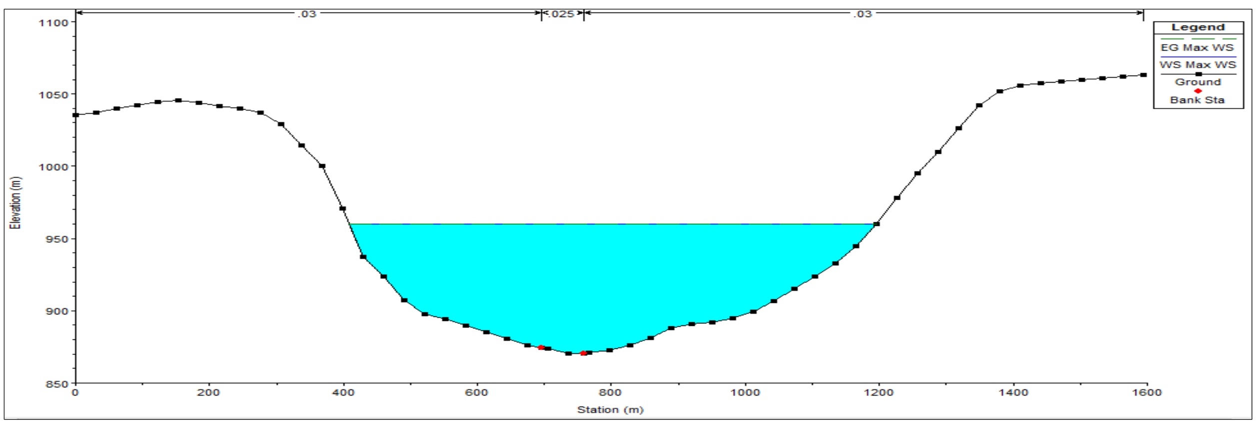
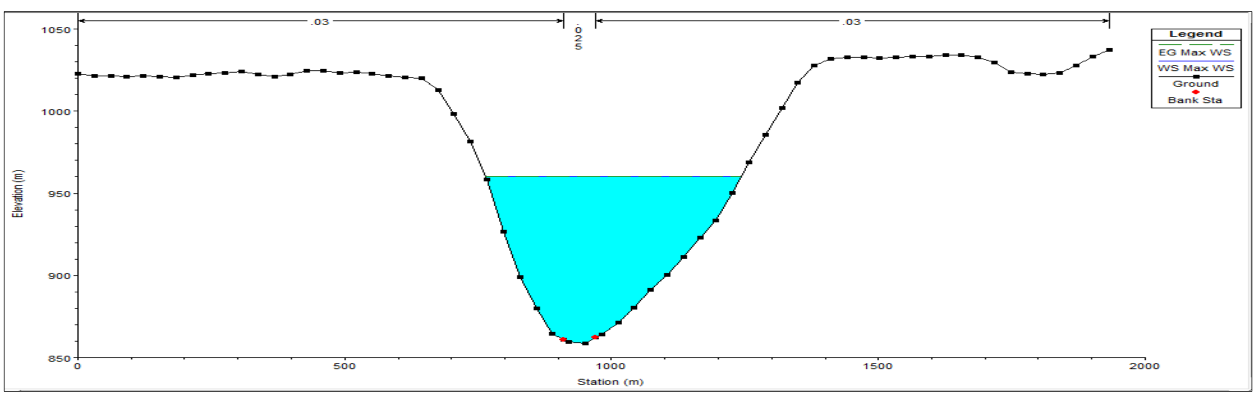


Figure 5:14 Water Surface Profile Plot

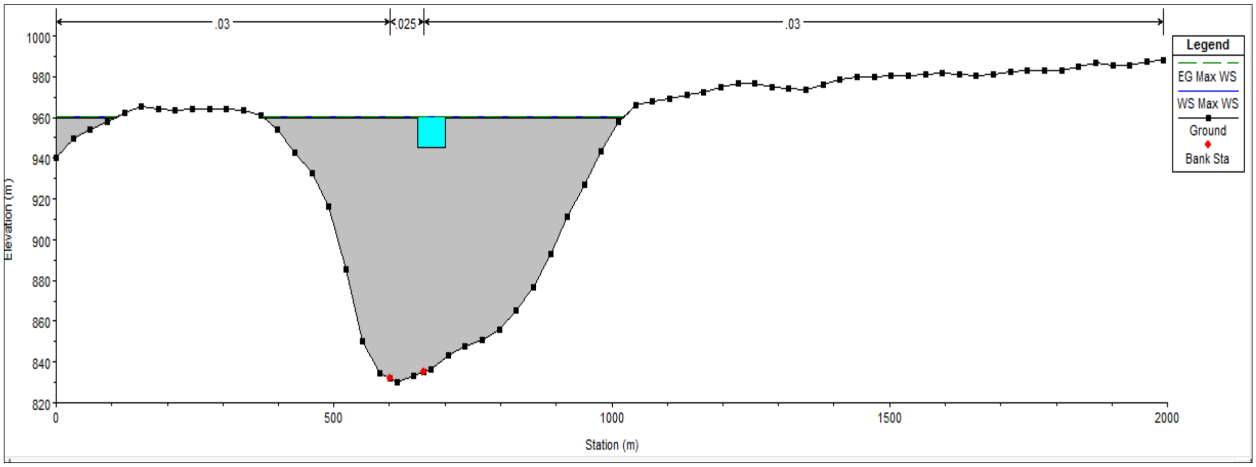
- The cross section view at selected stations:



Station=141417.6



Station=136520



Station=128190IS

Figure 5:15 Graphical Representation of river cross sections

5.4. Flood Inundation Map of Middle Awash Dam Failure

The inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It also identifies zones of high velocity flow and depth of inundation with time at representative cross-sections of the channel.

For Middle Awash Dam Break study the flood map was done using Ras Mapper in HEC-RAS and GIS. The method that has the shortest breach time arrives soon has selected for the inundation mapping as it would produce the most conservative results.

The area inundated with overtopping and piping failure is delineated. The flood depth and flood hazard map is presented. *(Fig5.16- Fig 5.21*))

For emergency planning purposes, it is important to determine the flood arrival time, flood depth, flood hazard class, velocity, flood recession and flood duration and the time to maximum discharge in the event of a dam breach. *(Table 5.4&5.5)*

Table 5:4 Inundation Map Result Froehlich (2008) - Overtopping

Location	Depth-m	Velocity	Hazard Map(V^D)	Flood inundation Boundary	Arrival Time -day	Recession Time -day	Duration	Flow	Percent Time Inundated	WSE
0m from the Dam(at the dam)	55	1.5	450	31000ha	0	0.125	0.42	115322	0.88	919
10km from Dam	20	0.8	30		0.083	0.166	2	75624	0.65	865
MelkaSedi(44km from Dam)	11	0.15	3		0.25	8.5	13.6	19359	100	754
MelkaWerer(80km from Dam)	10.2	0.045	0.6		0.4	19	19	3891	100	746
120 km from Dam	9.8	0.042	0.52		0.5	19	19	3829	100	736
End of the XS(126km)	9.5	0.041	0.45		0.6	19	19	3821	100	735

(ANNEX 9) –OV-INUNDATION MAPS

Table 5:5 Inundation Map Result Froehlich (2008)-Piping

Location	Depth	Velocity	Hazard Map(V^D)	Food Boundary	Arrival Time	Recession Time	Duration	Flow	Percent Time Inundated	WSE
0m from the Dam(at the dam)	40	1.11	370	30840ha	0	0.125	0.125	101971	0.65	913
10km from Dam	15	0.9	46		0.04	0.166	0.125	56378	0.47	865
MelkaSedi(44km from Dam)	9	0.12	3		0.05	19	19	21616	100	754
MelkaWerer(80km from Dam)	8.6	0.04	0.4		1.08	19	19	2467	100	747
120 km from Dam	8	0.04	0.3		2.7	19	19	1657	100	737
End of the XS(126km)	6	0.04	0.2		2.9	19	19	1609	100	735

(ANNEX 9) –PPG-INUNDATION MAPS

5.4.1. Flood Inundation Boundary

Inundation boundary is the generated polygon boundary of flood extent. During Middle Awash Dam Failure with overtopping and piping the area of 31000h and 30840 ha inundated respectively. This Flood inundates towns and roads (*Figure5:16&5:17*). The failure of the dam and inundation of the downstream environment leads to the life loss, environmental damage, infrastructure damage and economic loss.

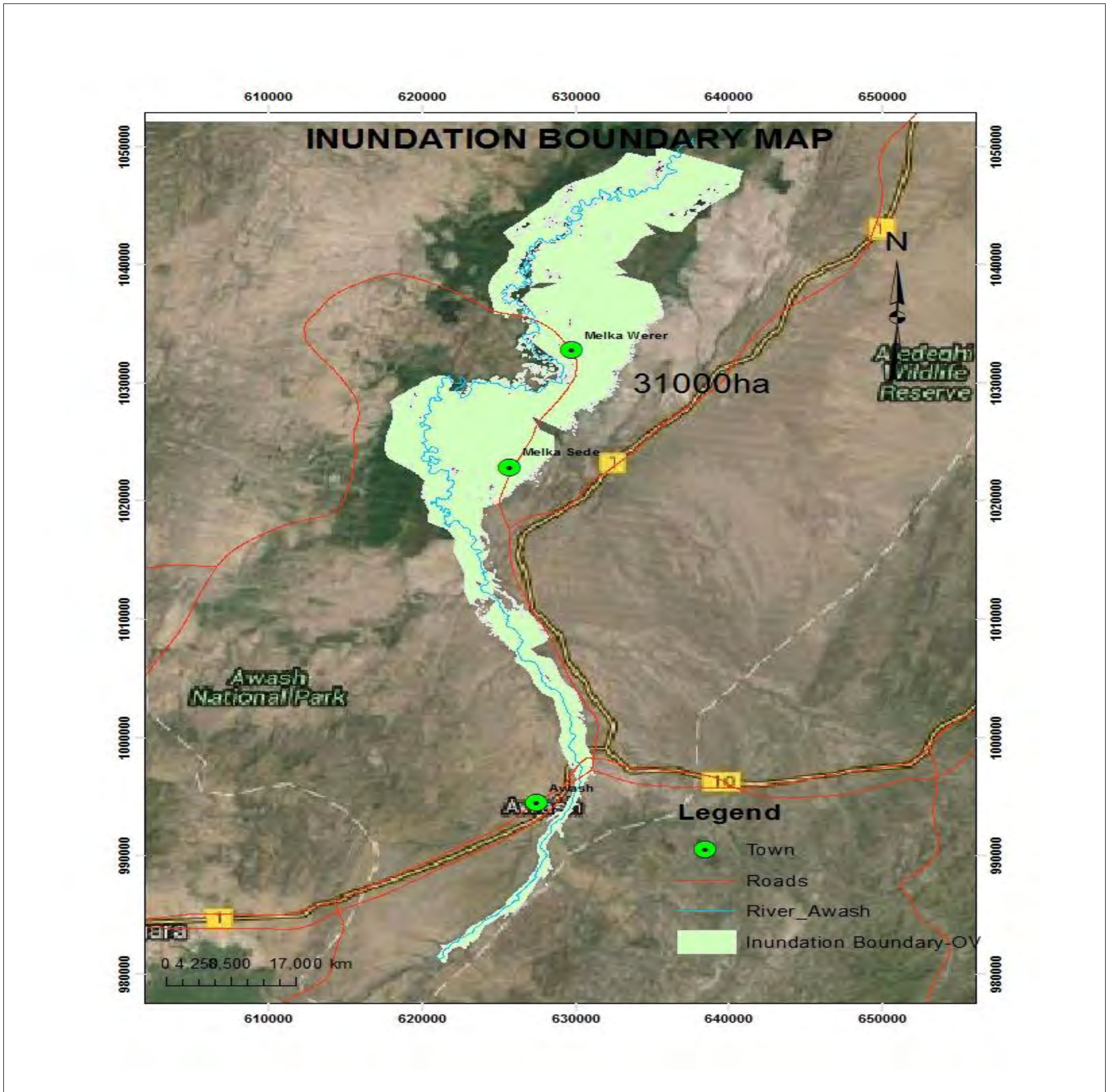


Figure 5:16 Flood Inundation Boundary Overtopping Failure

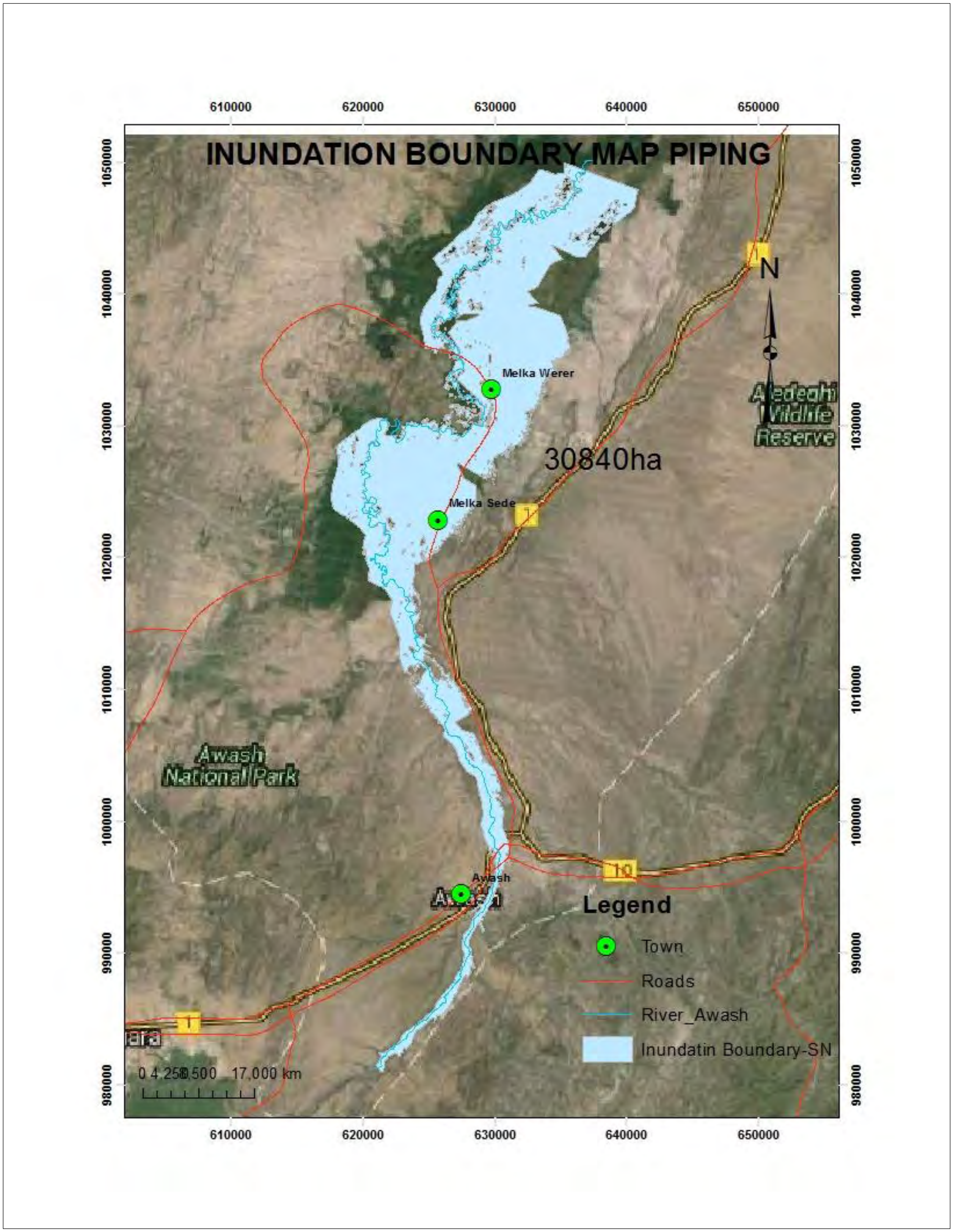


Figure 5:17 Flood Inundation Boundary Piping Failure

5.4.2. Flood depth

The depth map is computed based on the difference in water surface elevation and terrain layer based on the interpolated water surface. The depth of flood in the downstream around the towns where population, cultivation areas and infrastructures available is about 0m to 20m (*Figure 5:18&5:19) & (Table5:4&5:5)* for both overtopping and piping.

