



ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

DAM BREAK ANALYSIS AND RISK ASSESSMENT

[A CASE STUDY OF TENDAHO DAM]

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

By

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December 2015

Addis Ababa Institute of Technology

School of Graduate Studies

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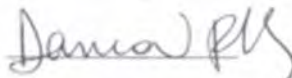
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Certification

I, the undersigned, certify that I read and hear and recommend for acceptance by Addis Ababa Institute of Technology a dissertation entitled “Dam Break Analysis and Risk Assessment (A Case Study of Tendaho Dam)” in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Major Hydraulic Engineering).

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Declaration

I, Tariku Tadesse Galasha, declare that this dissertation is my own original work and that it has not been presented and will not be presented to any other University for similar or any other degree award.

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Abstract

This thesis analyzes the probable failure of a dam under a set of pre-defined scenarios, within the framework of a case study, the case subject being the Tendaho dam located at Afar Region of Ethiopia. The failure of the dam is not analyzed neither structural nor hydraulic wise but, is assumed to be triggered when certain critical WSELs are exceeded. Hence, the analyses focused on the aftermath of the failure and strive to anticipate the level of inundation at downstream of the dam itself. Break parameters prediction, peak outflow prediction were shown as the essential for the dam break analysis and eventually determined the loss of the life. Tendaho dam break was further modeled and analyzed using USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS Version 4.1) model based on available technical and geometric data. Water surface elevations and discharges occurring at downstream of the Tendaho dam were predicted spatially and temporally under the defined set of eleven scenarios. Eight of the scenarios were analyzed for overtopping failure of the dam. Two of the scenarios were analyzed for piping failure and the last scenario was analyzed to investigate the sensitivity of Manning roughness. Overtopping failure yielded more severe flow conditions at downstream. The worst case scenario was found to be scenario W4 (when all the three spillway gates are completely closed). Hence, indicative inundation map and settlements under risk were identified. Total numbers of fatalities for precise and vague understanding of warnings were found 1,012 and 2,972 respectively. The thesis study further addressed some available pre-event measures that may be taken in advance.

Keywords: Tendaho Dam, HEC-RAS, HEC-GeoRAS, Breach, Dam Failure, Dam Break Analysis

Dedication

I dedicate this thesis work

To my son Ogdon

And

To my baby girl Mariamawit

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List of Abbreviations

m.a.m.s.l.	Meter above mean sea level
AV	Agricultural Village
B	Average breach bottom width
BC	Boundary Condition
CSA	Central Statistical Agency
DEM	Digital Elevation Model
D/s	Downstream
EAP	Emergency Action Plan
EMA	Ethiopian Mapping Agency
FEM	Finite Element Model
Fig	Figure
ft	feet
FRL	Full Retention Level
FSL	Full Supply Level
GIS	Geographic Information System
GPS	Global Positioning System
ha	Hectare
HEC	Hydrologic Engineering Center
HEC-GeoRAS	Hydrologic Engineering Center's for Geospatial River Analysis System
HEC-RAS	Hydrologic Engineering Center's for River Analysis System
hr	Hour
IC	Initial Condition
Km	Kilometer
Km ²	Kilometer square
m	Meter
m ³	Cubic meters
m ³ /s	Cubic meter per second
Max	Maximum
Max WS	Maximum Water Surface
MDDL	Minimum Draw Down Level

Min	Minimum
MMC	Million Metric Cubes
MoWIE	Ministry of Water, Irrigation and Electricity
MWL	Maximum Water Level
NA	Not Applicable
P	Piping
PCS	Projected Coordinate System
PMF	Probable Maximum Flood
Q_p	Peak Discharge
RS	River Station
S	Summer
Sc	Scenario
SC	Sugar Corporation
SWE	Shallow Water Flow Equations
t	time
U/s	Upstream
USACE	US Army Corps of Engineers
W	Winter
WS	Water Surface
WSEL	Water Surface Elevation
WWDSE	Water Works Design and Supervision Enterprise
XS	Cross-Section
XST	Cross-Section of Tributary

CHAPTER ONE

1. INTRODUCTION

1.1 Background

The term dam break analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting consequences at downstream region. Dam break analysis generally deals with simulation of probable failure for existing dams and analyzing the resulting consequences [19]. Dam break modeling is typically done within a larger study context that develops inflow hydrographs from various frequency storms, evaluates project spillway adequacy, estimates breach parameters, and performs routing and mapping of the resultant flood [7]. As land pressures for human settlement and development into down land region increase, the risk of human fatality and damage to assets and infrastructure from the rapid failure of large stream blocking events (dams) increase. Despite great benefit to people, dams might also lead to catastrophes such as dam failures because of several reasons including but, not limited to, failure to carry out necessary maintenance, lack of proper risk assessments, occurrence of an unforeseen runoff flood of great magnitude, piping of embankment materials, or incompetent/improper operation.

In the event of failure of a dam by overtopping /or piping, a breach is formed on the dam hull, which gradually expands in terms of hours or even minutes. The water stored behind the reservoir is then set free uncontrollably in great amounts and causes damage along its route in great proportions. A flood caused by a dam breach may be much larger in magnitude and behave much differently than a natural flood [20]. In the absence of preemptive measures geared towards such a risk, such as an emergency action plan, settlements and the properties located downstream of the dam and on the route of such a dam failure flood are put to great risk. Dam break analyses provide information on the consequences of a possible dam failure for risk estimation and rescue planning purposes. The development of effective emergency action plans and the design of early warning systems heavily rely on prediction results [16].

Currently, the exploitation of irrigation and hydropower potentials has been recognized by the Ethiopian government as key issues in the economic development

of the country. To meet the strong increase of food and energy demands in future years, the Ethiopian government has made large-scale investments to large dam projects particularly on the river of Awash river basin for sugar cane irrigation and on the rivers of Blue Nile and Omo-Gibe river basins for energy (including the current Great Ethiopian Renaissance Dam). However, the country cannot fully mitigate the impact of water infrastructure development through capital investments alone.

Tendaho dam and irrigation development project for sugar cane plantation is one of the Mega-projects undertaken by Ministry of Water, Irrigation and Electricity to irrigate gross land of 60,000ha of sugar cane and is targeted to produce 500,000 tons of sugar annually. A Zoned earthen dam was constructed across Awash River at Tendaho to store $1.86 \times 10^9 \text{m}^3$ of water at its FSL for sugar cane irrigation found at downstream of the dam [36]. This large stored volume of water makes Tendaho dam to be categorized as a large earth-fill dam. Hence, it often poses a higher hazard potential than any other embankment dams built in the country, given the extreme storage volumes and downstream population and properties.

But, if a development agenda risks human life it is not a development strategy. Thus, implementation of such a large dam needs a complete risk analysis and the identification of pre-event measures. It is, however, quite possible to make risk assessment and to forecast dam break floods for already constructed dams so as to identify the risks involved in order to plan for the future and make wise decisions on necessary maintenances and operation of dam and reservoir respectively.

In general, an analysis of a potential dam failure includes two primary tasks [25]: (1) the prediction of the reservoir outflow hydrograph and (2) the routing of that hydrograph through the downstream valley to determine dam failure consequences (i.e. inundated regions). There are three viable dam break characteristics prediction methods [7]: (1) comparison analysis method: comparing a dam under study with recorded failed dams of similar size and type, (2) empirical equations: use of different equations developed based on historical records of failed dams, and (3) physical based models: use of computer based models that model physical breaching process using sediment transport equations, soil mechanics, and principles of hydraulics. Dam break can be model with numerical/parametric models. Numerical models use computer program to model dam breach and are further classified as hydrologic and hydraulic

models. The Hydrologic models include HEC-HMS and HEC-1 and hydraulic models include HEC-RAS, NWS SMPDBK, NWS DAMBRK, NWS and FLDWAV.

The dam break analysis can best be modeled with hydraulic models, whereas, empirical equations are used to estimate input dam breach parameters to hydraulic models [6]. The governing equations of hydraulic models are conservation of mass and momentum equations. The models use these two equations to simulate unsteady state dam break floods and to calculate hydraulic conditions at downstream region. HEC-RAS (Hydrologic Engineering Center for River Analysis System) software is a widely used hydraulic model by several researchers (including at USA) for dam break analysis. In this work, the HEC-RAS Version 4.1 model is used to simulate the case study dam failure and empirical equations are used to predict breach parameters and fatalities at downstream region. The study delineated hereunder, however, is meant to approach the problem from the probable failure of the Tendaho dam under a set of pre-defined eleven scenarios and is solely focused on the risk thereof, the forecast of the dam break floods, the inundated areas, and the settlements at risk using mainly HEC-RAS and HEC-GeoRAS models.

1.2 Problem Statement

Like for other dams in Ethiopia, dam break analysis for Tendaho dam is not studied yet. However, it often poses a higher hazard potential than any other embankment dams built in the country, given the extreme storage volumes of $1.86 \times 10^9 \text{ m}^3$ [35] and downstream extensive, densely populated communities of about 83,000 residents [3] and of considerable economic importance.

The main peoples in the dam break flood risks are found in the existing towns: Logia, Dubti, Assaita and in the newly built Agricultural Villages (AV), Sub-Heads and Head-Quarter of sugar development project. The main properties that are at flood prone are irrigation canals and structures, sugar cane plantations, roads and bridges, schools, health centers, hospital, Sugar Factory etc. Furthermore, the project area is at natural flooding risk by the River Awash, which has a tendency to change its course very often [34] even if a dam failure doesn't occur; however, no great research effort has been put into evaluation of natural flooding problems in the area.

Although there has not yet been any large dam failure in Ethiopia so far, there are several examples throughout the world. For instance, in USA, the 1889 South Fork

dam failure has resulted in the death of 2,209 people and the 1976 Teton dam failure has resulted in the death of 11 people and economic loss of US\$400 million [32]. Simulation of such a dam break event and its resulting flood is crucial to characterizing and identifying threats due to potential failure. However, these will be possible by preparing inundation map which accurately predict dam breach flood depths and arrival times at critical locations.

Hence, this thesis study is attempted to fill this research gaps. As far as the dam is concerned, unless amount of dam breaking flood is estimated and its possible downstream flood inundation map is developed, emergency action plan (EAP) preparation will be impossible.

1.3 Research Questions

After completion of Tendaho dam break simulation, the following main questions will be answered:

1. How much flow will be released to downstream region for each scenario if the Tendaho dam were to breach?
2. Which downstream areas are more prone to dam break flood?
3. How long will it take a dam break flood to arrive at each key settlement on the route?
4. Are there any effective flood risk mitigation tools adopted so far to save loss of life and property?

1.4 Objectives of the Study

1.4.1 General Objective

The main objective of the study is to predict Tendaho dam break floods and to assess resulting risks at downstream regions.

1.4.2 Specific Objectives

The specific objectives of the study delineated hereunder are, therefore, to:

- 🚧 Predict the dam break peak outflows produced at Tendaho dam
- 🚧 Identify the inundated areas and the corresponding spatial and time-wise water depths and discharges at critical river cross-sections

- ✚ Identify the settlements at risk judging by the time of arrival of the dam break flood at each settlement on the route
- ✚ Present preliminary suggestive measures that could be implemented in areas under risk

1.5 Significance of Study Outcomes

An attempt is made to predict the Tendaho dam break flood and identify possible flood hazard zones at downstream region. A preliminary flood inundation map was developed for identified flood hazard zones. Hence, direct model outputs together with inundation map are assumed to assist the dam owner (MoWIE) to plan existing irrigation land use and for future expansion, to use proposed flood mitigation tools (structures) to protect loss of life and property, and in developing a detailed emergency action plans (EAP) to mitigate flood damage.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 General

Guidelines for dam breach analysis [6] defines, dam is a man-made barrier, together with appurtenant structures, constructed above the natural surface of the ground for the purpose of impounding water. Dams are an important part of this nation's infrastructure, providing flood control, water supply, irrigation, hydropower, navigation, and recreation benefits. Despite their many beneficial uses and value, dams also present risks to property and life due to their potential to fail and cause catastrophic flooding [25]. Dam failure can cause loss of life, property damage, cultural and historic losses, environmental losses as well as social impacts [37].