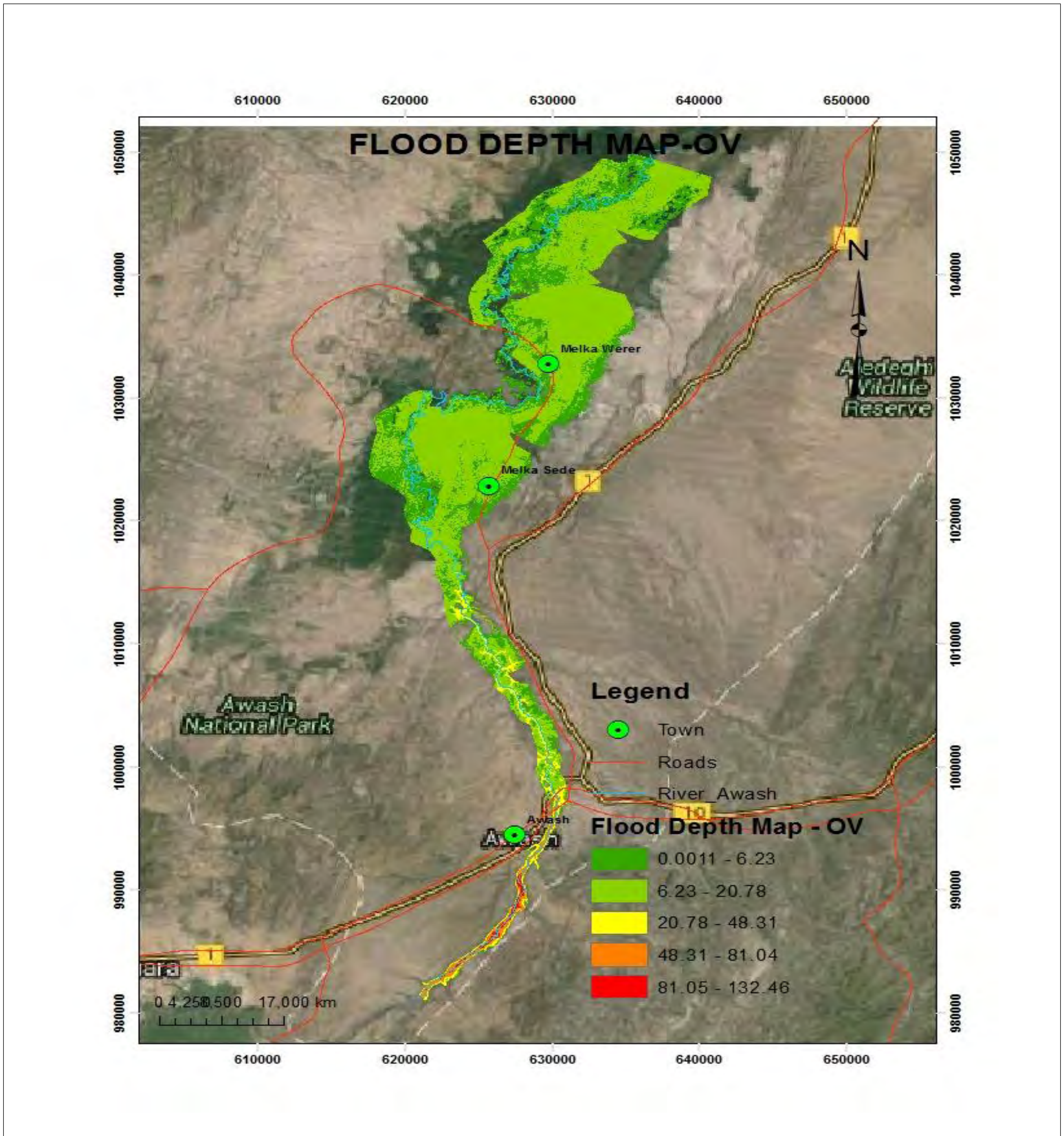


Figure 5:18 Flood Depth Map – Overtopping

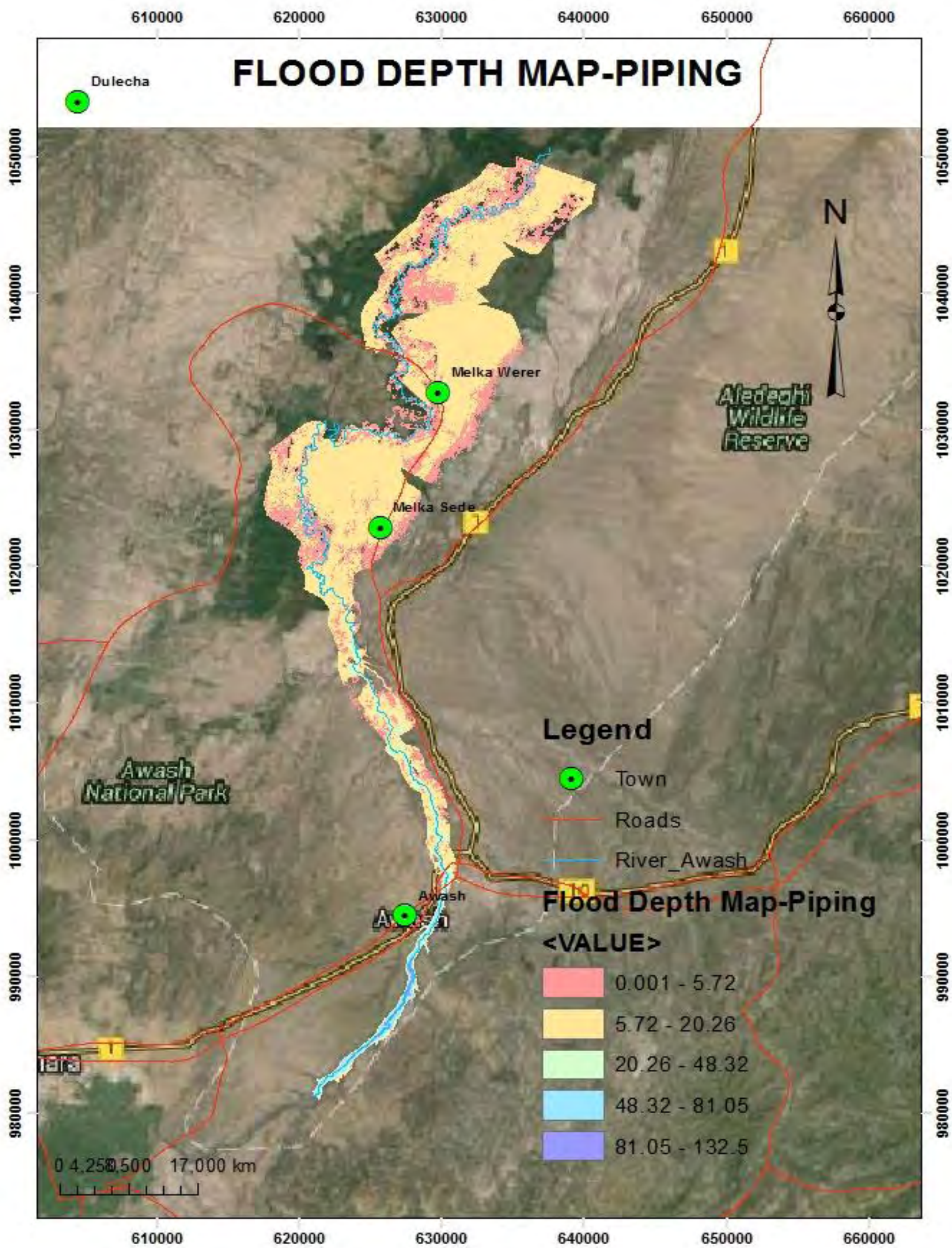


Figure 5:19 Flood Depth Map – Piping

5.4.3. Flood Hazard Map

The inundation areas were mapped as zones of hazard class based on a combination of flow depth and velocity and the overtopping is more hazardous than piping *(Fig 5.20 &5.21)*. First the failure of the dam itself is one economic loss. Because the Dam structures are constructed with high cost, the aim the dam is constructed like irrigation farms, water supply schemes and infrastructures developed following the construction of dam like newly developed industries and socio-economic establishments will be flooded with failure of the dam. In addition failure affects the towns at downstream area, farm lands, the population, animal life, environmental and the economic losses. The total population of 78,105of the downstream area located in Amibara Wereda of Afar Region (WWDSE S.-E. R., 2014) , towns at downstream Melkasedi and Melkawerer and infrastructures, Livestock Husbandry, state irrigated mechanized farm producing cotton, several crops, fruits and vegetables and vegetations shrubs and some wood land around river bank *(Fig5.22)*. The flood inundation boundary includes this area.

According to USACE Dam hazard potential classification*(Table2:6)* and flood hazard map *(Figure 5:20 &5:21)* the flood flow at these sites is classified as hazardous.

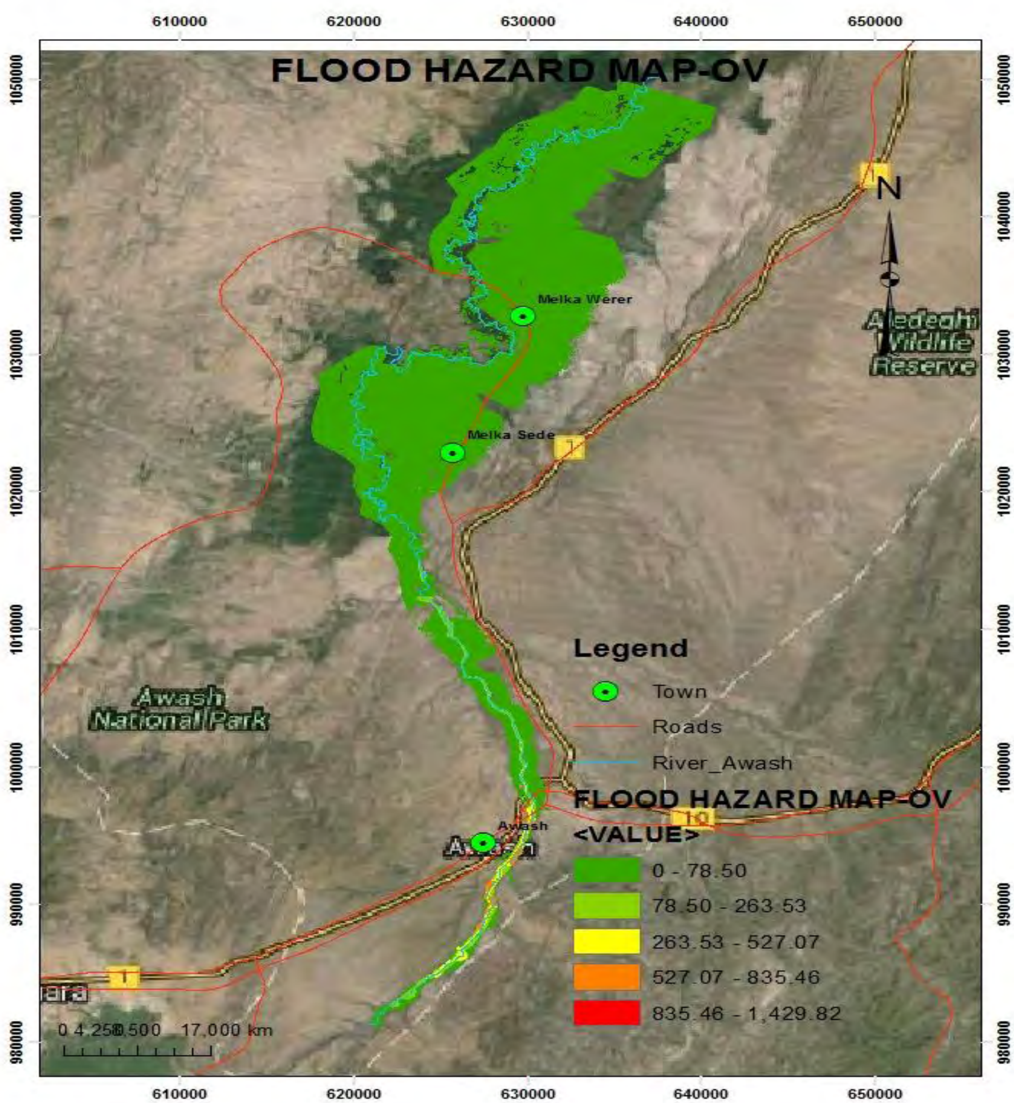


Figure 5:20 Flood Hazard Map -Overtopping

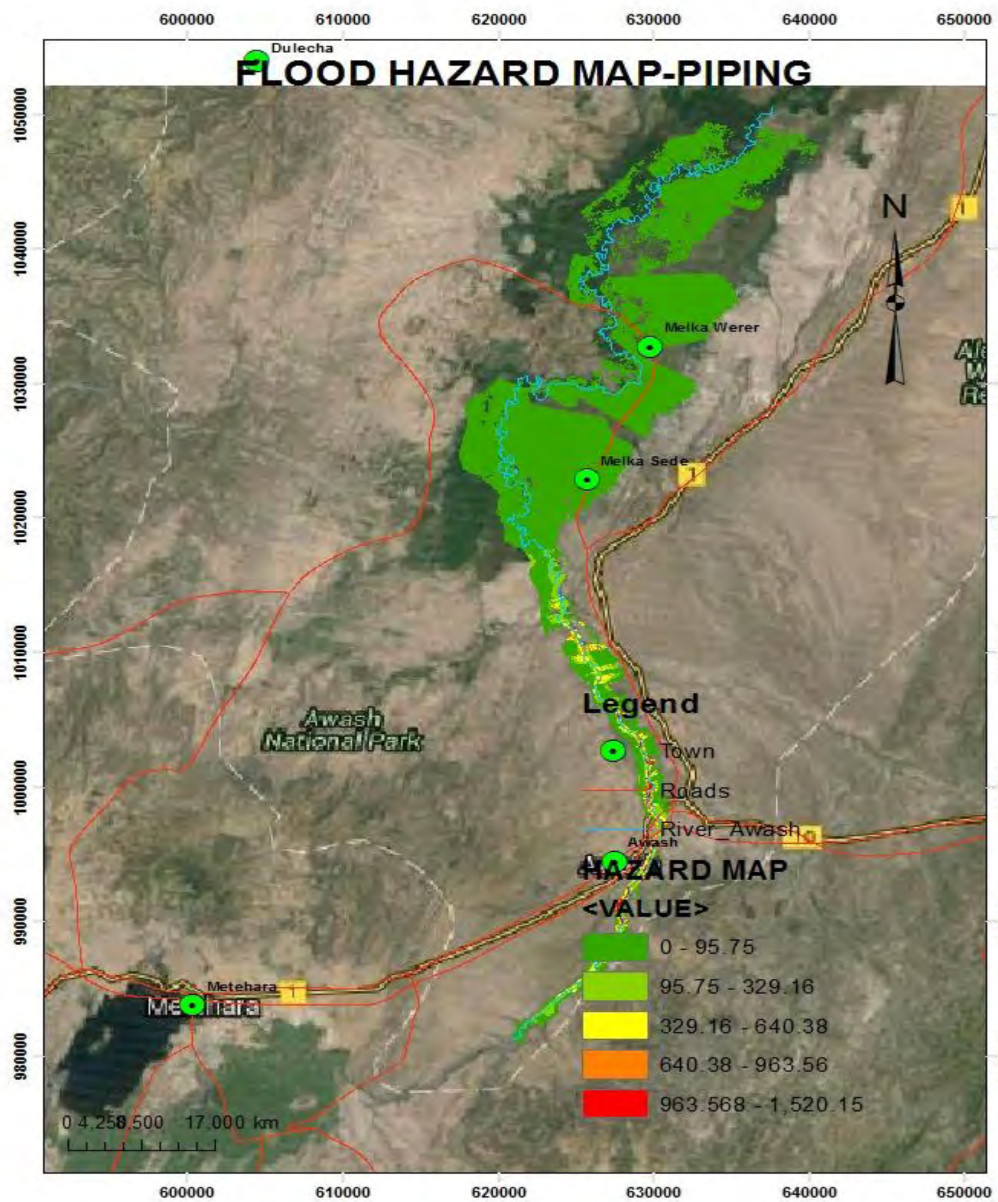


Figure 5:21 Flood Hazard Map –Piping

6. CONCLUSION

6.1. Breach Parameters, Methods and Flood hydrograph

In this thesis work, dam breach analysis for Middle Awash Multipurpose Dam, which is located in Afar region of Ethiopia, were done. Middle Awash dam failure was checked for two failure scenario using the HECRAS to model breaching of the embankment dam and the inundation extent during piping(sunny day) failure mode at normal pool level and hydrologically induced failure with PMF.