Sever loss of life and property were recorded in many countries due to failure of large dams and their examples will be presented in next section. A large dam has a height of not less than 15 m or stores more than three million cubic meters of water in the reservoir [13]. Dams are of numerous types, and type classification is sometimes less clearly defined. An initial broad classification into two generic groups can be made in terms of the principal construction material employed [18]:

1. Embankment dams are constructed of earth-fill and/or rock-fill. Upstream and downstream face slopes are similar and of moderate angle, giving a wide section and a high construction volume relative to height.
2. Concrete dams are constructed of mass concrete. Face slopes are dissimilar, generally steep downstream and near vertical upstream, and dams have relatively slender profiles dependent upon the type.

Embankment dams have served man at least 5,000 years [33]. Today, embankment dams exist in excess of 300 meters high with volumes of many millions of cubic meters of fill. Thousands of embankment dams exceeding 20 meters in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction. Embankment dams are numerically dominant for technical and economic reasons, and account for an estimated 80% of all dams built [13].

An earth-fill dam is an embankment dam, constructed primarily of compacted earth, either homogeneous or zoned, and containing more than 50% of earth fill in volume.

Earth dams require special attention to details: for example, the design and treatment of filters, drains, and interfaces may be critical to the control of concentrated leaks with destructive potential [34]. Until modern times, all earth-fill dams were designed by empirical methods, and engineering literature is filled with accounts of failures. These failures brought on the realization that empirical methods must be replaced by rational engineering procedures for both the design and construction of earth-fill dams [30].

In general, as to minimize a probability of an earth dam failure, the successful designing and construction of an earth dam should fulfill the following technical and administrative requirements [26]:

Technical requirements

- ✚ Dam foundation and abutment must be stable at all static and dynamic loading conditions.
- ✚ Should have a special design for control and collect seepage through the foundation, abutments and embankment.
- ✚ The outlet capacity of the spillway must be sufficient to prevent overtopping of embankment by reservoir

Administrative requirements

- ✚ Environmental responsibility
- ✚ Detailed operation and maintenance methods
- ✚ Monitoring plans
- ✚ Adequate instrumentation
- ✚ Records for all operation and maintenance activities
- ✚ Emergency action plan (include identification and immediate response)

2.2 Dam Failures

The term "dam failure" suggests a single event; a quickly rising super flood of fast flowing water presumably varying in scale (depending on the volume of water released) and in human impact (depending on the size of the downstream population) but otherwise of similar character wherever it may occur. In fact, of course, dam-failure floods can be quite different depending on both environmental conditions and mode of failure, and these differences are likely to be of great significance in terms of

defining appropriate emergency responses. In effect, different failure modes and different environments present quite different planning problems [6]. Ideally, of course, the plans which are drawn up must create a capacity for effective response regardless of environment or the precise nature of the flood which occurs. Given the existence of a plan or "family" of plans for each below dam area for which evacuation operations may be needed, the environmental issue is necessarily built in to the planning problem on a case-by-case basis. Variation by failure mode presents more serious complications, however [4].

Historically, many of the early dams were small, in-channel structures built by locals with little or no engineering background. Griffen (1974) suggests that dam safety for early dams was less of a concern because the areas around early dams were less densely populated and, therefore, few people were directly affected by the dams, these dams were generally small in relation to modern dams, and the dams were generally built by cultures who took pride in their work [14]. Loss of life sometimes results from dam failure. Loss of life is likely if a dam fails without warning and the failure produces flooding that destroys residential structures [37].

However, modern dam-safety analysis has been an evolving science since the failure of Teton dam (USA) in 1976, there has been a significant evolution in the understanding of floods, dams and other critical infrastructures on which modern society and people's wellbeing rely. Regardless of the reason of failure, the event of a dam failure is quite a disaster, even catastrophic in the absence of pre-event measures in place. However, until very recently, dams and reservoir safety had focused almost exclusively on the structure or the civil works and this, with an eminently determinist approach. In this respect, risks were controlled through the respect of certain rules and practices sanctioned by experience, as well as by checking safety factors that were understood as conservative measures of prudence [21].

Although there has not yet been any recorded large dam failures that has resulted loss of life and property in Ethiopia so far, there are several examples throughout the world. Among the recorded dam failures and resulted fatalities lists in guidelines for estimating loss of life for dam failure scenarios [32] the following are selected by the Author to emphasize the need of case study dam break simulation.

Table 2-1 Examples of the dam failures resulted in fatalities

Dam	Date and time of failure	Height (ft)	Volume Released (acre-ft)	Loss of life	Economic loss (million \$)
Williamsburg Dam, MA (Mill River Dam)	May 16, 1874 at 7:20 a.m.	43	307	138	Not available
South Fork Dam, PA, (Johnstown Dam) Walnut	May 31, 1889 at 3:10 p.m.	72	11,500	2,209	Not available
St. Francis Dam, CA	March 12-13, 1928 at midnight	188	38,000	420	14
Buffalo Creek, WV (Coal Waste Dam)	Feb. 26, 1972 at 8:00 a.m.	46	404	125	50
Black Hills Flood, SD (Canyon Lake Dam)	June 9, 1972 at about 11 p.m.	20	700	NA	160
Teton Dam, ID	June 5, 1976 at 11:57 a.m.	305	250,000	11	400
Kelly Barnes Dam, GA	Nov. 6, 1977 at 1:20 a.m.	40	630	39	3
Lawn Lake Dam, CO	July 15, 1982 at 5:30 a.m.	26	674	3	31



Figure 2-1 Example of dam failure breach (South Fork dam failure)

A review of the data from the 1975 and 1988 ASCE/USCOLD studies indicates that about 40 percent of failures and accidents to embankment dams are the result of leakage and piping through the dam, foundation, and/or the abutments. Flood discharge and/or overtopping and washout of the dam are a second major cause of failures and accidents. Slides within the abutments or the embankment slopes caused by a high phreatic surface within the downstream slope, drawdown of the reservoir, or earthquake are another major cause of failures and accidents to embankment dams [32]. Despite the fact that the main modes of failure have been identified as piping or overtopping, the actual failure mechanics are not well understood for either earthen or concrete dams [38]. However, breach forming mechanisms can be classified as [6]:

1. Breaches formed by the sudden removal of all or a portion of the impounding structure as a result of some over-stressing of the structure (for concrete or other rigid dams), and
2. Breaches formed by erosion of embankment material (overtopping and piping failure).

As per guidelines for dam breach analysis [6], “breach” is defined as the opening formed in the dam body that leads the dam to fail and this phenomenon causes the concentrated water behind the dam to propagate towards downstream regions.

In general, it is deduced that a dam failure may result in loss of life and economy. Hence, predicting dam failure floods and resulting flood hazards for existing and proposed dams is vital for emergency action planning (EAP) preparation for the settlements located at downstream of the dam.

2.3 Dam Failure Hazard Classification

A dam hazard classification is the placement of a dam into one of three/or four categories based on the hazard potential derived from an evaluation of the probable incremental adverse consequences due to failure or improper operation of the dam. Characterization of the threat to public safety (people at risk) that a dam poses establishes the hazard classification of the dam and the associated standard of care to which the dam is held. “People at risk” is defined as the number of people in the dam failure floodplain prior to the issuance of any flood or dam failure warnings [37].

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 (flood warning time) at critical locations. Adverse effects of a dam failure can be classified in to four categories [21].

Damage to people: in principle, apart from loss of life, damage to people could also consider other aspects such as people injured with different degrees of gravity. However, due to the difficulty of quantification of wounded numbers, quantitative analysis usually focuses only on the first aspect.

Direct economic damage: damage caused directly by the impact of the flood and is the most visible type. It includes the cost associated with the damage suffered by the dam itself.

Indirect economic damage: damage happened after the event as a result of the interruption of the economy and other activities in the area.


Other damages: these are related to environmental damage, social disturbing, loss of reputation, damages to historical or cultural heritage, etc. However, all of these aspects are difficult to quantify and so are usually treated in a qualitative way.

FEMA guidelines [31] recommends a three-step rating system that defines low-hazard, significant-hazard and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure.

Table 2-2 Dam hazard potential classification system

Hazard potential	Loss of human life	Economic, environmental, lifeline losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable, one or more expected	Yes

Moreover, dams can be classified in terms of potential hazards in to three groups based on their severity as follows [22].

 **Low hazard potential:** dams assigned the low hazard potential classification are those where failure or miss-operation results in no probable loss of human life and low economic losses, low environmental damage, and no significant disruption of lifeline facilities. Losses are principally limited to the owner’s property.

🚧 **Significant hazard potential:** dams assigned the significant hazard potential classification are those dams where failure or miss-operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns.

🚧 **High hazard potential:** dams assigned the high hazard potential classification are those where failure or miss-operation will probably cause in loss of one or more human lives.

Firat Ba [11] further defines lifelines as transportation systems (highways, airports, rail lines, waterways, ports and harbor facilities) and utility systems (electric power plants, gas and liquid fuel pipelines, telecommunication systems, water supply and waste water treatment facilities). Firat Ba [11] categorizes the lifeline facilities in two groups as “easy to restore” and “difficult to restore”. “Easy to restore lifeline facilities” are those that generally can be returned to service in seven days or less or for which there are alternative resources or routes available; and “difficult to restore lifeline facilities” are those that will take more than seven days to recover operation or for which there are no alternative resources available. Lists of lifeline facilities under "easy to restore in seven days or less [11] are:

1. Transportation infrastructure
2. Emergency shelters
3. Fuel supplies
4. Radio and telephone centers
5. Municipal services facilities
6. Fiber optic/phone trunk lines
7. Water lines
8. Emergency response services
9. Evacuation routes

Lists of lifeline facilities under difficult to restore in seven days or less [11] are:

1. Potable water treatment facilities
2. Wastewater treatment facilities
3. Power generation facilities
4. Communication facilities
5. Fire and police
6. Medical facilities

7. Railroads
8. Levies/flood control dams
9. Power transmission lines

The losses and damages that may be incurred by a potential dam failure flood are [21]:

Economic losses

1. Direct physical property damage: residential structures, industrial buildings (Factory building), commercial and public buildings, railroads, main highways, bridges on river, agricultural land, agricultural villages, irrigation canals and their structures...etc.
2. Disruption of utilities (i.e. electric, sewer, municipal and agricultural water supply)
3. Replacement water supply
4. Costs of alternative transportation or routings
5. Cleanup costs
6. Repair costs
7. Replacement costs
8. Commercial: Loss of commercial navigation
9. Recreation: economic loss due to lost recreation or damage to recreational facilities upstream and downstream of the dam
10. Livestock such as camel, cattle, goats, sheep...etc.

Environmental damages

1. Habitat and wetlands
2. Toxic and radiological waste
3. Mine waste
4. Animal waste

Other damages

1. National security issues (dams upstream of military facilities)
2. Archeological and historic sites
3. Facilities not easily evacuated (assisted living establishments, prisons, hospitals)

2.4 Preemptive Measures

2.4.1 Inundation Mapping

One of the preemptive measures is to prepare potential inundation map and to prepare a detailed emergency action plan before the occurrence of an actual dam failure. The inundation map provides a description of the areal extent of flooding which would be produced by the dam break. The inundation map will help in identification of areas under risk, the location where population is concentrated, locations that are hard to evacuate or where there are not any high grounds to seek shelter etc. It also identifies zones of high velocity flow and depicts inundation for representative cross-sections of the channel. This information is standard outputs from HEC-RAS program flood routing model and inundation map may be developed utilizing cross-section and flood height data in conjunction with Digital Elevation Model (DEM) or Digital Terrain Model [15].

Inundation mapping can be accomplished in the GIS using HEC-GeoRAS. GIS information is exported from HEC-RAS and read into the GIS with GeoRAS. The geo-referenced cross sections are imported and water surface elevations attached to the cross sections are used to create a continuous water surface. The water surface is then compared with the terrain model and the floodplain is identified where the water surface is higher than the terrain. The HEC-GeoRAS has an ability to produce inundation maps for flood extent and depth [2]. In general, suggested issues during preparation of inundation map are listed below [27].

Preparation of map: to evaluate the effects of dam failure, map should be prepared delineating the area, which would be inundated in the event of failure. Land uses and significant development or improvements within the area of inundation should be indicated. The maps should be equivalent to or more detailed than the USGS quadrangle maps, 7.5-min series, or of sufficient scale and detail to identify clearly the area that should be evacuated if there is evident danger of failure of the dam. Copies of the maps should be distributed to local government officials for use in the development of an evacuation plan. The intent of the maps is to develop evacuation procedures in case of collapse of the dam, so the travel time of the flood wave should be indicated on every significant habitation area along the river channel.

Evaluation of hazard potential: to assist in the evaluation of hazard potential, areas delineated on inundation maps should be classified in accordance with the degree of occupancy and hazard potential. The potential for loss of life is affected by many factors, including but not limited to the capacity and number of exit roads to if higher ground and available transportation. Hazard potential is greatest in urban areas. The evaluation of hazard potential should be conservative because the extent of inundation is usually difficult to delineate precisely.

Industries and utilities: many industries and utilities requiring substantial quantities of water are located on or near rivers or streams. Flooding of these areas and industries, in addition to causing the potential for loss of; life, can damage machinery, manufactured products, raw materials and materials in process of manufacture, plus interrupt essential community services.

Least hazard potential: rural areas usually have the least hazard potential. However, the potential for loss of life exists, and damage to large areas of intensely cultivated agricultural land can cause high economic loss.

2.4.2 Emergency Action Plan

As a pre-event measure, there is further need for preparation of comprehensive evacuation (emergency action) plan. Suggested issues during preparation of EAP are given below [27].

Evacuation plans should be prepared and implemented by the local jurisdiction controlling inundation areas. The assistance of local civil defense personnel should be requested in preparation of the evacuation plan. State and local law enforcement agencies usually will be responsible for the execution of much of the plan and should be represented in the planning effort. State and local laws and ordinances may require that other state, county and local government agencies have a role in the preparation, review, approval, or execution of the plan. Before finalization, a copy of the plan should be furnished to the dam agency or owner for information and comment.

Evacuation plans will vary in complexity in accordance with the type and degree of occupancy in the potentially affected area. The plans may include delineation of the area to be evacuated; routes to be used; traffic control measures; shelter; methods of providing emergency transportation; special procedures for the evacuation of people from institutions such as hospitals, schools, and prisons; procedures for securing the

perimeter and for interior security of the area procedures for the lifting of the evacuation order and reentry to the area; and details indicating which organizations are responsible for specific functions and for furnishing the materials, equipment, and personnel resources required.

2.5 Dam Break Analysis Tools

Guideline for dam breach analysis [6] suggests the following four critical approaches as to accomplish dam break analysis study:

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.

These procedures can be further categorized in to two main tasks [25, 38] as:

1. The prediction of the reservoir outflow hydrograph and
2. The routing of that hydrograph through the downstream valley to determine dam failure consequences (i.e. inundated regions).

There are two basic perspectives that could be viewed by the modeler [19]:

The first perspective is to answer the basic question that whether or not, a given dam will fail. This basically refers to the strength parameter of materials of dam section and this perspective basically deals with simulation of breaching process of dam section. Various parameters relating to dam failure like breach characteristics, time of formation etc. are estimated by this approach.

The second perspective is to assume a given dam failure and study the resulting consequences in the downstream area. This includes assuming breach geometry based on the guidelines provided by various research agencies. This approach helps in preparation of emergency action planning for a possible failure of dam. The final basic product of such analysis is inundation details of downstream area owing to the flood generated due to the failure of dam.

2.5.1 Breach Characteristics Prediction Tools

Breach characteristics can be estimated in several ways. However, there are three viable methods that are used to estimate breach characteristics [7]:

Comparative Analysis: -perhaps the simplest approach to dam breach characteristics estimation is comparative analysis. This method compares a given dam of interest with those in a database of well documented dam failure case histories. In this method, 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 characteristics reported from the failure case histories of similarly configured dams are then directly applied to the dam being analyzed. Due to lack of failed dam record of similar size and type in Ethiopia, this method is not selected for case study dam break analysis.

Empirical equations: -these are relations developed from historical dam failures that correlate breach size and development time with dam and reservoir characteristics. The empirical approaches rely on statistical analysis of data obtained from documented failures. In this work, regression/empirical equations are used to estimate breach characteristics for use with HEC-RAS model and most widely used empirical equations will be given under section-3.7.

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

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

elevation, dam dimensions, soil properties of the dam, and tail water effects downstream. It is relatively simple to run and is widely used within the United States. Unfortunately, BREACH is no longer supported by the National Weather Service and significant advances in the understanding of the complex mechanics of a dam failure have not been incorporated. Also, the model has only been calibrated with a very limited number of cases [6].

It is deduced that, physically based method has clear limitations. Furthermore, this method suffer from extreme data demands, (e.g. area versus elevation, dam dimensions, soil properties of the dam, tail water effects downstream...etc.). Due to stated limitations and high demand for input data, this method mostly is not practically feasible for limited time and finance. Hence, this method is not selected in this work.

2.5.2 Parametric Models

Hydrologic and hydraulic models are the two widely used parametric (numerical) models used to predict dam breach hydrographs based on breach parameters provided by the modeler.

Hydrologic models: commonly used hydrologic models for dam break analysis are: HEC-HMS and HEC-1. In the absence of significant backwater effects, the hydrologic routing models offer the advantages of simplicity, ease of use and computational efficiency [6]. However, during downstream flood routing, backwater effect is expected for realistic flood routing. Furthermore, the hydrologic models do not provide useful information on hydraulic conditions (i.e. water surface elevations, flow velocities). These and other critical requirements limit use of hydrologic model for dam break analysis. Hence, due to these limitations, hydrologic model is not selected by the Author.

Hydraulic models: hydraulic models are broadly classified into steady flow and unsteady flow models. In unsteady flow, time dependent changes in flow rate are analyzed explicitly as a variable, while steady flow analysis models neglect time all together [6].

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.

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. The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Unsteady flow analysis can be further classified in to one-dimensional and two-dimensional based on information on the lateral distribution of flow across a cross section [19].

Two-dimensional hydraulic models are based on integration over the flow depth to obtain depth averaged velocity values and are solved using an appropriate numerical approach such as a Finite Element Model (FEM). The routing is done in longitudinal as well as lateral directions. Some of the two dimensional models are MIKE FLOOD (developed by DHI Group, Denmark), BASEMENT, CCHE2D-FLOOD, FLO-2D and FESWMS model.

One-dimensional hydraulic models on the other hand treat the flow through the channel and the flood plains only in longitudinal direction. Hence the equations are solved from one cross-section to another in one dimension. These equations for modeling the one-dimensional flow are derived from the conservation of mass and conservation of momentum equations between adjacent cross-sections. Some of the one-dimensional routing models include HEC-RAS, Full Equations (FEQ) model, FLDWAV (developed by National Weather Service), ISIS Flow (Developed by Wallingford Software)...etc. However, readily available hydraulic models that can be used for performing unsteady dam breach outflow hydrograph computation and downstream flood routing are, HEC-RAS, NWS DAMBRK, NWS-FLDWAV and a few others [7]. The three models integrate shallow water flow equations (SWE) during downstream unsteady flood routing and the choice to use these models mainly depends on required detail data, the required degree of accuracy (stability of the model) and researcher's expertise on particular model.

The HEC-RAS model is the most widely used one-dimensional 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 (since version 3.0) have a parametric dam breach routine that can calculate a breach outflow hydrograph within an unsteady flow simulation. However, the recent version of DAMBRK so called BOSS DAMBRK was produced based upon the same basic unsteady routing hydraulic

principles as HEC-RAS. But, BOSS DAMBRK was specifically developed for modeling dam failures. The cross-section input requirements for routing dam break floods by BOSS DAMBRK requires the same number of points to represent every cross section, which limits its usefulness.

A detailed reviews of selected method as to achieve objectives of this research is given in the subsequent section.

2.6 Review of HEC-RAS Model

The HEC-RAS system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines [29].

The HEC-RAS system is comprised of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphing and reporting facilities. HEC-RAS is able to take into consideration hydraulic effects of bridges, culverts, weirs, and other structures in the river and floodplain on water surface calculations [28]. The HEC-RAS modeling system was developed as part of the Hydrologic Engineering Centers Next Generation software and replaces several existing Corps of Engineers programs, including the HEC-2 water surface profile program [1].

The dam break tool in HEC-RAS can simulate the breach of an inline structure such as dam, or a lateral structure such as a levee. The latest versions of the HEC-RAS model include algorithms to model both overtopping and piping breaches [29]. HEC-RAS uses hydraulic principles through cross-sections upstream and downstream of the dam to define how the reservoir drains during the formation of a dam breach. The dam crest is modeled as an inline weir and either a piping failure or overtopping failure is simulated with enlargement of the breach occurring over time as defined by a specified breach progression. Flow through the piping hole is calculated as orifice flow and flow through the breach is calculated as weir flow. The water surface profile upstream of the dam is back-calculated using unsteady momentum and hydraulic principles for each time step and the resulting drawdown through the hole and/or breach produces an outflow hydrograph. Resulting water levels for each time step

downstream of the dam are used to model potential backwater effects and the weir and orifice coefficients are automatically adjusted for submergence, if necessary [6].

HEC-RAS can also model a piping failure that does not progress to the point of collapsing the crest. In this scenario, the piping hole is simulated as a sluice gate [29]. The simplest case that can be used for testing the coupling of breach process models with HEC-RAS is that of a single reach bounded upstream by a dam with a reservoir described by a storage-elevation function, and the next refinement that can be simulated is the same system with dynamic routing being used for flow in the reservoir [7].

The HEC-RAS computational program has the ability to model extreme flow dynamics in the downstream of the reach due to a dam break flood waves and produce water surface profiles over the length of the modeled area [20]. 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 [6]. Moreover, the HEC-RAS outputs like water surface profiles can easily be converted to flood inundation maps by its companion program called HEC-GeoRAS which is an Arc-GIS extension [9].

Because of complexity of solving routines, the output solutions were thoroughly reviewed for stability and correctness. Furthermore, many studies indicate that the solution found by HEC-RAS is stable and trustworthy [2]. Hence, due to its extensive capabilities and availability in USACE website; its' outputs compatibility with Arc-GIS (HEC-GeoRAS) software packages and for the other advantages stated above, HEC-RAS model is selected by the Author to simulate the Tendaho dam break floods.