Piping occurs by erosion of fine sediments out of the soil matrix of the embankment dam starting from the dam face or downstream toe and its opening expands upward.The model is simulated at the reservoir normal pool elvation and initial piping elevation of 880m. The resulted piping breach outflow is 100045m3/s with the volume 4.7B m3. As the dam is rockfill embankment dam this failure can occur if impervious core material used is poor in quality and construction procedure due to the succeptibility of the dam and foundation material to concentrated seepage.

PMF was checked if it can overtop the dam and cause any threat. The model revealed at peak event the PMF can overtop the dam at full reservoir conditionis. The resulting breach outflow is 104814m3/s with the volume of 4.72B m3.

For the outflow hydrograph resulted from the breaching dam unsteady flow routing through the downstream river channel and flood plain was done and different maps of the resulting water surface extent, water depth and hazard map are produced.

The failure phenomena is unexpected and an abrupt. Hence, evacuating the downstream community after the failure process has already commenced may not be possible to save lives and property due to the aggressive nature of the wave front.

This research contribute some useful information for Middle Awash Dam to minimize these catastrophes of the dam failure and for information regarding dam safety issues that should be considered and precautions to bear in mind while implementing infrastructures on downstream areas and input data in preparing Emergency Action Planning. Moreover,designers, consultants and contractors engaged on dam works may gain an input data for their work and forecasting the possible dam break flood to develop disaster management plans such as constructing flood protection dykes, structuring evacuation techniques, transmitting inundation information and setting early warning systems.

6.2. Flood Inundation map

Estimating the inundation begins with an estimate of the flood hydrograph; how much water pours through the breach and how fast it pours through. Middle Awash Dam flood inundation maps were generated using RASMAPPER and ArcGIS.

Flood Map done for the area inundated with breach outflow of Middle Awash Dam with both overtopping and piping dam failure for selected sites at downstream.

Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at a given location in the event of a dam breach.

Information how much warning time will there be before the flood helsp in estimating failure consequences.

In addition, information of the flood warning time, time to peak, peak elevation, and peak discharges, flood depth , flow, velocity, arrival time, duration and recession time, WSE and hazard were depicted on the inundation maps at key locations, which is important for the purpose in providing information for the society awareness and preparedness for emergency action plan.

The dam break analysis helps in order to determine the failure consequence of the dam: the population at risk, the estimated loss of life, the cultural and environmental consequences, the economic losses and the impact on infrastructure considered which helps in emergency plan purpose

The area 31000ha and 30840ha are inundated with the overtopping and piping failure of Middle Awash dam.

The distance downstream form the dam 10km, 44km (MelkaSedi town), 80km(MelkaWerer), 120km and 126km.

- Inundated area is higher for-Overtopping.
- Flood hazard class is higher at the dam and 10km downstream from the dam site

The towns at downstream Melkasedi and Melkawerer and the farms are affected. The flood flow at these sites is classified as hazardous.

So the failure of this dam hazardous. It damages the towns at downstream area, farm lands, the population at risk, loss of animal life, and environmental damage and the economic losses..

7. RECOMMENDATION

- For the dam at the planning stage carefull selection of construction material and following the proper engineering standard procedure for design and construction is recommended to minimize the risk of failure of embankment dam with piping due to erosion of materials and overtopping.
- The breach parameter and breach method affects the result of breach out flow and flood inundation area during modeling these should be selected carefully..
- The dam breach modeling results inform the extents of flooding. There should be accurate estimation of the severity and extent of dam break flood prior to the construction of a dam Therefore, damages that could occur in the surrounding settlements, agricultural areas, on both lives and infrastructure can be minimized and even controlled.
- The maps used for many land use planning purposes considering the dam failure consequence flood warning, , hazard classification and updating the classification for a gap of year for existing dams, for dams under planning stage to identify flood prone areas,
- Also helps for Preparedness and emergency action planning related to dam failure by alert concerned government bodies to take a precaution on dam safety plans, prepare an early flood warning system, timely aware downstream inhabitants about the hazardous flood and plan a proper emergency evacuation method on disaster time

8. BIBLIOGRAPHY

Anon., n.d.

Behailu, S., 2004. *Stream flow simulation for UpperAwash Basin*. Addis Ababa University.

Brunner, G.W., 2002. *HEC-RAS, River Analysis System Hydraulic Reference Manual*. Davis: US ARMY CORPS OF ENGINEERS.

Brunner, G.W., 2006. *HEC-RAS, River Analysis Systme User's Manual*. Davis: US ARMY CORPS OF ENGINEERS HYDROLOGIC ENGINEERING CENTER (HEC).

Cameron T. Ackerman, P.E.a.G.W.B.P.E., n.d. Dam Failure Analysis Using HEC-RAS and HEC-GeoRAS.

Cameroon T. Ackreman, P.E.a.G.W.B.P.E., n.d. *Dam Failure Analysis UsingHec-Ras and Hec-GeoRas*. Davis,CA 95616: Hydrologic Engineering Center.

CDA, 2013. *Dam Safety Guidelines*. Canadian Dam Association.

Colorado, D.S.B., 2010. Guidelines for Dam Breach Analysis. In *State of Colorado Department of Natural Resources Division of Water Resources*. State of Colorado: Colorado Division of Water Resources. p.68.

Damsafety, G., 2007. Dam Break Inundation Analysis and Downsream Hazard Classification. Washington: MGS Engineering Consultants. p.34.

FEMA, 2004. *Hazard Potential Classification system for Dams*. Federal Guidelines for Dam Saftey.

FEMA, 2013. *Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures*. Federal Guidelines.

FEMA, July 2013. *Federal Guidelines for Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures*.

Fread, D.L., 1988. *An Erosion Model for Earthen Dam Failures*. Silver Spring, Maryland: National Weather Service, National Oceanic and Atmospheric Administration.

Gee, M., 2010. Use of Breach Processes Models to Estimate HEC-RAS Dam Breach Parameters. In *2nd Joint Federal Interagency Conference, Las Vegas, NV. June 27-July 1, 2010.*, 2010.

Goodell, C., 2006. *Flood Inundation Mapping Using Hec-Ras*. Obras Y Proyectos.

Guner, B.&., 2001. Pre-Event Dam Failure Analysis for Emergency Management. *Turk J Engin Environ Sci*.

Halcro-, 1989. Addis Ababa.

ICODS, 1983. *Dam Safety Emergency Action Planning Guidelines*. Integracy Committee on Dam Safety.

ICOLD, C.O.H.F.D., 1998. *DAM-BREAK FLOOD ANAYSIS*. Paris: International Committee on Large Dams.

Litani River Basin, M.S.P., January, 2012. *Dam Break Modeling for Qaraoun Dam Main report*.

MacDonald, T.C.a.L.-M., 1984. Breaching Characterstics of Dam Failures. *Journal of Hydraulic Engineering*.

Pistocchi, A..M.P., n.d. Use of HEC-RAS and HEC-HMS models with ArcView.

Ravindra, 1.S.L.2., , 2012. DYNAMIC FLOOD ROUTING AND UNSTEADY FLOW. *International Journal of Advanced Engineering Technology*.

Reclamation, B.o., 1988. *Downstream Hazard Classification Guidelines*. Denver,Colorado: Assistant Commissioner-Engineering and Research.

RIVER, L., LITANI.

Singh, V.a.S.P., 1998. *American Society of Civil Engineers*. [Online] Available at: http://dx.doi.org/10.1061/(ASCE)0733-9429(1988)114:1(21) [Accessed 27 June 2014].

Tariku, 2015. *Dam Break analysis and Risk assessment*. Addis Ababa.

Tony L, W., 1998. *Predicting Embankment Dam Breach Parameters*. Research Report. US: Dam Safety Office water resource research laboratory.

USACE, 2010. HECRAS REFERENCE MANUAL.

USACE, 2012. *Hecgeo Ras Manual*. USACE Hydrologic Engineering Center.

USACE, 2016. *HECRAS REFERENCE MANUAL*.

VanRijn, n.d. *Principles of Sediment Transport in Rivers, Estuaries and Coastal Seas*. Delft Hydraulics.

Wahl, T.L., 1997. *Prediction of Embankment Dam Breach Parmeters*. U.S. Department of the Interior Bureauof Reclamation.

Wahl, T.L., 2004. Uncertainty of Predictions of Embankment Dam. *JOURNAL OF HYDRAULIC ENGINEERING © ASCE*, pp.389-97.

Wahl, T.L., 2010. Dam Breach Modelling. In *2nd Joint Federal Integracy Conference on Sedimentation and Hydrologic Modeling*. Las Vegas, 2010. Hydraulic Engineering, Bureau of Reclamation, Denver, CO,twahl@usbr.gov.

Wermore, J.N.a.D.L.F., 1984. The NWS Simplified Dam Break Flood Forecasting Model for Desk-Top and Hand-Held Microcomputers. Federal Emergency Management Agency. p.122.

WIKIPEDIA, n.d. *WIKIPEDIA*.

Wurbs, R.A., 1987. Dam Breach Flood Wave Models. *Journal of Hydraulic Engineering*.

WWDSE, 2005. *Final Dam Design Report*. Addis Ababa.

WWDSE, S.-E.R., 2014. *Middle Awash Socio economic study feasibilty report*.

WWDSE, 2015. *Design Report*. Addis Ababa.

WWDSE, 2015. *Hydrologic Analysis*. Addis Ababa.

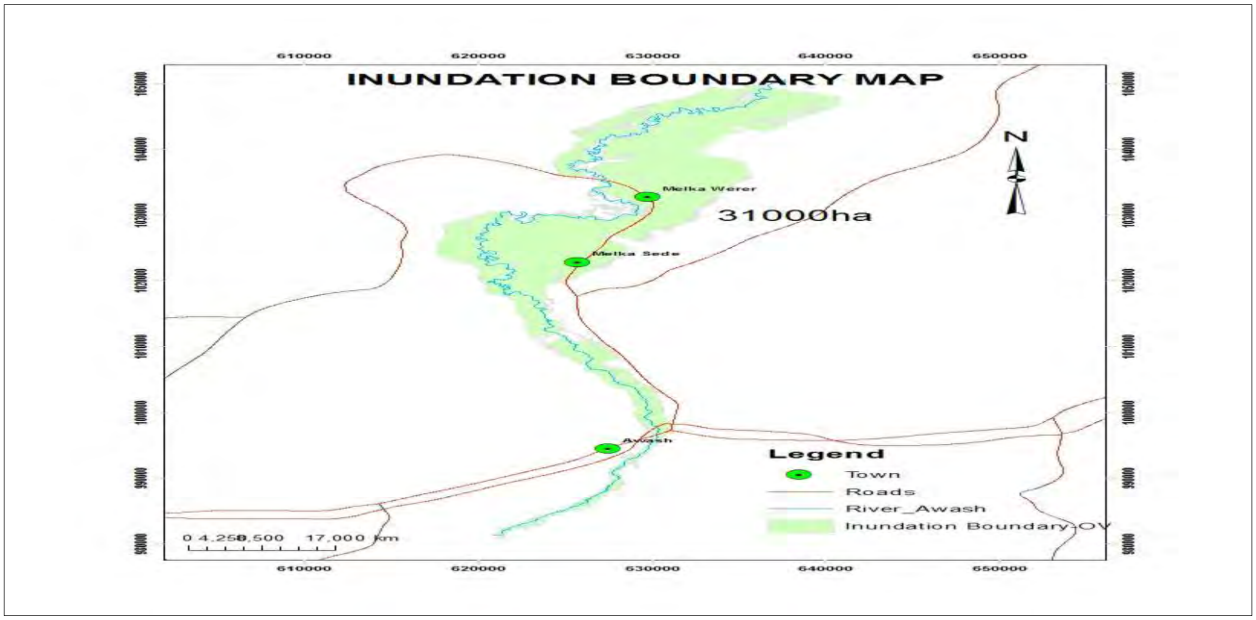
ZAGONJOLLI, M., 2007. *Dam break Modelling, Risk Assessment and Uncertainity Analysis for Flood Mitigation*. London, UK: Taylor & Francis.

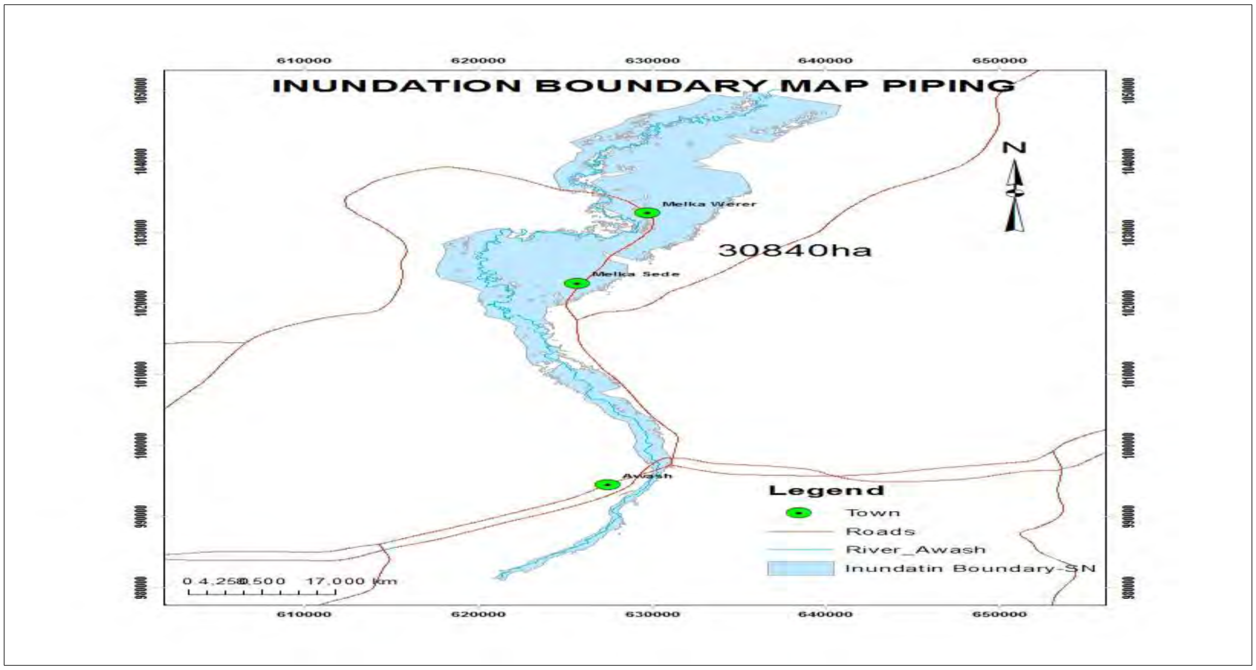
ZAGONJOLLI, M., 2007. *Dam break Modelling, Risk Assessment and Uncertainity Analysis for Flood Mitigation*. London, UK: Taylor & Francis.

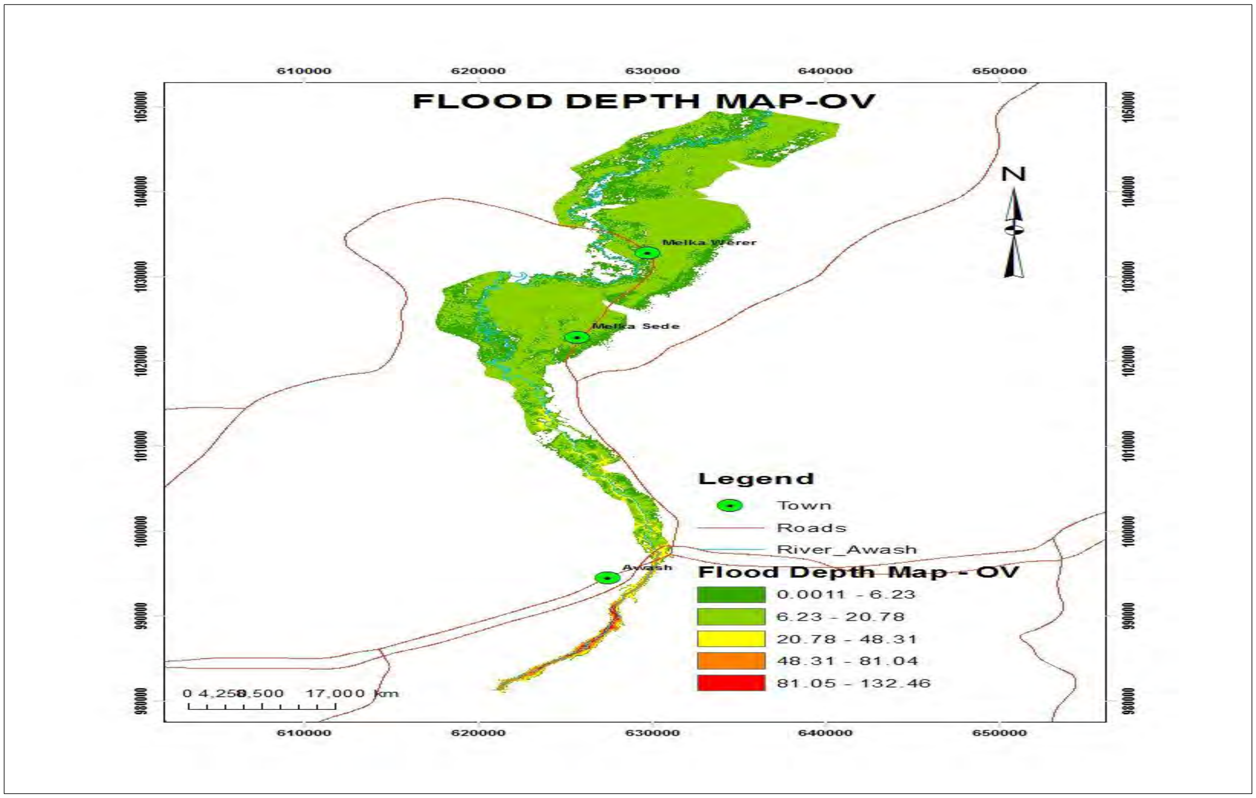
9. ANNEX

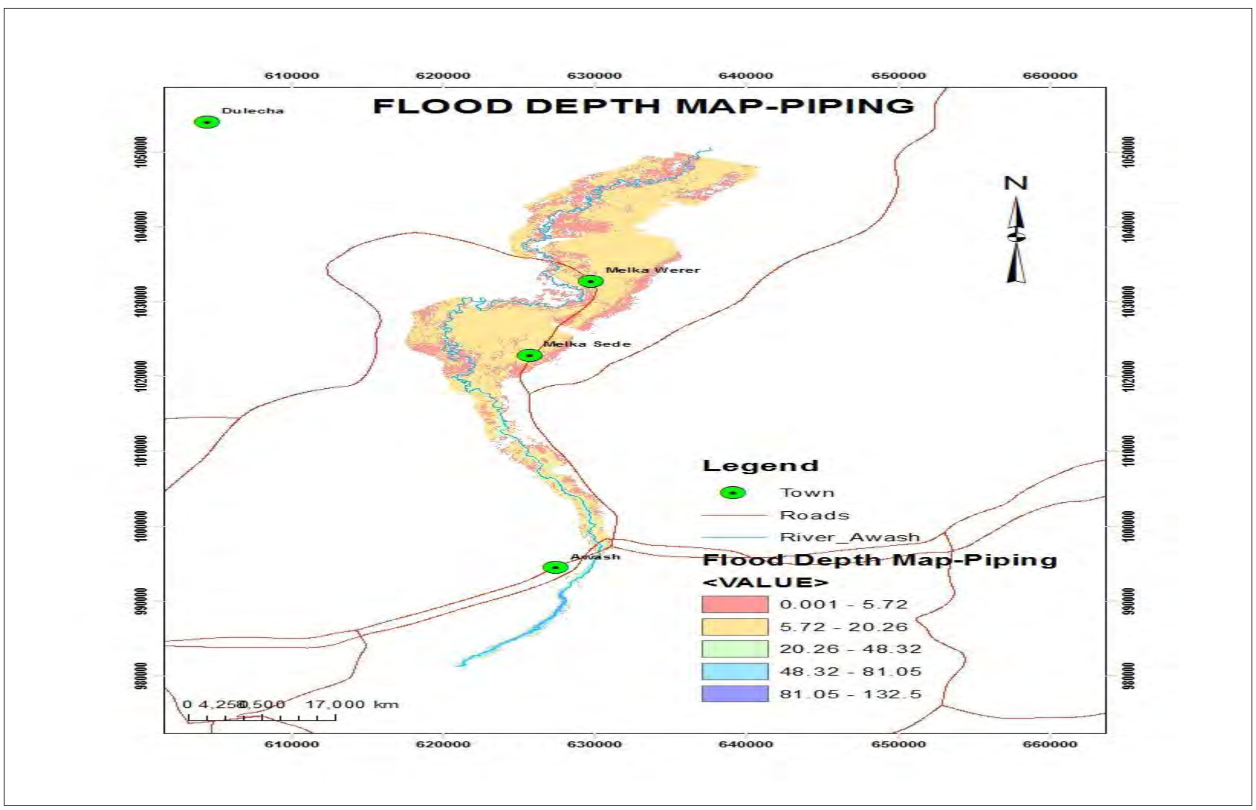
9.1. INUNDATION MAPS

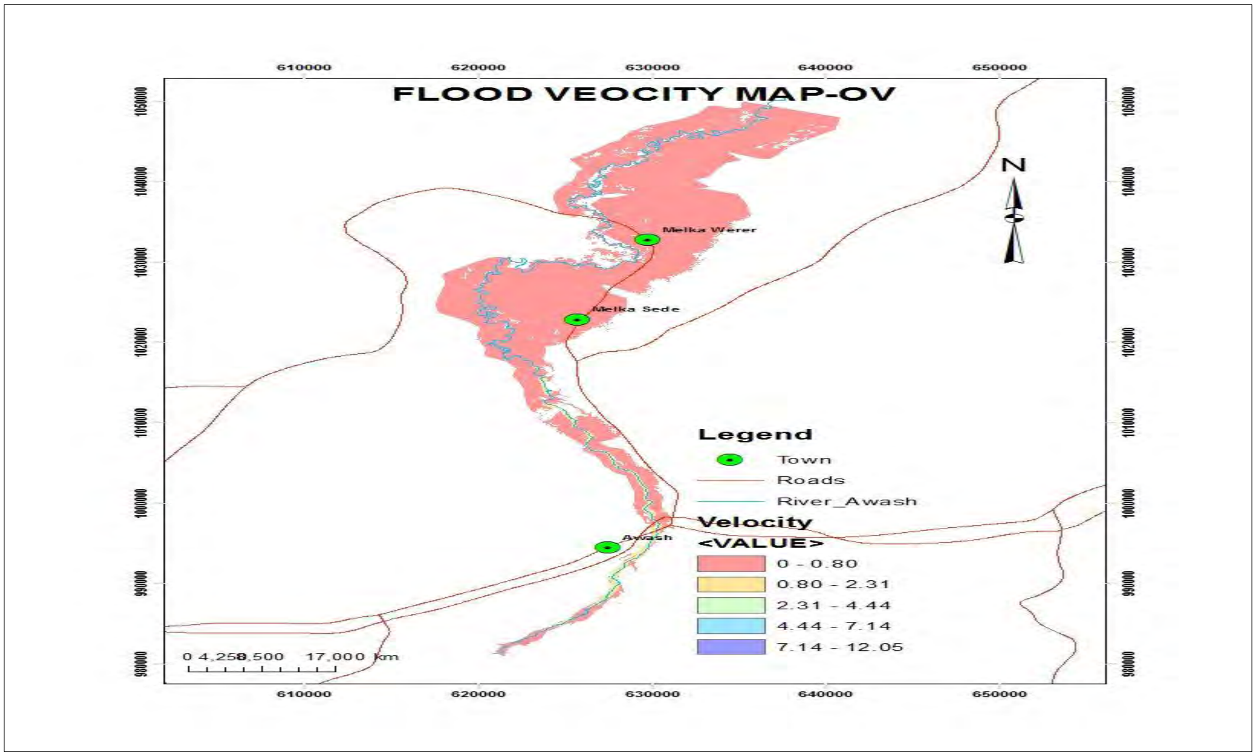
OVERTOPPING AND PIPING

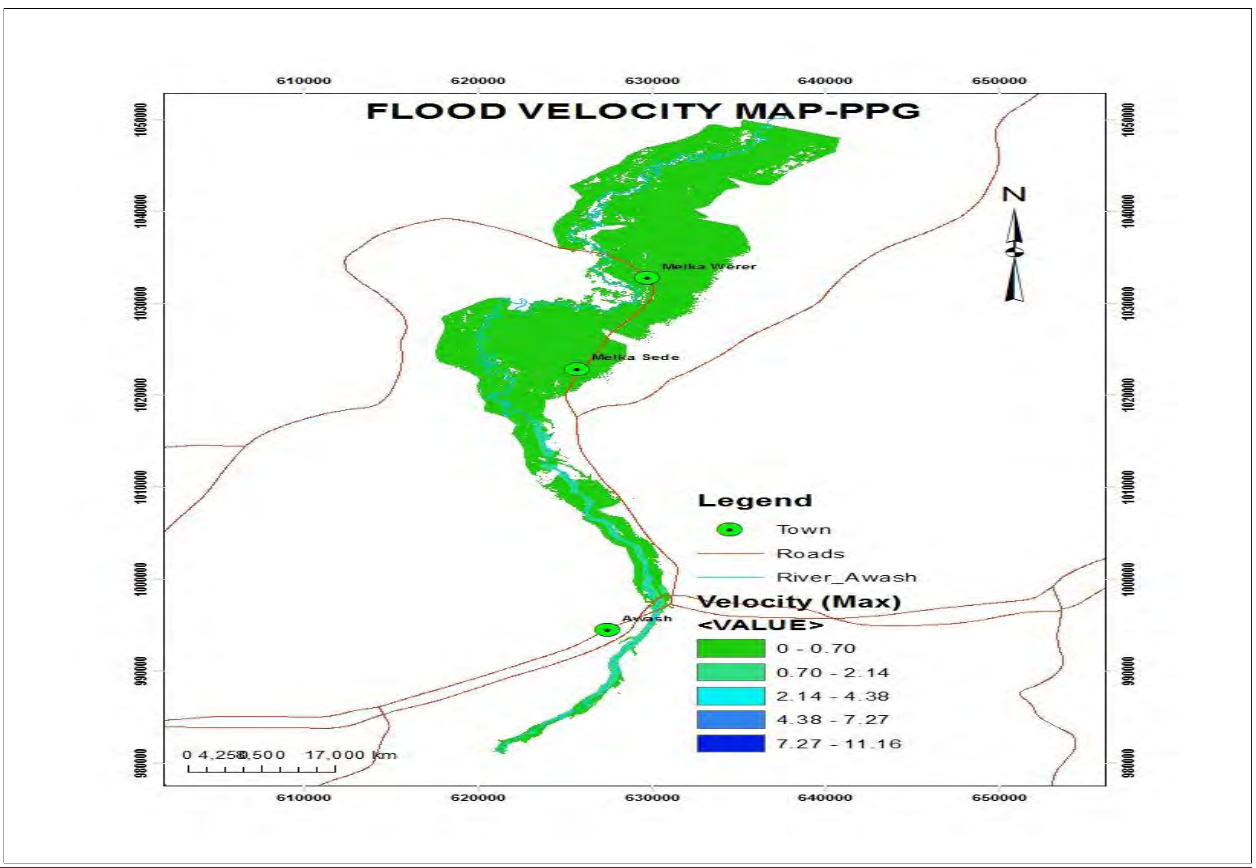


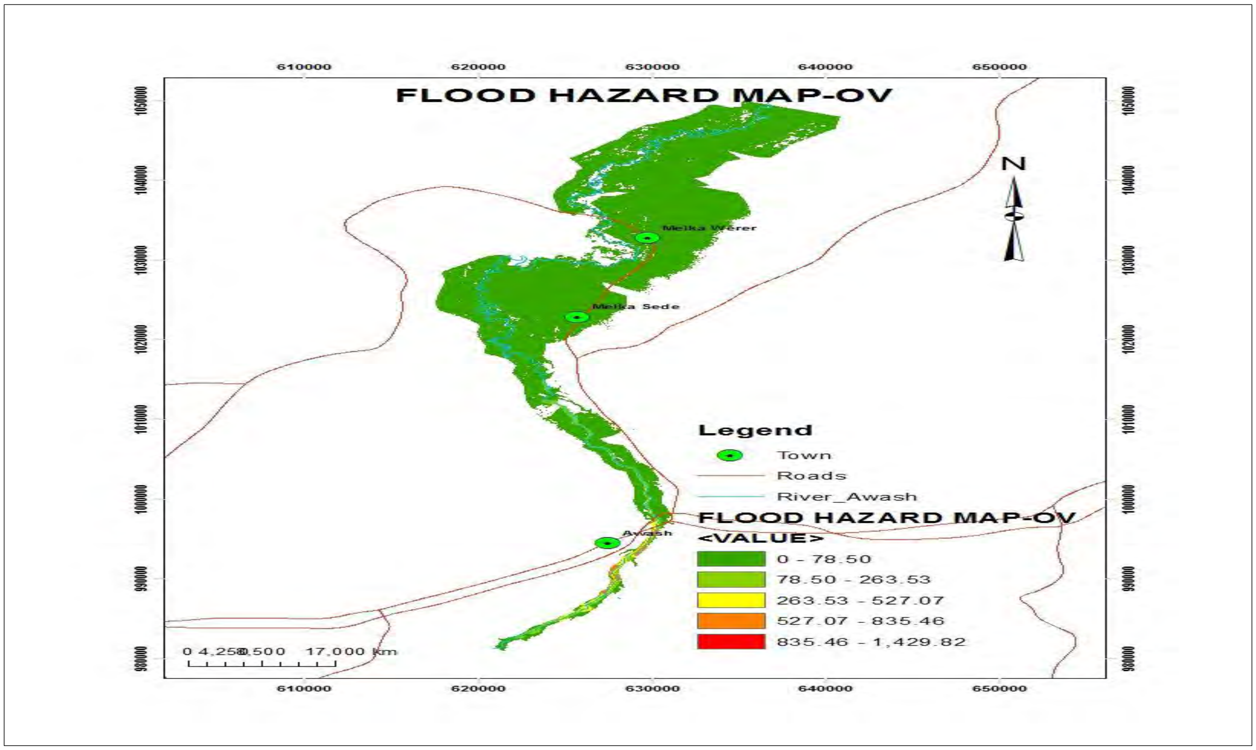


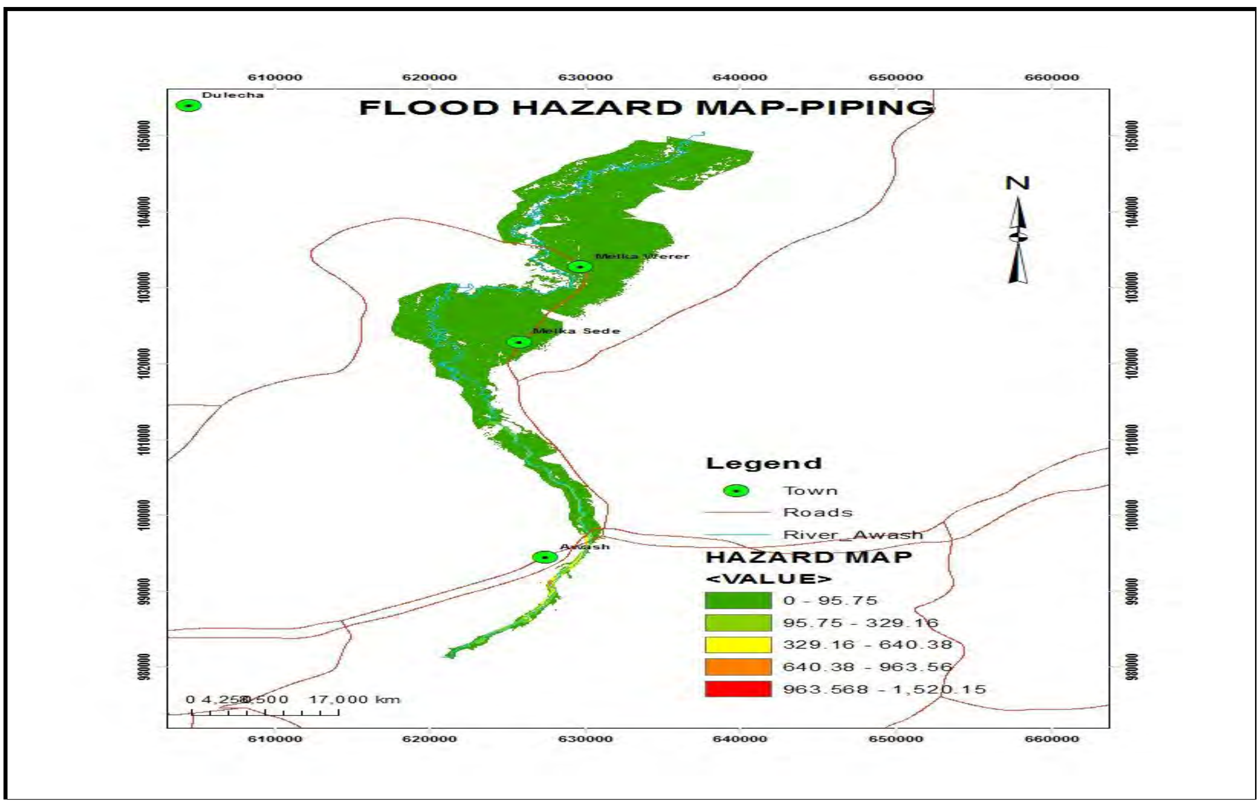

FLOOD DEPTH MAP-OV
610000
620000
630000
640000
650000
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Flood Depth Map - OV
0.0011 - 6.23
6.23 - 20.78
20.78 - 48.31
48.31 - 81.04
81.05 - 132.46
0 4,250 8,500 17,000 km