2.6.1 Dam Break Simulation with HEC-RAS Model

A dam is modeled within HEC-RAS model by using the Inline structure editor. An inline structure is represented with a weir profile (that includes the spillway) and gates for normal low-flow operation. An example of an inline structure is given in the following figure.

The Inline structure editor allows the user to put in an embankment, define overflow spillways and weirs, and gated openings (radial or sluice gate). The lake area upstream of the dam can either be modeled with cross sections or by using a storage area. If x-sections are used, then HEC-RAS will perform full unsteady flow routing through the reservoir pool and downstream of the dam. If a storage area is used, HEC-

RAS uses level pool routing through the lake, then unsteady flow routing downstream of the dam. When using a storage area to represent the reservoir pool, HEC-RAS requires two x-sections inside of the reservoir pool, then the Inline structure representing the dam, and then downstream x-sections.

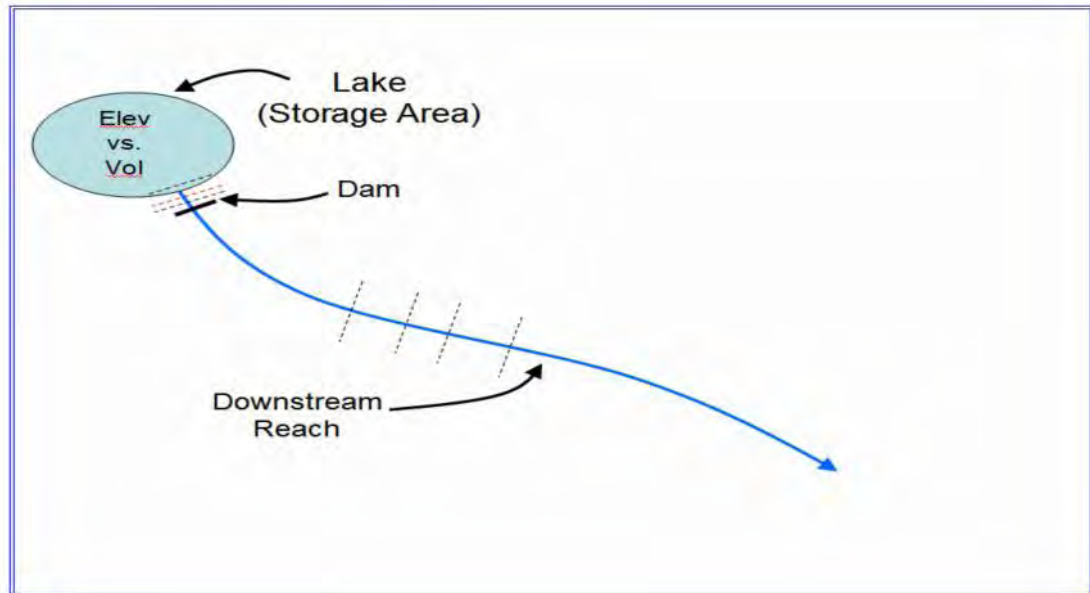


Figure 2-2 HEC-RAS dam break module concept

Dam break simulations are performed with HEC-RAS for dam safety studies as well as for flood damage analyses for situations that involve possible dam breaches [7]. These studies usually need to address the following situations and their associated uncertainties.

Dam failure scenarios: two scenarios were created using HEC-RAS, the first scenario is the Probable Maximum Flood (PMF) dam breach and the second is the Sunny day dam breach. These two scenarios were usually utilized by different researchers for earthen dam breach analysis using HEC-RAS model [38].

PMF scenario: this scenario relates to a dam's inability to cope with an extreme rain event. It is also called overtopping/rainy season failure scenario. Presumably, very heavy rainfall which causes a build-up of water capable of overwhelming a dam is not likely to be confined solely to the catchment above the dam and entirely held back until the moment of failure.

Rather, a dam-failure flood is likely to be accompanied, before and after failure actually occurs, by flooding downstream and in adjacent, possibly tributary, catchments. In such a situation it would be expected that the emergency services

would already be active by the time the failure took place and that some people would already have been evacuated from low-lying areas. Equally, though, evacuation routes may have already been lost. The cutting of such routes is predictable in the case of rain-related dam failure and must be considered as part of the planning process. Graphical presentation of dam failure during a flood season is shown in the following figure [10].

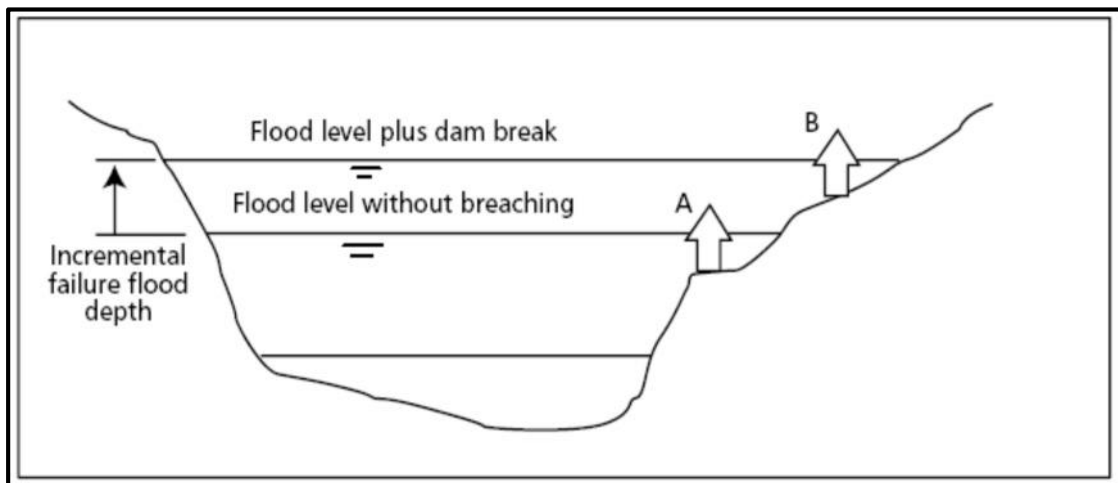


Figure 2-3 Graphical presentation of dam failure during PMF

Sunny day scenario: this may occur in many guises, have rather different features from an emergency management point of view and also called piping/dry season failure scenario. Worldwide, many dams have collapsed through internal erosion (piping), but failure may also result from "impact" events such as massive landslides, earthquakes or even terrorist activity.

Enough has been said to suggest that from a disaster management standpoint there can be substantial differences between different dam-break floods and that these differences are likely to be highly significant in terms of response characteristics. How, then, should planners proceed? The answer must be that they should identify the most credible failure modes but that in planning for them must develop arrangements which are sufficiently flexible to encompass the possibility of less likely kinds of failure.

Most recorded cases of embankment failure have resulted from internal erosion, frequently as a new dam has filled for the first time. This is because of the quality of dam construction and of surveillance [4]. In general, graphical presentation of sunny day dam failure is given below [10].

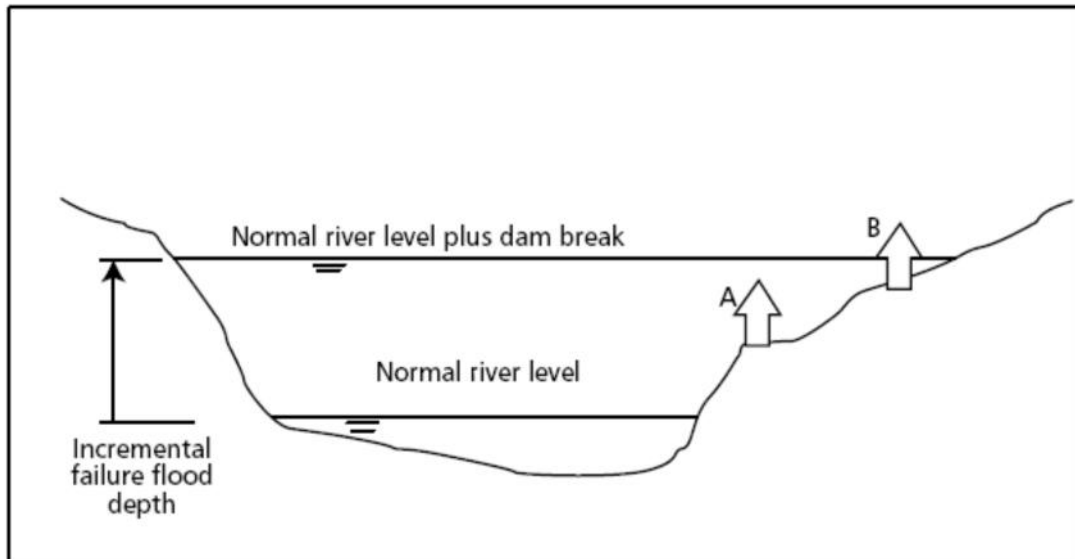


Figure 2-4 Graphical presentation of sunny day dam failure

In this work, both the PMF that is rainy season (winter case) and Sunny day (dry/summer case) scenarios were considered for a series of spillway gate opening defined by the Author.

Failure mode: overtopping or piping failure. The HEC-RAS breach computations include both overtopping and piping failure modes. Any other failure mode can be approximated with one of those two methods. The failure mode is the mechanism by which the breach occurs. Overtopping failures start at the top of the dam and grow to maximum extents; while a piping failure can start at any elevation/location and grow to the maximum extents. The ultimate breach size and breach time are much more critical in the estimation of the outflow hydrograph than the failure mode.

Trigger condition (failure initiation): reservoir elevation, reservoir elevation + duration, or selected time.

Breach description input data: these data include maximum breach bottom width, ultimate breach bottom elevation, breach development time, failure mode and initiation specification, breach growth progression (i.e. linear or nonlinear) and breach side slopes

2.6.2 Data Requirements for Dam Breach Simulation with HEC-RAS

The HEC-RAS simulation model uses the following parameters to describe a dam breach:

Failure location: transverse location of the centerline of the breach in the dam. This is selected based on many factors (type and shape of dam, failure type and mode). In general, one should consider all factors about the dam, including any historical knowledge of seepage and foundation problems, and place the breach location in the most probable location for each failure type. Commonly, the center of the breach is set to the centerline of the downstream main channel.

Failure mode: overtopping or piping. In general, the following data are required for overtopping and piping modes of failures [12].

For overtopping mode of failure

- ✚ Average width of the ultimate breach configuration
- ✚ Top and bottom elevations of the breach configuration
- ✚ Breach side slopes
- ✚ Time of breach formation

For piping mode of failure

- ✚ Average breach width of the ultimate breach configuration
- ✚ Top elevation of the ultimate breach configuration
- ✚ Breach side slopes
- ✚ Centerline elevation of initial piping location
- ✚ Time of breach formation

Shape and progression: the description of the breach shape in HEC-RAS consists of the height of the breach, breach width, and side slopes. These values represent the maximum breach size. The breach width is described as the average breach width (b) in several of the empirical equations. HEC-RAS uses the breach bottom width for input. The breach height (h_b) is the vertical extent from the top of the dam to the invert elevation of the breach. Many publications and equations also use the height of the water (h_w), which is the vertical extent from the maximum water surface to the invert elevation of the breach. A trapezoidal shape (geometry) has been widely used by several researchers as shown in the following figure [19].