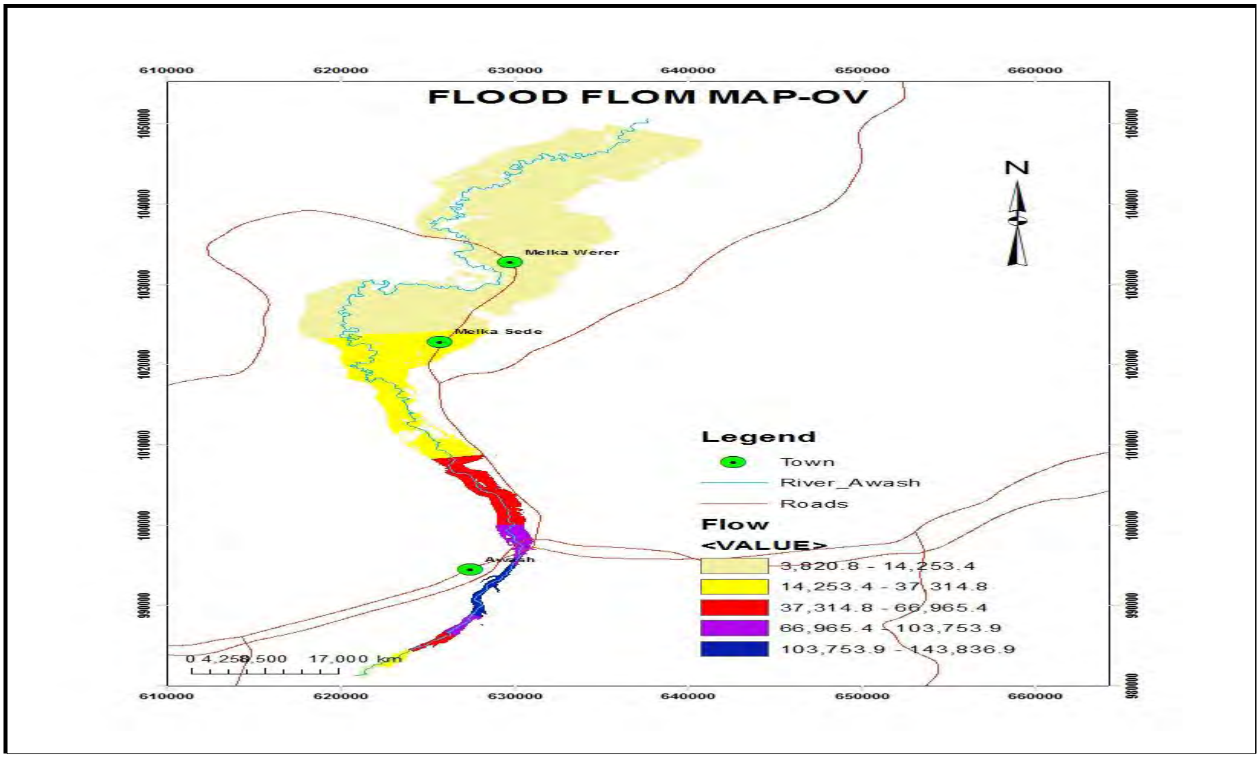

FLOOD DEPTH MAP-PIPING
610000
620000
630000
640000
650000
660000
Dulecha
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Flood Depth Map-Piping
<VALUE>
0.001 - 5.72
5.72 - 20.26
20.26 - 48.32
48.32 - 81.05
81.05 - 132.5
0 4,250 8,500 17,000 km

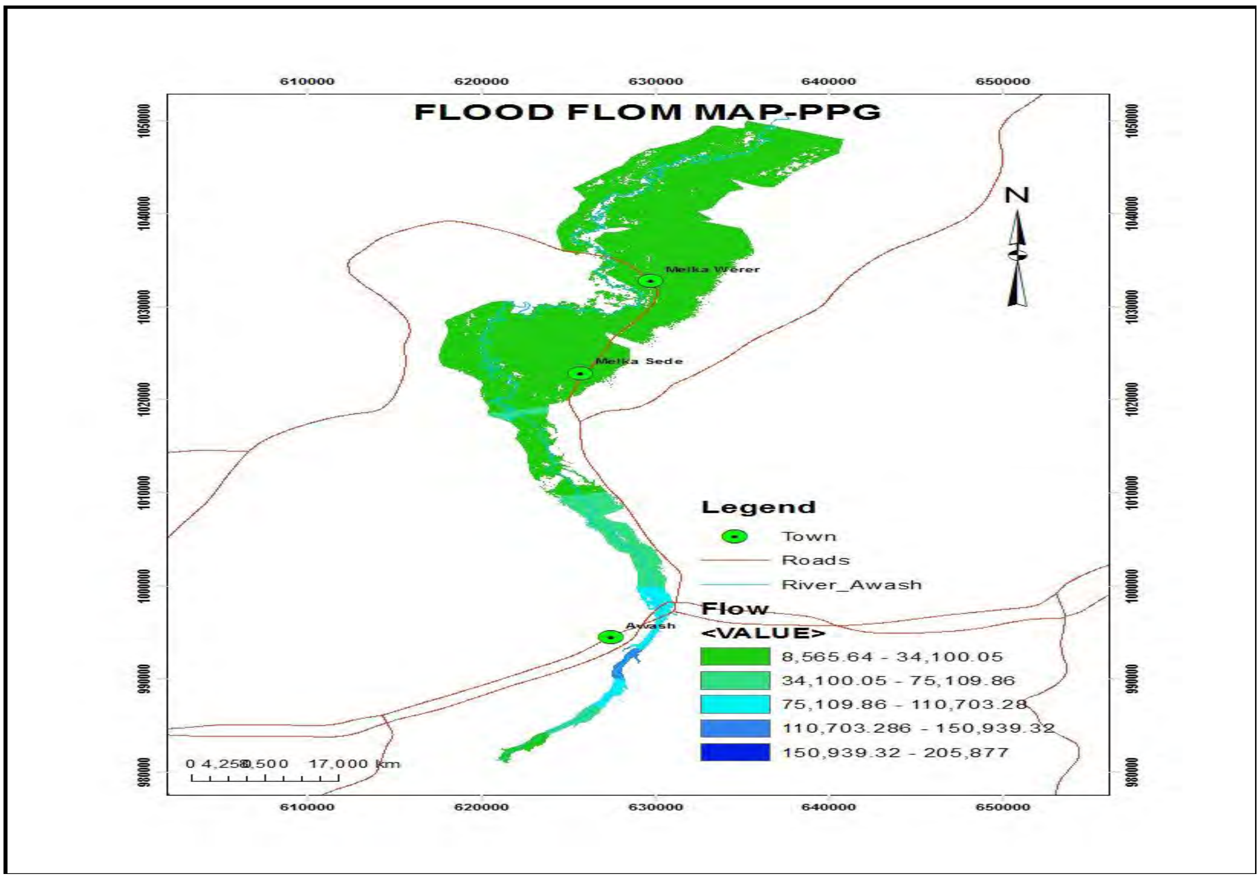

FLOOD VEOCITY MAP-OV
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Velocity
<VALUE>
0 - 0.80
0.80 - 2.31
2.31 - 4.44
4.44 - 7.14
7.14 - 12.05
0 4,2508,500 17,000 km
610000
620000
630000
640000
650000

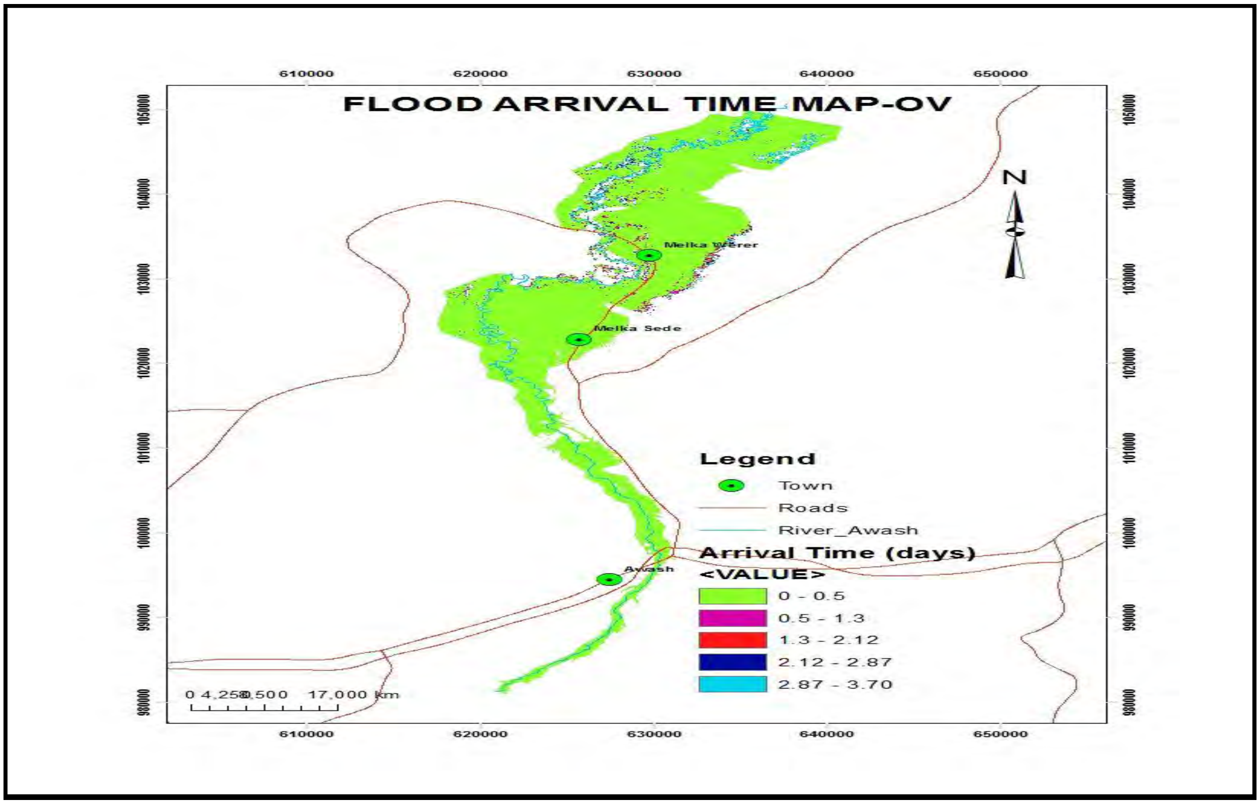

FLOOD VELOCITY MAP-PPG
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Velocity (Max)
<VALUE>
0 - 0.70
0.70 - 2.14
2.14 - 4.38
4.38 - 7.27
7.27 - 11.16
0 4,2508,500 17,000 km
610000
620000
630000
640000
650000

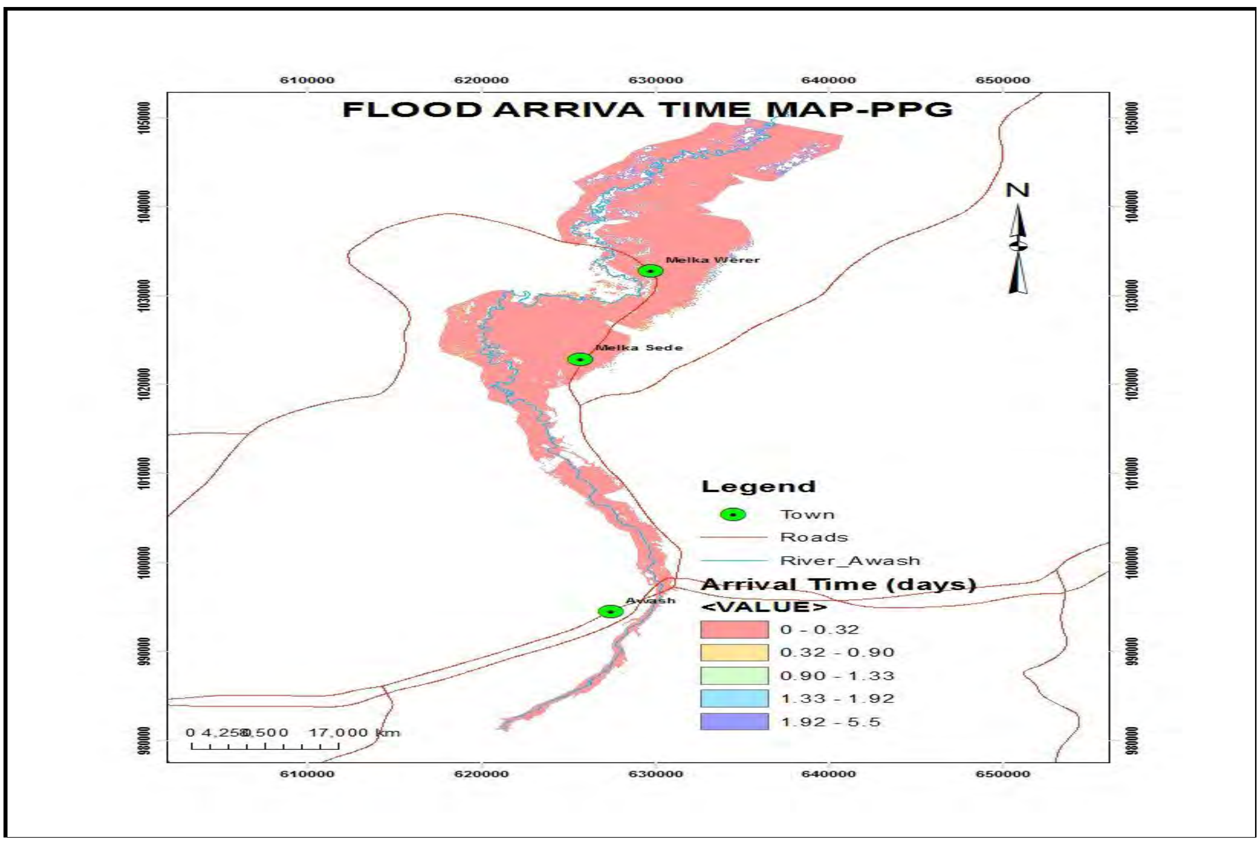

FLOOD HAZARD MAP-OV
Meika Werer
Meika Sede
Awash
Legend
Town
Roads
River_Awash
FLOOD HAZARD MAP-OV
<VALUE>
0 - 78.50
78.50 - 263.53
263.53 - 527.07
527.07 - 835.46
835.46 - 1,429.82
N
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000