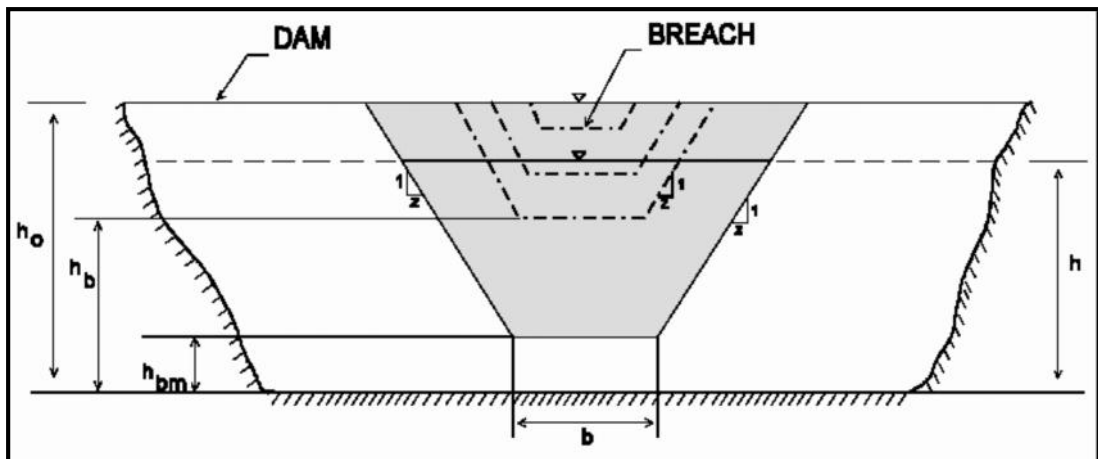


Figure 2-5 Example of breach geometry

The HEC-RAS model assumes that the breach bottom width starts at a point at the dam and enlarges downward at a linear or non-linear rate over the failure time (T_f) until the terminal bottom width (b) is attained and the breach bottom has eroded to the terminal elevation h_{bm} . If (T_f) is less than one minute, the width of the breach bottom starts at a value of “ b ” rather than zero; this represents more of a sudden collapse failure than an erosion failure. The bottom elevation of the breach (h_b) is simulated as a function of time (T_f).

A breach progression must be specified for HEC-RAS piping failure analyses. The breach progression defines the rate at which the piping hole or breach width enlarges with time. Both of these models allow the user to choose a linear progression, sine wave progression (starts out slowly, with a more rapid increase in the center of the progression and then ending more slowly), or a user defined (manual) progression [6, 29]. Like with piping failures, a breach progression must be specified for HEC-RAS analyses of overtopping failures. Overtopping failures do not have the transition from orifice to weir flow with a crest collapse that a piping failure requires. The breach progression is therefore not as variable and a more uniform progression rate is applicable. In an overtopping failure, the failure time is assumed to start when the upstream slope starts eroding. This may not happen until a significant time period after the dam starts to overtop. Both the linear and sine wave progressions are appropriate in this situation.

Formation/development time: 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. HEC-RAS requires that a breach formation time be provided. This

time represents the “duration from when the breach begins to have some significant erosion, to the full development of the breach”. The estimation of the breach formation time is done outside of the HEC-RAS software. It can be estimated from HEC-RAS user’s manual [29], empirical methods or from the results of process model applications. The use of process models should significantly decrease the uncertainty in the estimation of this parameter as a process model can be tailored to simulate a specific structure rather than using an estimate for a class of structures.

Trigger condition: reservoir elevation, reservoir elevation + duration, or selected time.

Weir and pipe flows coefficients: weir coefficient is used to compute overtopping (weir) flow and an orifice coefficient is used to compute piping (pressure) flow. The orifice coefficient (C_p) defines flow through the piping hole before the models shift to weir flow as [6]:

$$Q = C_p * A * \sqrt{2gH_p} \dots \dots \dots (2.1)$$

Where, H_p = piping head (difference between the water level, centroid elevation of the piping hole), C_p = piping orifice coefficient and A = cross sectional area of the piping hole.

Danny Fread, the Author of BREACH program, outlines a method of computing the orifice coefficient, C_p , which is dependent on the material of the dam and length and size of the piping hole. This equation is based upon the Darcy friction factor (f) where:

$$f = 0.015 * \left(\frac{D_{50}}{D}\right)^{0.167} \dots \dots \dots (2.2)$$

Where, D_{50} = average grain size in millimeters for the dam embankment material, D = piping hole width in feet (assumed to be square).

$$C_p = \sqrt{\frac{1}{1 + fL/D}} \dots \dots \dots (2.3)$$

Where, L = length of the piping hole (in feet) along its centerline.

Note: C_p is always less than or equal to 1. C_p is also inversely proportional to the grain size, being larger for smaller D_{50} values.

Weir Coefficients (C_w): after the dam crest collapses, the breach section erodes laterally and outflow through the breach is modeled as weir flow. During a piping mode, the configuration of the weir consists of a flat approach channel through the breach with a vertical drop-off to the tail water section resulting from head-cutting during both the piping mode and weir flow. The maximum allowable weir coefficient for a broad-crested weir for all piping failure mode is 3.087.

During the overtopping phase before failure, the crest acts like a broad crested weir with side slopes. Extrapolating from King and Brater, C_w is usually around 2.6 to 2.8 for a typical dam, but can vary from 2.6 to 3.087 for a broad crested weir. However, the recent version of HEC-RAS software (HEC-RAS version 4.1) includes the capability to model the dam crest weir flow and breach weir flow with different C_w values, the crest profile length will not need to be adjusted. The following figure summarizes the required parameters and encoding for HEC-RAS model [8].

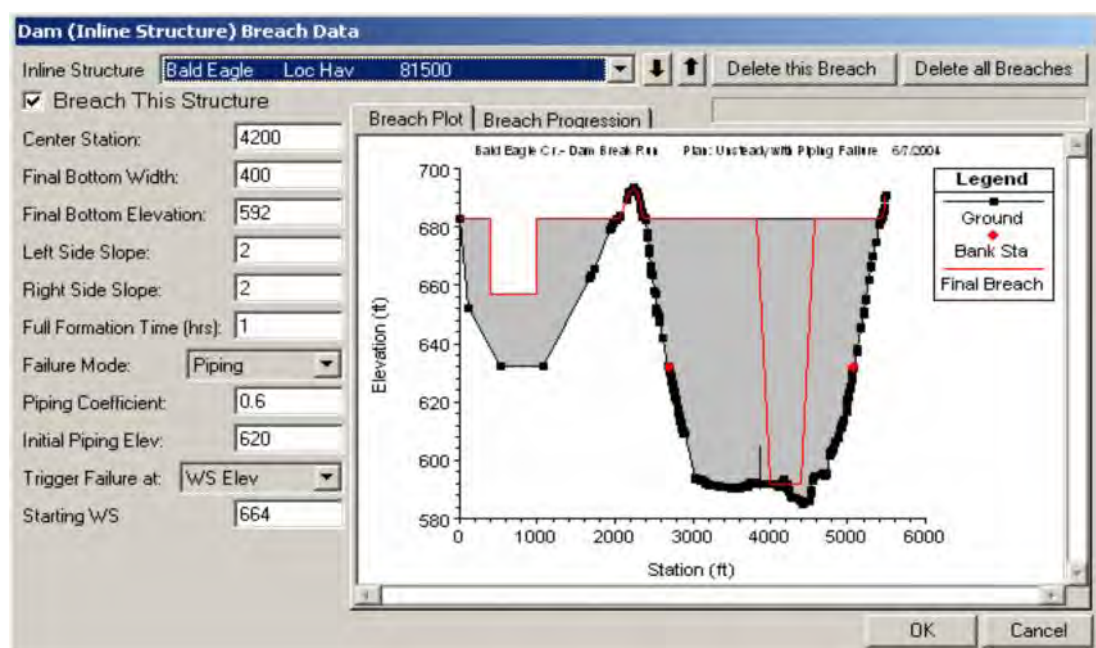


Figure 2-6 Example of dam (inline structure) breach data editing

In conclusions, the breach dimensions, as well as the breach formation time, must be estimated outside of HEC-RAS model. Many case studies have been performed on data from historic dam failures leading to guidelines, regression equations, and computer modeling methodologies for estimation of the dam breach size and formation time.

CHAPTER THREE

3. METHODOLOGY

In this work, the overtopping and piping failure of the Tendaho dam caused by a large storm hydrograph will be simulated under eleven different scenarios using the HEC-RAS software developed by USACE. The study area description, modeling tools, modeling domain, input data into the model, and prediction equations will be discussed in detail in the following sections.

3.1 Study Area Description

3.1.1 Location of Tendaho Dam and Irrigation Project

The Tendaho dam and irrigation development project is found in the Lower Awash valley in the Afar Regional State in the North Eastern part of Ethiopia. Dam site is located at 600Km from Addis Ababa. The project area is situated between Latitude $11^{\circ}-30'$ to $11^{\circ}-50'$ North and Longitude $40^{\circ}-45'$ to $41^{\circ}-03'$ East as shown in the following figure [36]. Tendaho dam and reservoir are located in Zone-one of Mille Woreda whereas, Irrigation command is located between Tendaho dam and the delta region downstream of Assaita town, almost up to the outfall of the river Awash into Lake Abbe; that is in the same Zone-one, in Dubti, Afambo and Assaita Woredas.

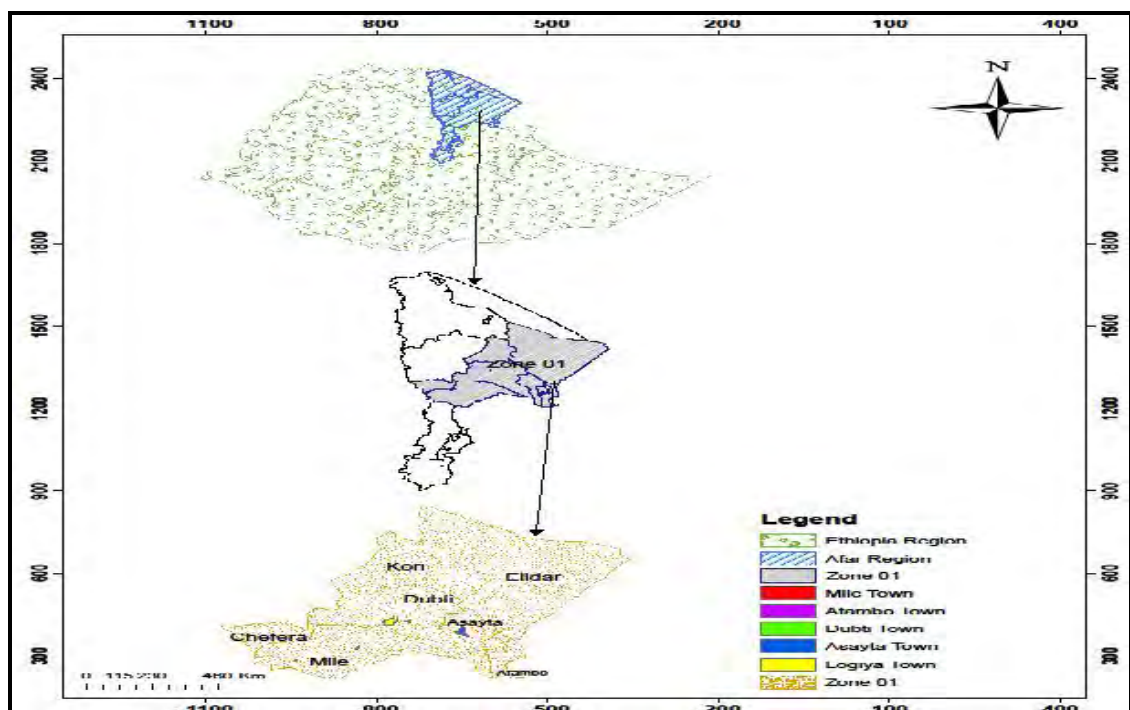


Figure 3-1 Location map of study area

3.1.2 Climate

The climate of the Tendaho dam and irrigation project area is generally arid, with temperatures varying from 17 to 43°C. The mean annual rainfall is 200 mm having two rainy seasons, March to April and July to August. The mean annual sunshine hour is 9.8 hours. The relative humidity varies from 31% in June to 96% in February. Wind speed is maximum in March reaching 187Km/day and minimum in September reaching 95.4km/day. The mean annual potential evapotranspiration is 2,078mm and mean annual reservoir evaporation is 2,899mm [36].