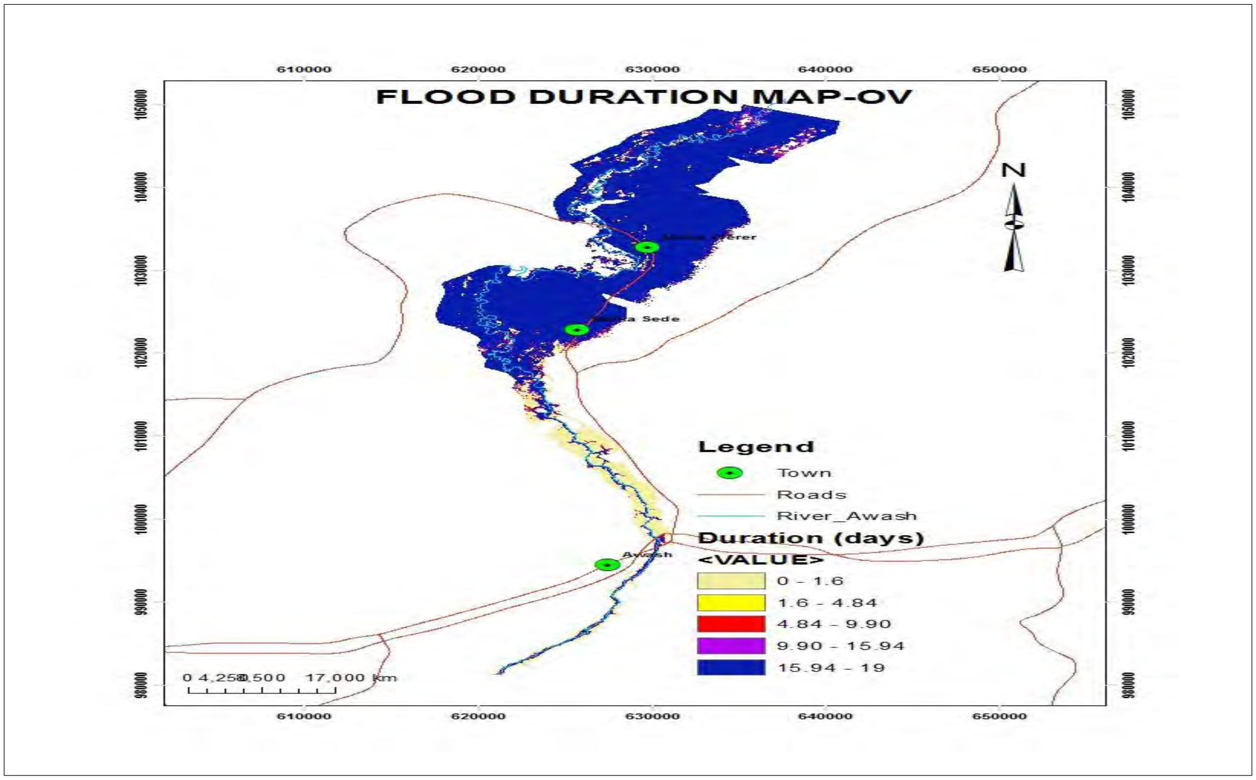

FLOOD HAZARD MAP-PIPING
Dulecha
Meika Werer
Meika Sede
Awash
Legend
Town
Roads
River_Awash
HAZARD MAP
<VALUE>
0 - 95.75
95.75 - 329.16
329.16 - 640.38
640.38 - 963.56
963.568 - 1,520.15
N
0 4,250 8,500 17,000 km
610000
620000
630000
640000
650000
660000

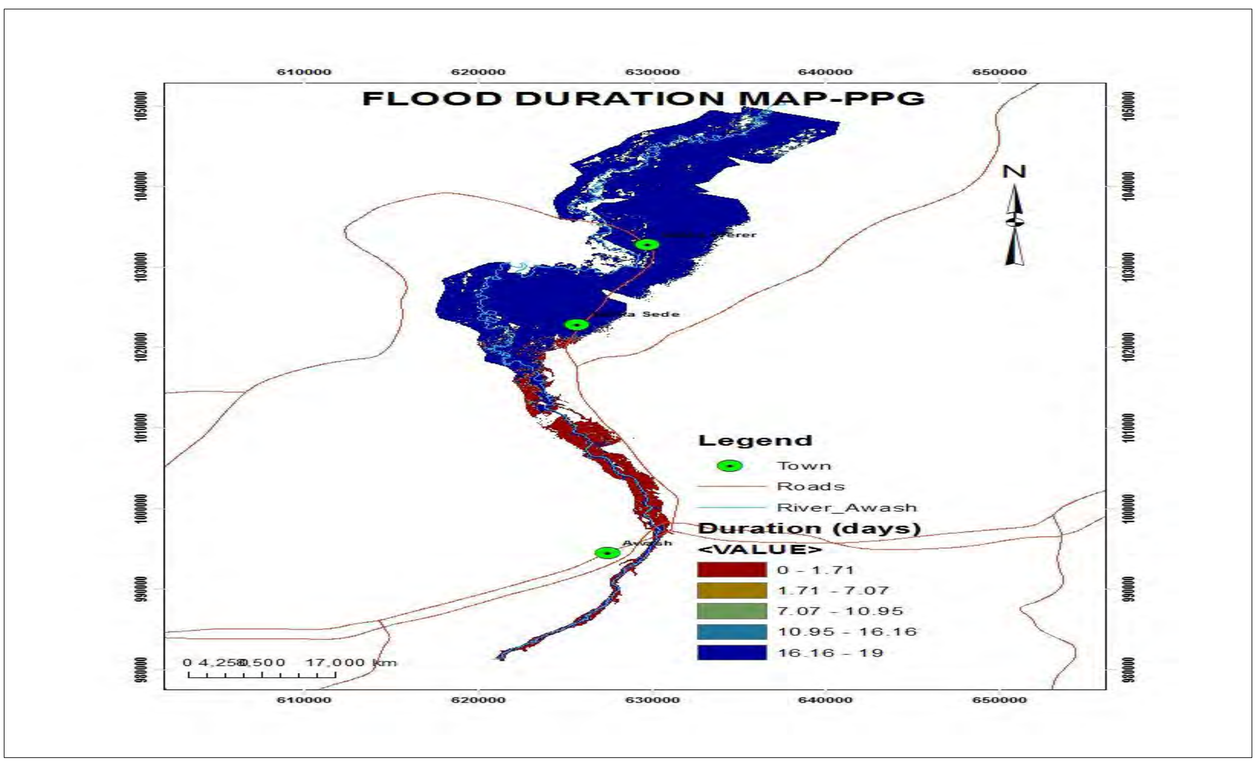

FLOOD FLOM MAP-OV
Melka Werer
Melka Sede
Awash
N
Legend
Town
River_Awash
Roads
Flow
<VALUE>
3,820.8 - 14,253.4
14,253.4 - 37,314.8
37,314.8 - 66,965.4
66,965.4 - 103,753.9
103,753.9 - 143,836.9
0 4,250 8,500 17,000 km

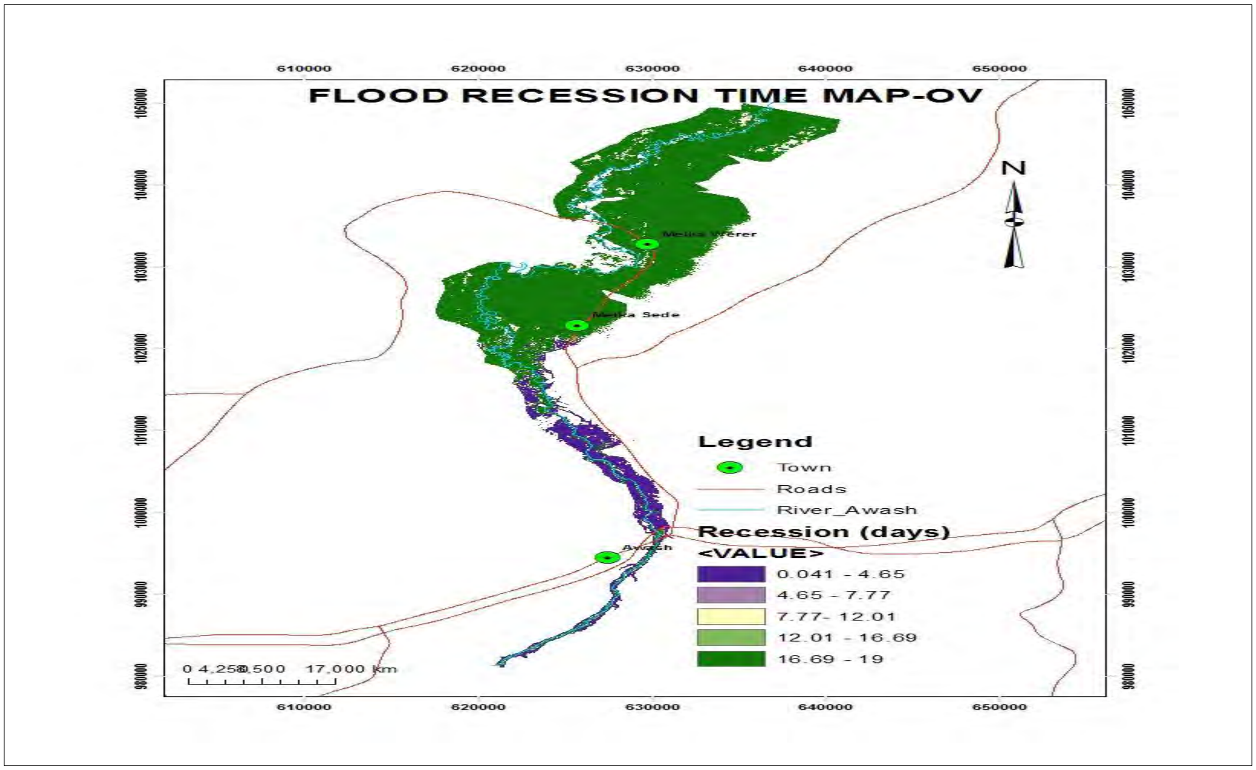

FLOOD FLOM MAP-PPG
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
Flow
<VALUE>
8,565.64 - 34,100.05
34,100.05 - 75,109.86
75,109.86 - 110,703.28
110,703.286 - 150,939.32
150,939.32 - 205,877
0 4,250 8,500 17,000 km

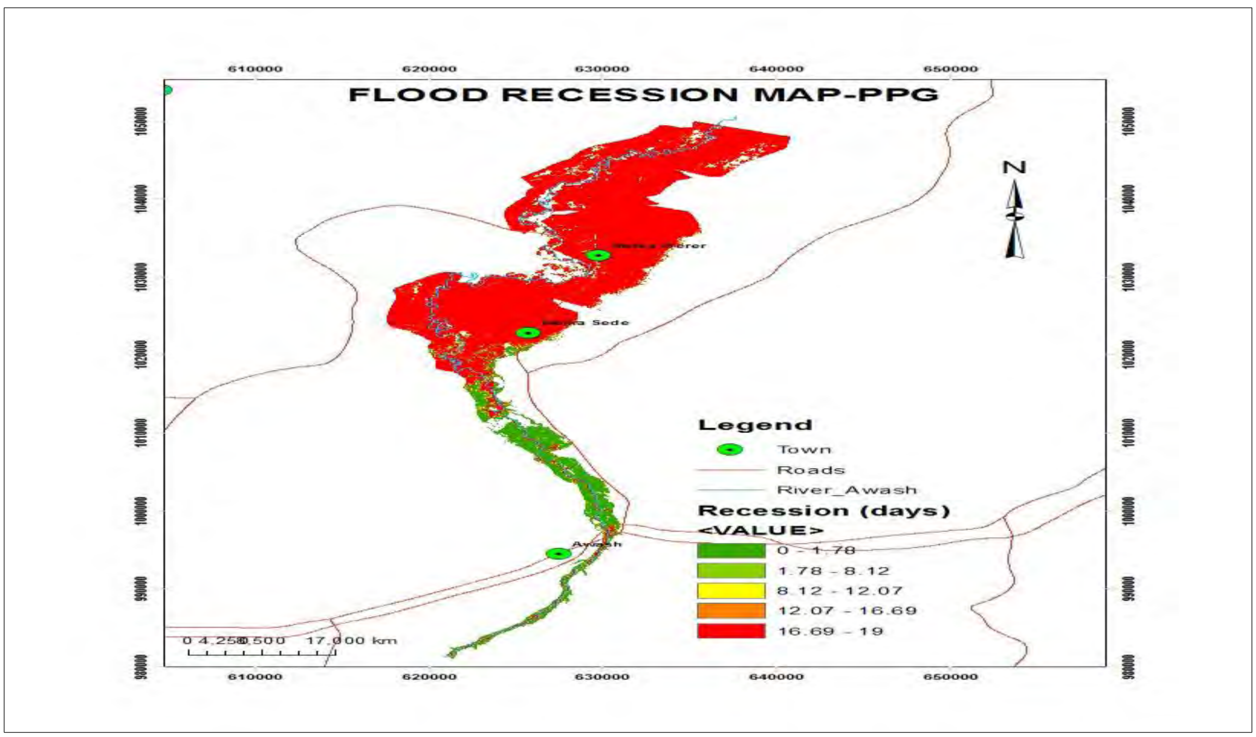

FLOOD ARRIVAL TIME MAP-OV
610000
620000
630000
640000
650000
N
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Arrival Time (days)
<VALUE>
0 - 0.5
0.5 - 1.3
1.3 - 2.12
2.12 - 2.87
2.87 - 3.70
0 4,250 8,500 17,000 km

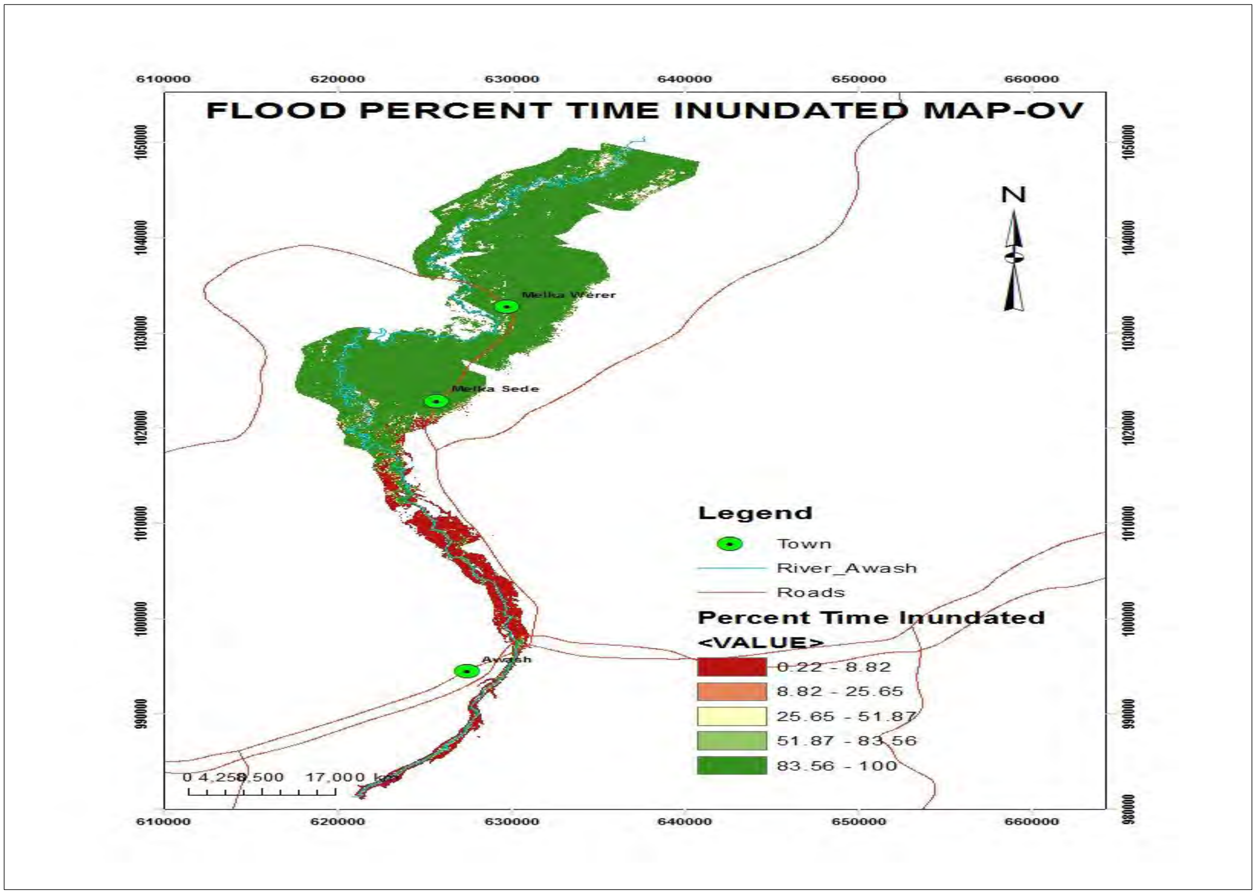

FLOOD ARRIVA TIME MAP-PPG
610000
620000
630000
640000
650000
N
Melka Werer
Melka Sede
Awash
Legend
Town
Roads
River_Awash
Arrival Time (days)
<VALUE>
0 - 0.32
0.32 - 0.90
0.90 - 1.33
1.33 - 1.92
1.92 - 5.5
0 4,250 8,500 17,000 km