3.1.3 Topography

Elevation of the study area varies from 440m.a.m.s.l. at Tendaho to 340m.a.m.s.l. near Assaita. The entire irrigation command lies in deltaic alluvial plains with very mild slope varying from 0.05 to 0.10m per Km. The area is prone to flooding by river Awash, which has a tendency to change its course very often [35]. Study domain DEM is shown in the following figure, which is further used by the Author for reach and floodplain cross-sectional geometry generation and for downstream inundation mapping.

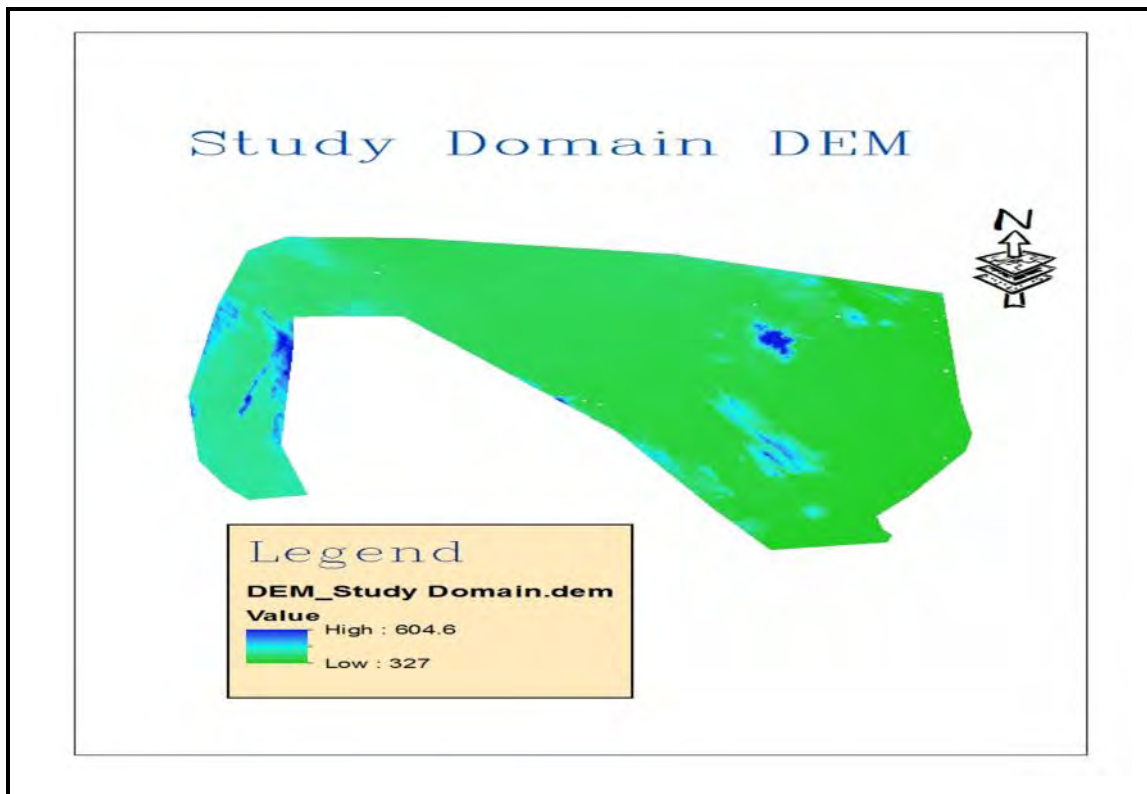


Figure 3-2 Study domain DEM

3.1.4 Land Use

Most of the downstream commands are under extensive cultivation for sugar cane plantation. As per WWDSE, the proposed 50,000 ha net irrigable land is to be irrigated in two phases. About 18,219 ha net land was already covered by sugar cane plantation in the first phase. The second phase sugar cane plantation would cover about 13,850 ha. This phase is under construction progress and the remaining lands are covered by forests and bushes and expected to be cultivated within phase two plan. There is a provision for pasture land in each border of irrigation land as most of the communities are pastoralists. About 7,200 hectares of land are urban lands (i.e. Logia, Dubti and Assaita towns) which are densely populated settlements within study domain.

3.2 Modeling Tools

In this work, Arc view GIS Version 10 extension program HEC-GeoRAS Version 10 tool was used for HEC-RAS model development, to generate modeling reach and floodplain cross-sectional geometric data and for downstream inundation mapping. Global Mapper Version 13 with Google Earth software are used for assessing and editing study area map and for remote sensing of the developed infrastructures like hydraulic structures, towns and current land use information. HEC-RAS Version 4.1 tool was used for subject dam break simulation and unsteady flood routing at downstream regions. Finally, Microsoft EXCEL 2007 was used to analyze HEC-RAS outputs.

3.3 Modeling Domain

The modeling domain for the purpose of this thesis study is composed of Tendaho dam and its reservoir at the upstream boundary, the downstream most cross-section of River Awash up to end of Assaita town and Tendaho Sugar Factory at the downstream boundary that is the reach of about 95.8 Km lying in between these two boundaries. The reach starts with Awash River, where the Tendaho dam is located, for some 6.5 Km at the upstream end and then confluences with the Logia River. River Awash then joins with the Logia River, which runs for some 89.3 Km up to end of Assaita town and Tendaho Sugar Factory to the East. Flow coming from Logia River was specified as lateral flow to the reach based on the compiled data as to monthly

observation of flow values on Logia River by MoWIE's observation station located just upstream of the confluence point.

Hence, the modeling reach is made up of a 6.5 Km portion of Awash River till the confluence followed by the 89.3 Km long River Awash till the end of Assaita town from the confluence, for a total of some 95.8 Km as given in the following figure. There are several Agricultural Villages, Sub-Heads, Head-Quarter and towns located in the vicinity of Awash. Logia town through which the both Rivers flow is therefore one of the most critical and populated settlement in the vicinity in terms of a potential flood risk.

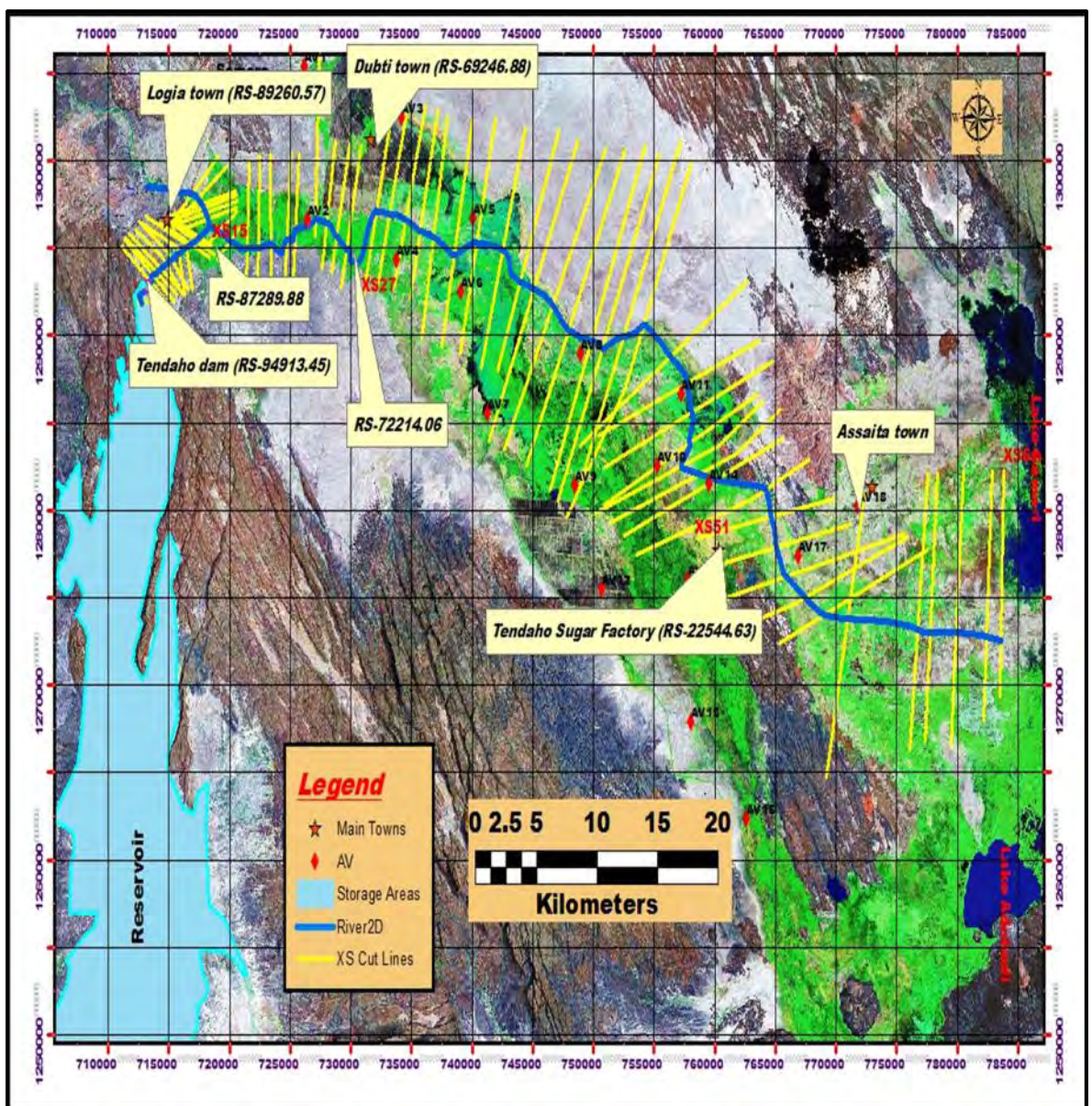


Figure 3-3 Study domain definition

3.4 Input Data

The primary activity of the research was getting relevant information and data of the study domain. The HEC-RAS model requires two main input data groups to simulate dam break failure and the resultant WSELs and discharge values spatial and temporal-wise. These data can be categorized in two groups as:

3.4.1 External Data

The term “external data” implies that this data has to be acquired from external sources and books so as to base it on official documents and measurements and that it lies outside the scope of judgment of the user of the HEC-RAS Model.

In case of modeling a dam break failure and the resultant flood, these data mainly include but not limited to: geometrical and technical data of the subject dam, Digital Elevation Model (DEM), layout map for modeling reaches and flood plains, hydrology of the Tendaho dam, rating curve of River Awash at Tendaho, monthly observed flow data for the River Logia, Manning’s coefficient and population at dam break flood risk data. The collection and justification of these data will be given under section-4.3

3.4.2 Internal Data

The term “internal data” implies that these are the data that either the operator has to base on his/her own judgment, on the Model’s Manual or on pertinent equations or preceding academic studies. These data mainly include but not limited to: breach parameters, Automatic Manning "n" calibration, initial conditions, failure initiation criteria, expansion/contraction coefficients, weir coefficients (C_w), lateral structure modeling, friction slope for x-sections, low flow filter option, breach progressions and modeling scenarios. The details and justification of the selection of values for these data will be discussed under section-4.4.

After collecting all the necessary external data, initial model development was performed using HEC-GeoRAS and using a HEC-RAS option in Arc view GIS data were imported. The data import had established the river/reach schematic and the description of cross-sections. Additional information, like hydraulic structures data, flow data, boundary conditions (i.e. upstream and downstream boundary conditions), initial conditions and breach parameters were established prior to model simulation.

Then, the HEC-RAS model will be set up to simulate the Tendaho dam breach which will provide peak flood outflows and maximum WSELs at each river cross-sections for each predefined scenarios. Finally, empirical equation developed by Graham (1999) will be used to predict number of fatalities at downstream region due to probable failure of subject dam.

3.5 Application of HEC-GeoRAS

3.5.1 Building River Hydraulics Model

HEC-GeoRAS is a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results [29]. The HEC-GeoRAS 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-GeoRAS 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. HEC-GeoRAS requires that the user have a DEM. The DEM must be projected into a coordinate system (PCS). The coordinate system of the DEM is used as the basis for developing each of the RAS Layers. The HEC-GeoRAS also requires that the stream centerline layer and cross-sectional cut line layer be created. The development of all other RAS Layers is optional based on the data needs for the river hydraulics model. A summary of RAS layers and their use in building a hydraulic model relevant to this study is provided in the following table.

The stream centerline layer is used to identify the connectivity of the river system. It is created in the downstream direction and is used to assign river stations to the cross sections, bridges to order computational nodes in the HEC-RAS model.

The cross-sectional cut lines layer is the principal data constructed using HEC-GeoRAS. Cut lines are digitized across the floodplain area to capture the profile of the land surface. Cross sections should be digitized perpendicular to the path of flow in the channel and overbank areas to be consistent with one-dimensional flow characteristics. Having created the bank lines and flow path centerlines prior to laying out cut line locations is advantageous. Once the RAS layers have been created, GeoRAS tools and menus are available to assign and populate attribute data. Lastly, the data are written out to the HEC-RAS geospatial data exchange format and are imported into HEC-RAS model.

Table 3-1 Summary of HEC-GeoRAS layers for HEC-RAS model development

RAS Layer	Description
Stream centerline	Used to identify the connectivity of the river network and assign river stations to computation points.
X-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
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.

3.6 Application of HEC-RAS

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a simulation software developed by the US Army Corps of Engineers and has been developed to manage rivers and other public works under their jurisdiction. The HEC-RAS software has found wide acceptance among hydraulic engineers and researchers due to its robust channel flow analysis capabilities and its ability to simulate unsteady flood wave propagation and identifies flood prone areas as the areas where

the ground is lower than the computed water elevation and allows the user to visualize the flood propagation in real time, thus making the software ideal for dam breach modeling. The HEC-RAS model was previously reviewed in detail under section-2.6: review of HEC-RAS model.

3.6.1 HEC-RAS Solution Techniques for Unsteady State Flow Equations

Despite the three-dimensional structure of the detailed free surface flows, it is possible to simplify the governing equations of unsteady open channel flow in cases in which the longitudinal flow scale is greater than the transversal dimensions and the problem is approximated as variable only in the spatial direction along the main flow (one-dimensional flow model). This is a reasonable approach in many practical applications assuming that water is incompressible, pressures are hydrostatic, vertical accelerations are negligible, and waves are non-dispersive. However, the numerical simulation of unsteady flow in rivers is a complicated task and the difficulties grow as the pretensions to obtain better quality or more general solutions [10].

The hydrographs from each dam break scenarios will tend to converge as they are routed downstream. Therefore, the proximity of the location of interest will govern the set of breach parameters adopted for the dam failure model. If the location of interest is in close proximity to the dam, the breach width and formation time will be critical and a conservative estimate of the parameters is prudent. The importance of the breach parameters will become less significant as the location of interest moves farther downstream.

The comparison of modeling the reservoir as a series of cross-sections versus a storage-area results in very significant rise in stage near the dam; however, the effects are less pronounced farther downstream. Perhaps the most critical output from dam failures simulations is predicting the flood wave travel time to populated areas. Given flood warning, the loss of human life may be mitigated. Hydrograph output from HEC-RAS can be used to estimate the arrival time of the waters to flood levels.

The governing physical law equations (conservation of mass and momentum equations) derived by Barkau (1982) are bases for unsteady flow solution within the HEC-RAS Version 4.1 software package. However, these equations along the direction of the main flow are similar to Saint-Venant (Navier-Stokes) equations for full hydrodynamic shallow Water Equations [29]. Thus, the simplified and

approximated one-dimensional equation of motion (in the spatial direction along the main flow) is given below.

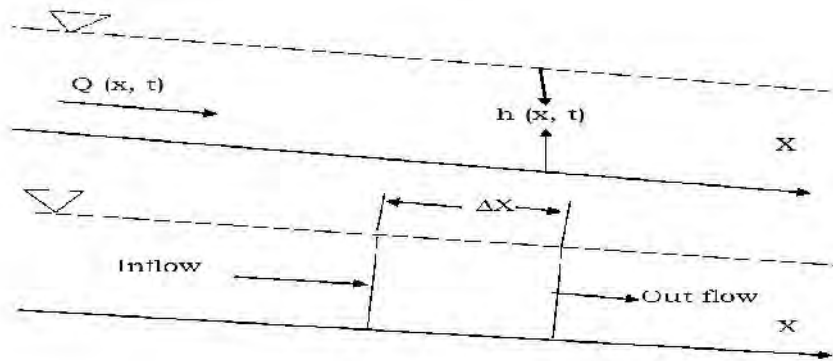


Figure 3-4 Elementary control for deriving of continuity and momentum equations

Continuity equation: continuity equation (conservation of mass) for a control volume states that the net rate of flow in to the volume is equal to the rate of change of storage inside the volume.

The rate of inflow to the control volume may be written as;

$$Q = \frac{\partial Q}{\partial x} * \frac{\Delta x}{2} \dots \dots \dots (3.1)$$

The rate of outflow as:

$$Q + \frac{\partial Q}{\partial x} * \frac{\Delta x}{2}$$

And the rate of change in storage as:

$$\frac{\partial AT}{\partial t} * x$$

Assuming that x is small, the change in mass in the control volume is equal to:

$$\rho * \frac{\partial AT}{\partial t} * x = \rho * [(Q - \frac{\partial Q}{\partial x} * \frac{\Delta x}{2}) - (Q + \frac{\partial Q}{\partial x} * \frac{\Delta x}{2} + Q_1)] \dots \dots \dots (3.2)$$

Where, Q_1 is the lateral flow entering the control volume and ρ is the fluid density. Simplifying and dividing through by ρx yields the final form of the continuity equation:

$$\frac{\partial AT}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \dots \dots \dots (3.3)$$

Where, q is the lateral inflow per unit length.

Momentum equation: conservation of momentum is expressed by Newton's Second law as:

$$\sum F_x = \frac{dM}{dt} \dots\dots\dots(3.4)$$

Conservation of momentum for a control volume states that the net rate of momentum entering the volume (momentum flux) plus sum of all external forces acting on the volume be equal to the rate of accumulation of momentum. This is a vector equation applied in the x-direction. The momentum flux (MV) is the fluid mass times the velocity vector in the direction of flow. Three forces will be considered: (1) pressure, (2) gravity, and (3) boundary drag/or friction force.

Pressure force: pressure distribution is assumed to be hydrostatic (pressure varies linearly with depth) and the total pressure force is the integral of the pressure-area product over the x-section. After all simplification, the net pressure force, F_{pn} due to control volume and forces exerted by banks in the x-direction is given as,

$$F_{pn} = -\rho g A \frac{\partial h}{\partial x} x \dots\dots\dots(3.5)$$

Where, h is flow depth,

Gravitational force: the force due to the gravity on the fluid in the control volume in x-direction is given as,

$$F_g = -\rho g A \frac{\partial Z_o}{\partial x} x \dots\dots\dots(3.6)$$

Where, Z_o is the invert elevation and the force will be positive for negative bed slopes

Boundary drags (friction force): after all simplifications friction forces between the channel and the fluid may be written as:

$$F_f = -\rho g A S_f x \dots\dots\dots(3.7)$$

Where, S_f is the friction slope, which is positive for flow in the positive x-direction. The friction slope is related to flow and stage by Manning's and Chezy friction equations and given below (i.e. used in HEC-RAS model):

$$S_f = \frac{Q/Q/n^2}{2.208R^{4/3}A^2} \dots\dots\dots(3.8)$$

Where, R is the hydraulic radius and “n” is the Manning’s friction coefficient.

Momentum flux: is defined as the net rate of the momentum. The net rate of momentum (momentum flux) entering the volume plus the sum of all external forces acting on the volume is equal to the rate of accumulation of momentum. Hence, after all mathematical simplifications and valid assumptions, the final form of the momentum equation (conservation of momentum) is given as:

$$\frac{\partial Q}{\partial t} + \frac{\partial Qv}{\partial x} + gA \left(\frac{\partial z}{\partial x} + S_f \right) = 0 \dots\dots\dots(3.9)$$

Where, Z is elevation of water surface (WSEL)

When the river is rising water moves laterally away from the channel, inundating the flood plain and filling available storage areas. As the depth increases, the floodplain begins to convey water downstream generally along a shorter path than that of the main channel. When the river stage is falling, water moves toward the channel from the overbank supplementing the flow in the main channel. Because the primary direction of flow is oriented along the channel, the two-dimensional flow field can be often be accurately approximated by a one-dimensional representation. Off-channel ponding areas can be modeled with storage area that exchange water with the channel. Flow in the overbank can be approximated as flow through a separate channel.

This channel/floodplain problem has been addressed in many different ways. A common approach is to ignore overbank conveyance entirely, assuming that the overbank is used as only for storage (i.e. this assumption is suitable for large streams. Fread (1976) and Smith (1978) approached this problem by dividing the system into two separate channels and writing continuity and momentum equations for each channel. To simplify the problem they assumed a horizontal water surface at each x-section normal to the direction of flow; such that the exchange of momentum between the channel and the floodplain was negligible and that the discharge was distributed according to conveyance, i.e.

$$Q_c = Q \dots\dots\dots(3.10)$$

Where, Q_c = flow in channel, Q = total flow, $\Phi = K_c / (K_c + K_f)$, K_c = conveyance in the channel, and K_f = conveyance in the floodplain

With these assumptions, the one-dimensional equation of motion (i.e. which HEC-RAS model used for unsteady flow routing in the downstream region) can be combined into a single set (i.e. conservation of mass and momentum equations developed by Barkau, 1982 [29]):

$$\frac{\partial A}{\partial t} + \frac{\partial(\phi Q)}{\partial X_c} + \frac{\partial[(1-\phi)Q]}{\partial X_f} = 0 \dots\dots\dots(3.11)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(\phi^2 Q^2/A_c)}{\partial X_c} + \frac{\partial(1-\phi^2)Q^2/A_f}{\partial X_f} + gA_c \left[\frac{\partial z}{\partial X_c} + S_{fc} \right] + gA_f \left[\frac{\partial z}{\partial X_f} + S_{ff} \right] = 0 \dots\dots(3.12)$$

Where, Q is total flow, q is the lateral inflow per unit length, A is flow area, x is flow distance in X-direction, t is flow time, S_{fc} is the friction slope of channel, S_{ff} is the friction slope of floodplain and Z is elevation of water surface (WSEL).