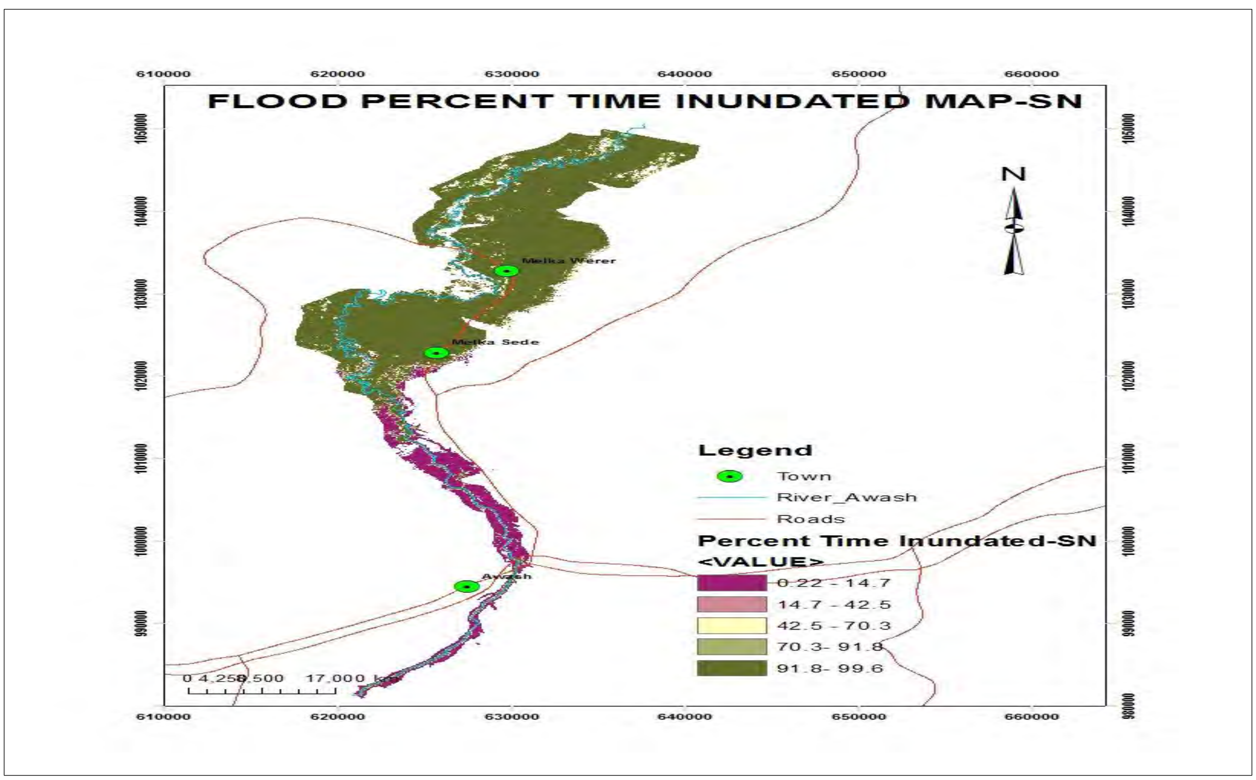

FLOOD DURATION MAP-OV
610000
620000
630000
640000
650000
N
Awash
Legend
Town
Roads
River_Awash
Duration (days)
<VALUE>
0 - 1.6
1.6 - 4.84
4.84 - 9.90
9.90 - 15.94
15.94 - 19
0 4,250 8,500 17,000 km

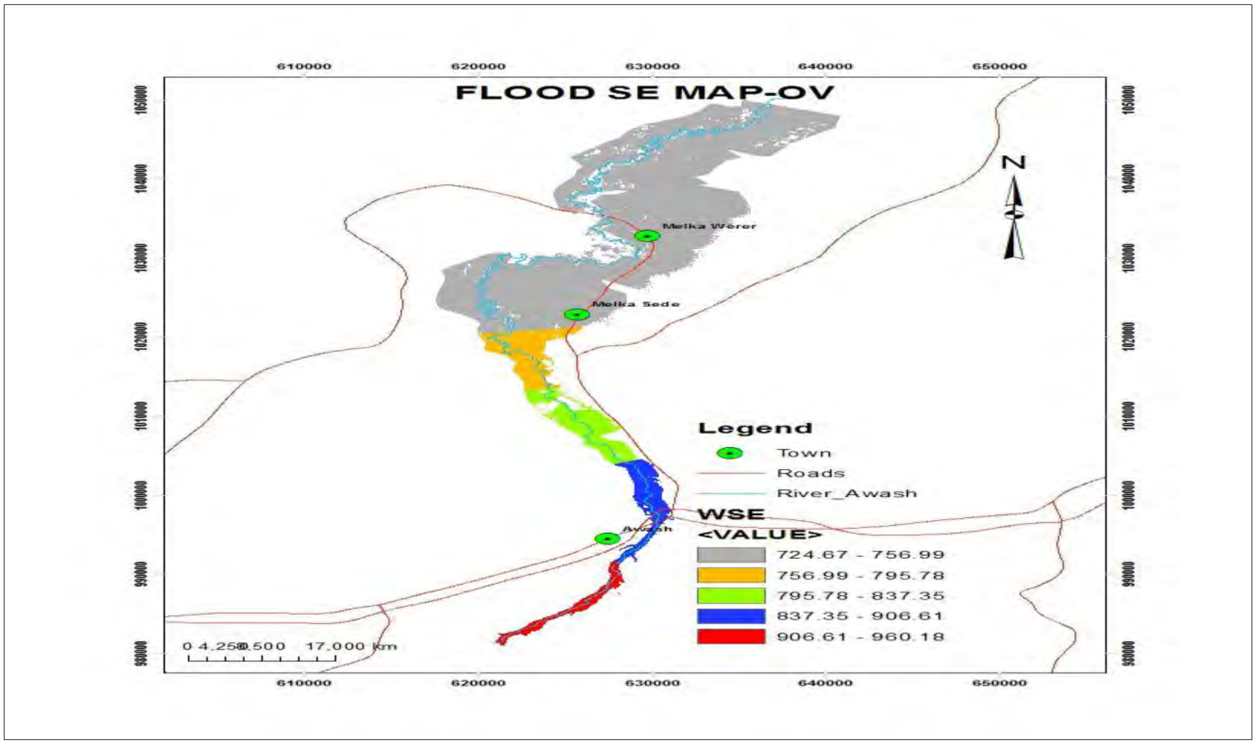

FLOOD DURATION MAP-PPG
610000
620000
630000
640000
650000
N
Legend
Town
Roads
River_Awash
Duration (days)
<VALUE>
0 - 1.71
1.71 - 7.07
7.07 - 10.95
10.95 - 16.16
16.16 - 19
0 4,250 8,500 17,000 km

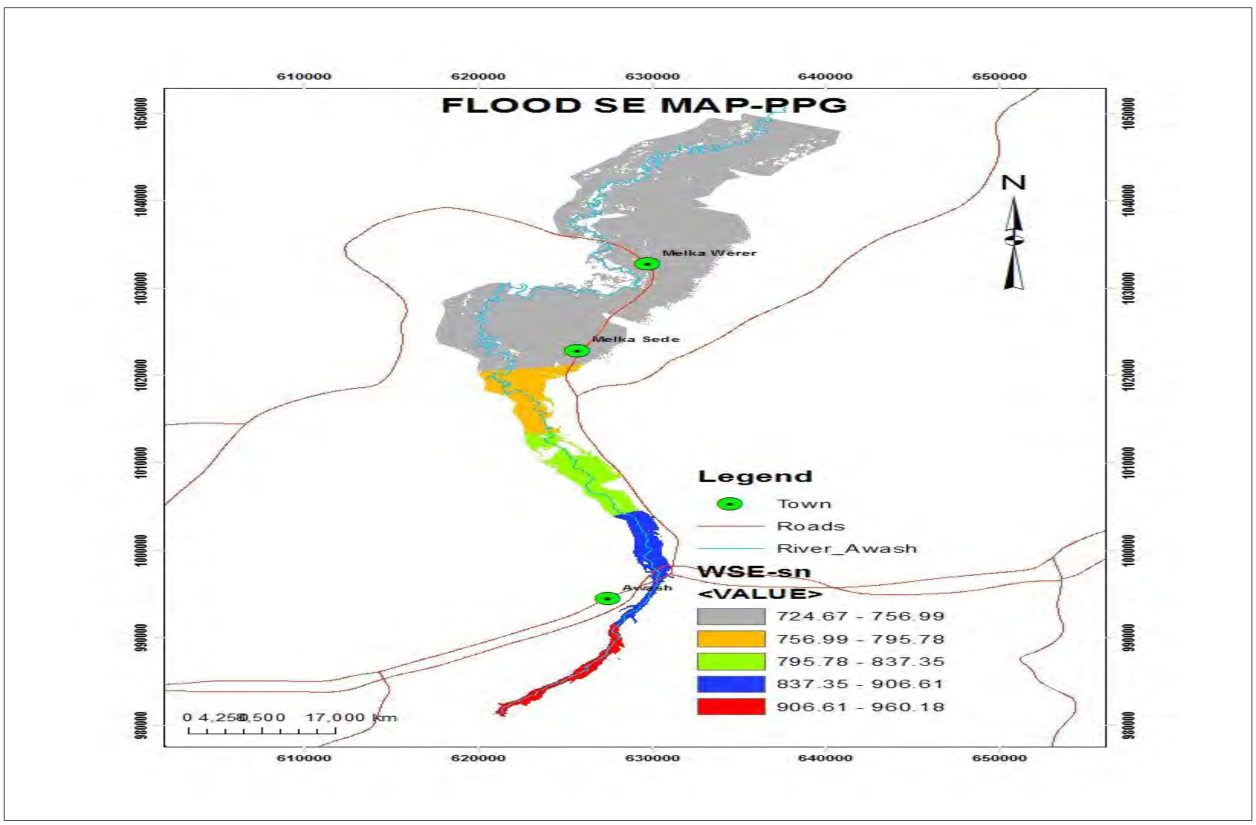

FLOOD RECESSION TIME MAP-OV
610000
620000
630000
640000
650000
N
Legend
Town
Roads
River_Awash
Recession (days)
<VALUE>
0.041 - 4.65
4.65 - 7.77
7.77- 12.01
12.01 - 16.69
16.69 - 19
Awash
0 4,250 8,500 17,000 km


FLOOD RECESSION MAP-PPG
610000
620000
630000
640000
650000
N
Legend
Town
Roads
River_Awash
Recession (days)
<VALUE>
0 - 1.78
1.78 - 8.12
8.12 - 12.07
12.07 - 16.69
16.69 - 19
Awash
0 4,250 8,500 17,000 km


FLOOD PERCENT TIME INUNDATED MAP-OV
Melka Werer
Melka Sede
Awash
N
Legend
Town
River_Awash
Roads
Percent Time Inundated
<VALUE>
0.22 - 8.82
8.82 - 25.65
25.65 - 51.87
51.87 - 83.56
83.56 - 100
0 4,258,500 17,000 km
610000
620000
630000
640000
650000
660000


FLOOD PERCENT TIME INUNDATED MAP-SN
Melka Werer
Melka Sede
Awash
N
Legend
Town
River_Awash
Roads
Percent Time Inundated-SN
<VALUE>
0.22 - 14.7
14.7 - 42.5
42.5 - 70.3
70.3- 91.8
91.8- 99.6
0 4,258,500 17,000 km
610000
620000
630000
640000
650000
660000


FLOOD SE MAP-OV
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
WSE
<VALUE>
724.67 - 756.99
756.99 - 795.78
795.78 - 837.35
837.35 - 906.61
906.61 - 960.18
610000
620000
630000
640000
650000


FLOOD SE MAP-PPG
Melka Werer
Melka Sede
Awash
N
Legend
Town
Roads
River_Awash
WSE-sn
<VALUE>
724.67 - 756.99
756.99 - 795.78
795.78 - 837.35
837.35 - 906.61
906.61 - 960.18
610000
620000
630000
640000
650000

9.2. Middle Awash PMF

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
9:00	292.969	11:00	4267.296	15:00	3193.752	19:00	2103.543
10:00	293.064	12:00	4271.671	16:00	3180.491	20:00	2094.89
11:00	293.627	13:00	4275.209	17:00	3167.275	21:00	2086.278
12:00	295.293	14:00	4277.983	18:00	3154.102	22:00	2077.706
13:00	298.7595	15:00	4279.977	19:00	3140.973	23:00	2069.171
14:00	305.1095	16:00	4281.182	20:00	3127.891	0:00	2060.675
15:00	316.5975	17:00	4281.671	21:00	3114.857	1:00	2052.22
16:00	336.107	18:00	4281.512	22:00	3101.867	2:00	2043.803
17:00	365.429	19:00	4280.683	23:00	3088.922	3:00	2035.417
18:00	405.1955	20:00	4279.162	0:00	3076.027	4:00	2027.06
19:00	456.5335	21:00	4277.021	1:00	3063.181	5:00	2018.732
20:00	521.699	22:00	4274.311	2:00	3050.381	6:00	2010.435
21:00	602.9785	23:00	4271.006	3:00	3037.627	7:00	2002.172
22:00	701.155	0:00	4267.098	4:00	3024.922	8:00	1993.944
23:00	816.1735	1:00	4262.667	5:00	3012.268	9:00	1985.755
0:00	948.172	2:00	4257.763	6:00	2999.661	10:00	1977.604
1:00	1098.286	3:00	4252.366	7:00	2987.101	11:00	1969.49
2:00	1258.6	4:00	4246.47	8:00	2974.591	12:00	1961.412
3:00	1425.488	5:00	4240.129	9:00	2962.133	13:00	1953.378
4:00	1595.601	6:00	4233.382	10:00	2949.724	14:00	1945.404
5:00	1769.435	7:00	4226.214	11:00	2937.362	15:00	1937.497
6:00	1944.691	8:00	4218.618	12:00	2925.051	16:00	1929.626
7:00	2108.005	9:00	4210.638	13:00	2912.793	17:00	1921.764
8:00	2265.451	10:00	4202.313	14:00	2900.583	18:00	1913.916
9:00	2411.71	11:00	4193.644	15:00	2888.419	19:00	1906.092
10:00	2543.13	12:00	4184.626	16:00	2876.308	20:00	1898.299
11:00	2658.566	13:00	4175.292	17:00	2864.248	21:00	1890.542
12:00	2758.235	14:00	4165.675	18:00	2852.237	22:00	1882.821
13:00	2843.317	15:00	4155.773	19:00	2840.273	23:00	1875.133
14:00	2915.188	16:00	4145.58	20:00	2828.362	0:00	1867.481
15:00	2974.438	17:00	4135.121	21:00	2816.503	1:00	1859.864
16:00	3022.018	18:00	4124.425	22:00	2804.693	2:00	1852.284

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
17:00	3059.883	19:00	4113.492	23:00	2792.93	3:00	1844.736
18:00	3090.107	20:00	4102.32	0:00	2781.218	4:00	1837.223
19:00	3114.245	21:00	4090.937	1:00	2769.557	5:00	1829.745
20:00	3133.789	22:00	4079.366	2:00	2757.945	6:00	1822.302
21:00	3150.041	23:00	4067.608	3:00	2746.381	7:00	1814.892
22:00	3163.836	0:00	4055.658	4:00	2734.867	8:00	1807.516
23:00	3175.489	1:00	4043.538	5:00	2723.406	9:00	1800.175
0:00	3185.141	2:00	4031.265	6:00	2711.994	10:00	1792.867
1:00	3192.888	3:00	4018.838	7:00	2700.629	11:00	1785.593
2:00	3198.699	4:00	4006.255	8:00	2689.314	12:00	1778.35
3:00	3202.386	5:00	3993.535	9:00	2678.05	13:00	1771.141
4:00	3204.052	6:00	3980.695	10:00	2666.834	14:00	1763.966
5:00	3204.108	7:00	3967.738	11:00	2655.665	15:00	1756.823
6:00	3202.914	8:00	3954.662	12:00	2644.546	16:00	1749.712
7:00	3200.694	9:00	3941.481	13:00	2633.478	17:00	1742.635
8:00	3197.726	10:00	3928.208	14:00	2622.457	18:00	1735.591
9:00	3194.293	11:00	3914.846	15:00	2611.484	19:00	1728.578
10:00	3190.575	12:00	3901.392	16:00	2600.56	20:00	1721.596
11:00	3186.679	13:00	3887.857	17:00	2589.687	21:00	1714.647
12:00	3182.886	14:00	3874.251	18:00	2578.861	22:00	1707.73
13:00	3180.054	15:00	3860.58	19:00	2568.08	23:00	1700.844
14:00	3179.824	16:00	3846.841	20:00	2557.348	0:00	1693.988
15:00	3184.675	17:00	3833.045	21:00	2546.665	1:00	1687.164
16:00	3196.131	18:00	3819.2	22:00	2536.029	2:00	1680.373
17:00	3213.81	19:00	3805.311	23:00	2525.439	3:00	1673.613
18:00	3236.677	20:00	3791.375	0:00	2514.897	4:00	1666.882
19:00	3264.098	21:00	3777.401	1:00	2504.404	5:00	1660.182
20:00	3295.281	22:00	3763.394	2:00	2493.959	6:00	1653.514
21:00	3329.183	23:00	3749.358	3:00	2483.565	7:00	1646.876
22:00	3365.018	0:00	3735.293	4:00	2473.246	8:00	1640.267
23:00	3402.383	1:00	3721.204	5:00	2463.007	9:00	1633.688
0:00	3440.931	2:00	3707.099	6:00	2452.812	10:00	1627.141
1:00	3480.23	3:00	3692.979	7:00	2442.633	11:00	1620.623
2:00	3519.782	4:00	3678.845	8:00	2432.473	12:00	1614.134
3:00	3559.089	5:00	3664.702	9:00	2422.349	13:00	1607.675

Middle Awash PMF							
Time	Q	Time	Q	Time	Q	Time	Q
4:00	3597.864	6:00	3650.553	10:00	2412.267	14:00	1601.247
5:00	3636.025	7:00	3636.401	11:00	2402.228	15:00	1594.847
6:00	3673.499	8:00	3622.248	12:00	2392.235	16:00	1588.476
7:00	3710.106	9:00	3608.096	13:00	2382.288	17:00	1582.134
8:00	3745.718	10:00	3593.95	14:00	2372.386	18:00	1576.236
9:00	3780.328	11:00	3579.811	15:00	2362.527	19:00	1569.974
10:00	3813.915	12:00	3565.681	16:00	2352.712	20:00	1563.715
11:00	3846.349	13:00	3551.563	17:00	2342.942	21:00	1557.485
12:00	3877.561	14:00	3537.459	18:00	2333.215	22:00	1551.285
13:00	3907.595	15:00	3523.37	19:00	2323.529	23:00	1545.112
14:00	3936.467	16:00	3509.299	20:00	2313.885	0:00	1538.966
15:00	3964.079	17:00	3495.247	21:00	2304.286	1:00	1532.848
16:00	3990.374	18:00	3481.215	22:00	2294.727	2:00	1526.759
17:00	4015.411	19:00	3467.206	23:00	2285.206	3:00	1520.698
18:00	4039.24	20:00	3453.222	0:00	2275.724	4:00	1514.663
19:00	4061.792	21:00	3439.263	1:00	2266.285	5:00	1508.656
20:00	4083.044	22:00	3425.332	2:00	2256.887	6:00	1502.678
21:00	4103.094	23:00	3411.428	3:00	2247.53	7:00	1496.727
22:00	4122.003	0:00	3397.554	4:00	2238.215	8:00	1490.802
23:00	4139.708	1:00	3383.713	5:00	2228.945	9:00	1484.905
0:00	4156.181	2:00	3369.902	6:00	2219.718	10:00	1479.037
1:00	4171.528	3:00	3356.124	7:00	2210.532	11:00	1473.194
2:00	4185.799	4:00	3342.382	8:00	2201.388	12:00	1467.377
3:00	4198.928	5:00	3328.677	9:00	2192.288	13:00	1461.588
4:00	4210.909	6:00	3315.007	10:00	2183.229	14:00	1455.825
5:00	4221.858	7:00	3301.373	11:00	2174.211	15:00	1450.089
6:00	4231.842	8:00	3287.779	12:00	2165.233	16:00	1444.378
7:00	4240.817	9:00	3274.224	13:00	2156.298	17:00	1438.694
8:00	4248.781	10:00	3260.709	14:00	2147.404	18:00	1433.037
9:00	4255.825	11:00	3247.232	15:00	2138.55	19:00	1427.406
10:00	4262.007	12:00	3233.798	16:00	2129.737	20:00	1421.799
	0	13:00	3220.408	17:00	2120.966	21:00	1416.219
	0	14:00	3207.059	18:00	2112.235	22:00	1410.665
	0		0		0	23:00	1405.137

9.3. Middle Awash Volume

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
821	591.0642	0	0	-	20.1043	0
822	3887.7328	2239.3985	2239.3985	2,239.40	28.7079	1
823	7051.1977	5469.46525	5469.46525	7,708.86	37.0005	2
824	12048.9865	9550.0921	9550.0921	17,258.96	43.5288	3
825	18486.6343	15267.8104	15267.8104	32,526.77	48.5649	4
826	24222.8807	21354.7575	21354.7575	53,881.52	53.5665	5
827	32127.3056	28175.0932	28175.09315	82,056.62	57.8792	6
828	40507.8955	36317.6006	36317.60055	118,374.22	62.1875	7
829	48005.9194	44256.9075	44256.90745	162,631.12	66.4957	8
830	54540.3721	51273.1458	51273.14575	213,904.27	70.2646	9
831	60514.1879	57527.28	57527.28	271,431.55	73.4954	10
832	67876.7093	64195.4486	64195.4486	335,627.00	76.7052	11
833	73801.0404	70838.8749	70838.87485	406,465.87	79.9143	12
834	97139.6831	85470.3618	85470.36175	491,936.24	83.1234	13
835	109932.857	103536.27	103536.2701	595,472.51	86.3324	14
836	123461.1768	116697.017	116697.0169	712,169.52	89.5129	15
837	137752.8201	130606.998	130606.9985	842,776.52	93.4736	16
838	152360.3035	145056.562	145056.5618	987,833.08	97.8895	17
839	167007.6403	159683.972	159683.9719	1,147,517.05	102.3025	18
840	200363.6649	183685.653	183685.6526	1,331,202.71	106.7113	19
841	225480.6699	212922.167	212922.1674	1,544,124.0	115.5007	20
842	252490.5225	238985.596	238985.5962	1,783,110.47	168.826	21

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
843	287151.0776	269820.8	269820.8001	2,052,931.27	181.6663	22
844	333898.9208	310524.999	310524.9992	2,363,456.27	193.1585	23
845	369292.288	351595.604	351595.6044	2,715,051.87	205.3197	24
846	406721.0278	388006.658	388006.6579	3,103,058.53	218.1153	25
847	446801.7165	426761.372	426761.3722	3,529,819.90	228.5584	26
848	554181.5199	500491.618	500491.6182	4,030,311.52	236.7833	27
849	626123.4892	590152.505	590152.5046	4,620,464.03	242.958	28
850	686284.1296	656203.809	656203.8094	5,276,667.84	248.52	29
851	750227.8482	718255.989	718255.9889	5,994,923.83	252.6816	30
852	820573.914	785400.881	785400.8811	6,780,324.71	256.2757	31
853	893678.9252	857126.42	857126.4196	7,637,451.13	259.8812	32
854	961983.1217	927831.023	927831.0235	8,565,282.15	262.9809	33
855	1032051.897	997017.509	997017.5092	9,562,299.66	265.4513	34
856	1104881.702	1068466.8	1068466.8	10,630,766.46	267.9032	35
857	1177252.222	1141066.96	1141066.962	11,771,833.42	270.3563	36
858	1250099.57	1213675.9	1213675.896	12,985,509.32	272.7852	37
859	1327214.506	1288657.04	1288657.038	14,274,166.35	275.0011	38
860	1405443.375	1366328.94	1366328.94	15,640,495.30	277.0965	39
861	2357579.477	1881511.43	1881511.426	17,522,006.72	279.168	40
862	2488163.764	2422871.62	2422871.621	19,944,878.34	281.2395	41
863	2623987.327	2556075.55	2556075.546	22,500,953.89	283.3111	42
864	2764880.02	2694433.67	2694433.673	25,195,387.56	285.3824	43
865	2946190.583	2855535.3	2855535.302	28,050,922	287.4537	44

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
866	3104947.871	3025569.23	3025569.227	31,076,492.09	289.5761	45
867	3265895.714	3185421.79	3185421.793	34,261,913.88	291.8011	46
868	3428110.993	3347003.35	3347003.354	37,608,917.24	294.0967	47
869	3585082.336	3506596.66	3506596.665	41,115,513.90	296.7493	48
870	3743959.358	3664520.85	3664520.847	44,780,034.75	299.6159	49
871	3896402.984	3820181.17	3820181.171	48,600,215.92	302.5569	50
872	4066266.247	3981334.62	3981334.615	52,581,550.53	305.7529	51
873	4227857.433	4147061.84	4147061.84	56,728,612.37	309.1325	52
874	4386212.56	4307035	4307034.996	61,035,647.37	312.5081	53
875	4542658.303	4464435.43	4464435.432	65,500,082.80	315.8807	54
876	4695026.088	4618842.2	4618842.196	70,118,925.00	319.2841	55
877	4847834.877	4771430.48	4771430.483	74,890,355.48	322.6998	56
878	5000455.073	4924144.98	4924144.975	79,814,500.46	326.7623	57
879	5184726.296	5092590.68	5092590.685	84,907,091.14	330.998	58
880	5348332.2	5266529.25	5266529.248	90,173,620.39	335.2569	59
881	5498008.099	5423170.15	5423170.15	95,596,790.54	339.3181	60
882	5635537.801	5566772.95	5566772.95	101,163,563.49	343.3681	61
883	5768271.988	5701904.89	5701904.895	106,865,468.38	347.1197	62
884	5901734.473	5835003.23	5835003.231	112,700,471.61	350.8517	63
885	6038913.858	5970324.17	5970324.166	118,670,795.78	354.5471	64
886	6169907.64	6104410.75	6104410.749	124,775,206.53	358.2091	65
887	6300579.218	6235243.43	6235243.429	131,010,449.96	361.8717	66
888	6436724.353	6368651.79	6368651.786	137,379,101	365.5341	67

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
889	6572571.447	6504647.9	6504647.9	143,883,749.64	369.1249	68
890	6720317.381	6646444.41	6646444.414	150,530,194.06	372.8904	69
891	6859463.731	6789890.56	6789890.556	157,320,084.61	376.5965	70
892	6996913.09	6928188.41	6928188.41	164,248,273.02	380.3057	71
893	7126006.469	7061459.78	7061459.779	171,309,732.80	384.0149	72
894	7250783.13	7188394.8	7188394.8	178,498,127.60	387.7213	73
895	7371608.709	7311195.92	7311195.92	185,809,323.52	391.048	74
896	7492988.546	7432298.63	7432298.627	193,241,622.15	393.7936	75
897	8216240.365	7854614.46	7854614.455	201,096,236.60	396.5095	76
898	8362147.348	8289193.86	8289193.856	209,385,430.46	399.2481	77
899	8511637.08	8436892.21	8436892.214	217,822,322.67	401.9762	78
900	8653054.323	8582345.7	8582345.702	226,404,668.38	404.7028	79
901	8791962.931	8722508.63	8722508.627	235,127,177.00	407.3564	80
902	8941094.047	8866528.49	8866528.489	243,993,705.49	409.9596	81
903	9085347.914	9013220.98	9013220.98	253,006,926.47	412.5627	82
904	9247779.737	9166563.83	9166563.825	262,173,490.30	415.1576	83
905	9389454.229	9318616.98	9318616.983	271,492,107.28	417.7826	84
906	9534390.867	9461922.55	9461922.548	280,954,029.83	420.5161	85
907	9669099.838	9601745.35	9601745.352	290,555,775.18	423.2212	86
908	9804422.625	9736761.23	9736761.231	300,292,536.41	425.9258	87
909	9942238.124	9873330.37	9873330.375	310,165,866.79	428.6309	88
910	10125427.65	10033832.9	10033832.89	320,199,699.67	431.3363	89
911	10258991.71	10192209.7	10192209.68	330,391,909	434.042	90

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
912	10395157.06	10327074.4	10327074.38	340,718,983.74	436.7477	91
913	10537767.87	10466462.5	10466462.46	351,185,446.20	439.441	92
914	10677170.97	10607469.4	10607469.42	361,792,915.62	442.8257	93
915	10838424.46	10757797.7	10757797.71	372,550,713.33	447.084	94
916	10986345.16	10912384.8	10912384.81	383,463,098.13	451.1967	95
917	11136516.14	11061430.6	11061430.65	394,524,528.78	455.2691	96
918	11281518.49	11209017.3	11209017.32	405,733,546.10	459.1173	97
919	11419535.11	11350526.8	11350526.8	417,084,072.90	462.5048	98
920	11609746.74	11514640.9	11514640.92	428,598,713.82	465.8482	99
921	11759671.07	11684708.9	11684708.9	440,283,422.72	469.199	100
922	11911085.71	11835378.4	11835378.39	452,118,801.11	472.5491	101
923	12052983.5	11982034.6	11982034.61	464,100,835.72	475.8849	102
924	12198196.9	12125590.2	12125590.2	476,226,425.92	479.1528	103
925	12344589.76	12271393.3	12271393.33	488,497,819.25	482.3581	104
926	12507769.81	12426179.8	12426179.79	500,923,999.04	485.5851	105
927	12666745.25	12587257.5	12587257.53	513,511,256.57	488.8752	106
928	12902254.66	12784500	12784499.96	526,295,756.53	492.2296	107
929	13095449.76	12998852.2	12998852.21	539,294,608.74	495.5168	108
930	13293573.07	13194511.4	13194511.41	552,489,120.16	498.7246	109
931	13486780.65	13390176.9	13390176.86	565,879,297.02	501.8982	110
932	13727144.3	13606962.5	13606962.47	579,486,259.49	505.0713	111
933	13933404.88	13830274.6	13830274.59	593,316,534.07	508.4633	112
934	14154450.97	14043927.9	14043927.92	607,360,462	511.9001	113

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
935	14333935.01	14244193	14244192.99	621,604,654.99	515.2738	114
936	14516463.36	14425199.2	14425199.18	636,029,854.17	518.5641	115
937	14740538.59	14628501	14628500.97	650,658,355.14	521.7788	116
938	14930100.62	14835319.6	14835319.6	665,493,674.74	524.98	117
939	15115696.14	15022898.4	15022898.38	680,516,573.12	528.4333	118
940	15297284.38	15206490.3	15206490.26	695,723,063.38	531.8448	119
941	15479319.88	15388302.1	15388302.13	711,111,365.51	535.1497	120
942	15662183.92	15570751.9	15570751.9	726,682,117.41	538.4751	121
943	15870450.97	15766317.4	15766317.45	742,448,434.85	541.7265	122
944	16086050.94	15978251	15978250.96	758,426,685.81	545.7146	123
945	16334615.03	16210333	16210332.99	774,637,018.80	549.9058	124
946	16589254.17	16461934.6	16461934.6	791,098,953.40	554.0968	125
947	19295800.35	17942527.3	17942527.26	809,041,480.66	558.1797	126
948	21913195.08	20604497.7	20604497.71	829,645,978.37	562.0447	127
949	27338642.14	24625918.6	24625918.61	854,271,896.98	566.0401	128
950	32508422.64	29923532.4	29923532.39	884,195,429.37	570.2715	129
951	47606165.62	40057294.1	40057294.13	924,252,723.50	574.7674	130
952	79942339.18	63774252.4	63774252.4	988,026,975.90	579.2107	131
953	94357889.79	87150114.5	87150114.49	1,075,177,090.38	583.5838	132
954	111503951	102930920	102930920.4	1,178,108,010.79	587.9541	133
955	125796552.6	118650252	118650251.8	1,296,758,262.62	593.1929	134
956	138748430.5	132272492	132272491.6	1,429,030,754.18	599.3797	135
957	148876413.1	143812422	143812421.8	1,572,843,175	605.8756	136

Middle Awash Volume						
ELEVATION	AREA	AV.AREA	PAR.VOLUME	TOTAL VOL.	CRUST LENG.	DAM HEIGHT
958	158479830.6	153678122	153678121.9	1,726,521,297.85	612.5823	137
959	169427955.1	163953893	163953892.9	1,890,475,190.73	618.755	138
960	179551726.2	174489841	174489840.7	2,064,965,031.41	625.8409	139
961	188274176.7	183912951	183912951.5	2,248,877,982.86	645.8167	140
962	196208067.1	192241122	192241121.9	2,441,119,104.75	676.7237	141
963	205458463.8	200833265	200833265.4	2,641,952,370.17	714.2359	142
964	215069047.4	210263756	210263755.6	2,852,216,125.77	773.7969	143