3.7 Equations to Predict Dam Break 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. The breach width (b), breach height (h_b) and breach time (T_f) have a great influence on the forecast of the outflow and the flooded area downstream of the dam. For relatively small reservoirs, the dam break peak outflow usually occurs before the breach is completely developed, resulting from a consistent drop in reservoir levels during the formation of the rupture, while in large reservoirs the dam break peak outflow occurs as soon as the breach has extended to its maximum size [6].

Geometric description of a dam breach must be estimated to simulate the resultant flood wave and downstream consequences. Reasonable values for the breach size and development time are needed to make a reliable estimate of the outflow hydrographs and resulting downstream inundation, flood travel times, water velocities, etc. Parameters that describe the breach size and rate of development have large uncertainty [7]. This is because; each of the breach parameter estimates will yield a different outflow hydrograph from HEC RAS. However, once these hydrographs are routed downstream, they will tend to converge towards a common result. How close they get to each other will depend on the distance they are routed, the steepness of the stream, the roughness of the river and floodplain, and the amount of floodplain storage available for attenuating the hydrograph. If the populated areas below the dam are quite a distance away (say 20 miles or more), then the resulting hydrographs

from the various dam breaches may be very similar in magnitude by the time they reach the area of interest. However, if the areas of interest are close to the dam, then the resulting breach hydrographs could produce a significant range in results. In this situation, the selection of the breach parameters is even more crucial [29].

A number of empirical equations that estimate breach parameters have been evaluated for use with HEC-RAS. The emphasis of that work [1, 2, 7, 8, 9, 20, 22, 38] has been on interpreting and applying the results methods for modeling overtopping and piping failures with HEC-RAS. The following five regression equations have been used for several dam safety studies [6, 7, 10, 11, 19, 24].

3.7.1 USBR (1988)

According to USBR (1988), breach width (B) and failure time (Tf) are given as:

$$\text{Breach width, } B \text{ (m)} = 3 \cdot h_w \dots\dots\dots(3.13)$$

$$\text{Failure Time, } T_f \text{ (hr)} = 0.011 \cdot B \dots\dots\dots(3.14)$$

Where, h_w = height measured from the initial reservoir water level to the breach bottom elevation which is assumed to be the streambed elevation at the toe of the dam.

3.7.2 MacDonald and Langridge-Monopolis (1984)

They estimated the quantity of eroded embankment materials V_{er} (m^3) for earth and rock dams as:

$$V_{er} = 0.0261 \cdot (V_w \cdot h_w)^{0.769}; \text{ for earth-fill} \dots\dots\dots(3.15)$$

$$V_{er} = 0.00348 \cdot (V_w \cdot h_w)^{0.852}; \text{ for rock-fill} \dots\dots\dots(3.16)$$

$$T_f \text{ (hr)} = 0.0179 \cdot (V_{er})^{0.364} \dots\dots\dots(3.17)$$

Where, V_w = volume of water discharged through breach (m^3), and h_w = hydraulic depth of water at dam at failure above breach bottom (m)

3.7.3 Von Thun and Gillette (1990)

The equations proposed by Von Thun and Gillette (1990) for the average breach width and failure time are given by:

$$B \text{ (m)} = 2.5 \cdot h_w + C_b \dots\dots\dots(3.18)$$

$$T_f \text{ (hr)} = 0.015 \cdot h_w \text{ for highly erodible dam} \dots\dots\dots(3.19)$$

$$T_f (\text{hr}) = 0.0209 * h_w + 0.25, \text{ for erosion resistant dam} \dots \dots \dots (3.20)$$

Where, h_w (m) = the depth of water at the dam at the time of failure,

C_b is function of reservoir size and given in the table below.

Table 3-2 Values of C_b based on reservoir size

Size of reservoir (m ³)	C_b (m)
<1.23*10 ⁶	6.3
1.23*10 ⁶ -6.17*10 ⁶	18.3
6.17*10 ⁶ -1.23*10 ⁷	42.7
>1.23*10 ⁷	54.9

3.7.4 Froehlich (1995b)

Froehlich has defined the average breach width, side slope and the time of failure as follows,

$$B (\text{m}) = 0.1803 * K_o * V_w^{0.32} * h_b^{0.19} \dots \dots \dots (3.21)$$

$$T_f (\text{hr}) = 0.00254 * (V_w)^{0.53} * h_b^{-0.9} \dots \dots \dots (3.22)$$

Where, $K_o = 1.4$, for overtopping failure and $K_o = 1$, for piping failure, h_b (m) = height of breach, V_w (m³) = volume of breached water

Froehlich further suggested breach side slopes of 0.7:1 (horizontal: vertical) for piping and 1:1 for overtopping.

3.7.5 Pandya and Jitaji (2013)

Pandya and Jitaji [19] suggest possible breach parameters estimation techniques for earth-fill dam, concrete gravity and concrete arch dam's failure by assuming trapezoidal shape of a breach. Recommended breach parameter values by Pandya and Jitaji are given in the following table.

Table 3-3 Suggested breach parameters for earth-fill dams

Dam type	B (expressed as dam height)	Z (ZH:1V)	T_f (hr)
Earth-fill dam	Min: 0.4	Min: 0	Min: 0.1
	Max: 13	Max: 6	Max: 12
	Mean: 4	Mean: 1	Mean: 2

3.8 Equation to Predict Loss of Life

Graham [1999] suggests a simple procedure for estimating loss of life from a dam failure, based on 40 dam failure cases in US with observed fatalities of 50 or more. As given in the following table, the user first decides on the severity of the flood and then finds the appropriate warning time in the 2nd column and the understanding of the flood severity by those under risk in the 3rd column. The 4th column indicates the suggested multiplier and the multiplier range for the fatality rate which yields the number of fatalities when multiplied by the population of the subject settlement. This is of course a very rough estimate but certainly presents some indicators that can be used while preparing the emergency action plans.

Table 3-4 Suggested fatality rates for predicting loss of life

Flood Severity	Warning time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)			
			Suggested	Suggested Range		
High	No warning	Not Applicable	0.75	0.3 to 1.00		
	15 to 60	Vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.			
		Precise				
	More than 60	Vague				
		Precise				
	Medium	No warning			Not Applicable	0.15
15 to 60		Vague			0.04	0.01-0.08
		Precise	0.02	0.005-0.04		
More than 60		Vague	0.03	0.005-0.06		
		Precise	0.01	0.002-0.02		
Low		No warning	Not Applicable	0.01	0.0-0.02	
	15 to 60	Vague	0.007	0.0-0.015		
		Precise	0.002	0.0-0.004		
	More than 60	Vague	0.0003	0.0-0.0006		
		Precise	0.0002	0.0-0.0004		

Flood severity can be categorized in to three categories as low, medium and high [37].

Low severity: occurs when no buildings are washed off their foundations. Use the low severity category if most structures would be exposed to depths of less than 10 ft (3.3 m) or if **DV**, defined in the following equation is less than 50 ft²/s (4.6m²/s).

Medium severity: occurs when homes are destroyed but trees or mangled homes remain for people to seek refuge in or on. Use medium flood severity if most structures would be exposed to depths of more than 3.3 m or if **DV** is more than 4.6m²/s.

High severity: occurs when the flood sweeps the area clean and nothing remains.

High flood severity should be used only for locations flooded by the near instantaneous failure of a concrete dam, or an earth-fill dam that turns into "jello" and washes out in seconds rather than minutes or hours. In addition, the flooding caused by the dam failure should sweep the area clean and little or no evidence of the prior human habitation remains after the floodwater recedes. This type of flooding occurred below St. Francis Dam in California and Vajont dam in Italy. The flood severity will usually change to medium and then low as the flood water travels farther downstream.

$$DV = \frac{Q_{df} - Q_{2.33}}{W_{df}} \dots\dots\dots(3.23)$$

Where,

- ✚ **Q_{df}** is the peak discharge at a particular site caused by dam failure
- ✚ **Q_{2.33}** is the mean annual discharge at the same site. This discharge can be easily estimated and it is an indicator of the safe channel capacity
- ✚ **W_{df}** is the maximum width of flooding caused by dam failure at the same site.

“Warning time” can be categorized in to three categories as given below [37].

No warning: means that no warning is issued by the media or official sources in the particular area prior to the flood water arrival; only the possible sight or sound of the approaching flooding serves as a warning.

Some warning: means officials or the media begin warning in the particular area 15 to 60 minutes before floodwater arrival. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives.

Adequate warning: means officials or the media begin warning in the particular area more than 60 minutes before the floodwater arrives. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives. The warning time for a particular area downstream from a dam should be based on when a dam failure warning is initiated and the flood travel time. For instance, assume a dam with a campground immediately downstream and a town where flooding begins 4 hours after the initiation of dam failure. If a dam failure warning is initiated 1 hour after dam failure, the warning time at the campground is zero and the warning time at the town is 3 hours. The fatality rate in areas with medium severity flooding should drop below that recommended in above table as the warning time increases well beyond one hour. Repeated dam failure warnings, confirmed by visual images on television showing massive destruction in upstream areas, should provide convincing evidence to people that a truly dangerous situation exists and of their need to evacuate. This should result in higher evacuation rates in downstream areas and in a lowering of the fatality rate.

Regarding the “**flood severity understanding**” the warning is comprised of two elements [37]:

- 🚨 Alerting people to danger
- 🚨 Requesting that people at risk take some action

The flood severity understanding can further defined as [37]:

Vague understanding of flood severity: means that the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.

Precise understanding of flood severity: means that the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others.

Estimating loss of life from dam failure is an art as much as it is a science [37]. There may never be a procedure available that will provide precise and accurate estimates of the loss of life those results from failure. Thus, the increase in fatality coefficient does not produce a significant effect. This is certainly the case with the subject reach of this thesis work, that is to say, in different reaches and regions where there are settlements with high population close to a dam, the time of issuance of warning would certainly have considerable impact on the fatality figure

CHAPTER FOUR

4. DATA COLLECTION AND ANALYSIS

4.1 General

This section discusses the types and source of data required for the study and their analysis.

4.2 External Data Collection

In this work, the external data were made from the following sources.

- ✚ Water Works Design and Supervision Enterprise (WWDSE): geometrical and technical data of the subject dam, Digital Elevation Model (DEM), layout map for modeling reaches and flood plains and hydrology of the Tendaho dam have been collected.
- ✚ Ministry of Water, Irrigation and Electricity (MoWIE): rating curve of River Awash at Tendaho and monthly observed flow data for the River Logia have been collected
- ✚ Central Statistical Agency: projected population data for year 2015 for peoples at dam break flood risks (i.e. Logia town, Dubti town, Afambo town and Assaita town) have been collected.
- ✚ Sugar Corporation: population data of Tendaho Sugar Factory, Subheads, Agricultural Villages, Schools, Hospitals, Health Centers and Health posts have been collected.

4.3 External Data Analysis

4.3.1 Geometrical and Technical Data

From geometric and technical data of Tendaho dam and reservoir, it was found that the Tendaho dam is a zoned earth-fill dam of 44m high (above river bed) and 412 m long constructed across the River Awash to store $1.86 \times 10^9 \text{ m}^3$ of water at Full Retention Level (FRL) to supply water to 60,000ha of gross irrigation land found at downstream of the dam [36]. These technical data will be used for breach parameters prediction and for x-sectional geometric data development of the subject dam, spillway and reservoir within HEC-RAS model. Detailed technical specifications of

Tendaho dam's components are given at Appendix-B of this paper. Furthermore, geometry of the Tendaho dam is given in the following figure.

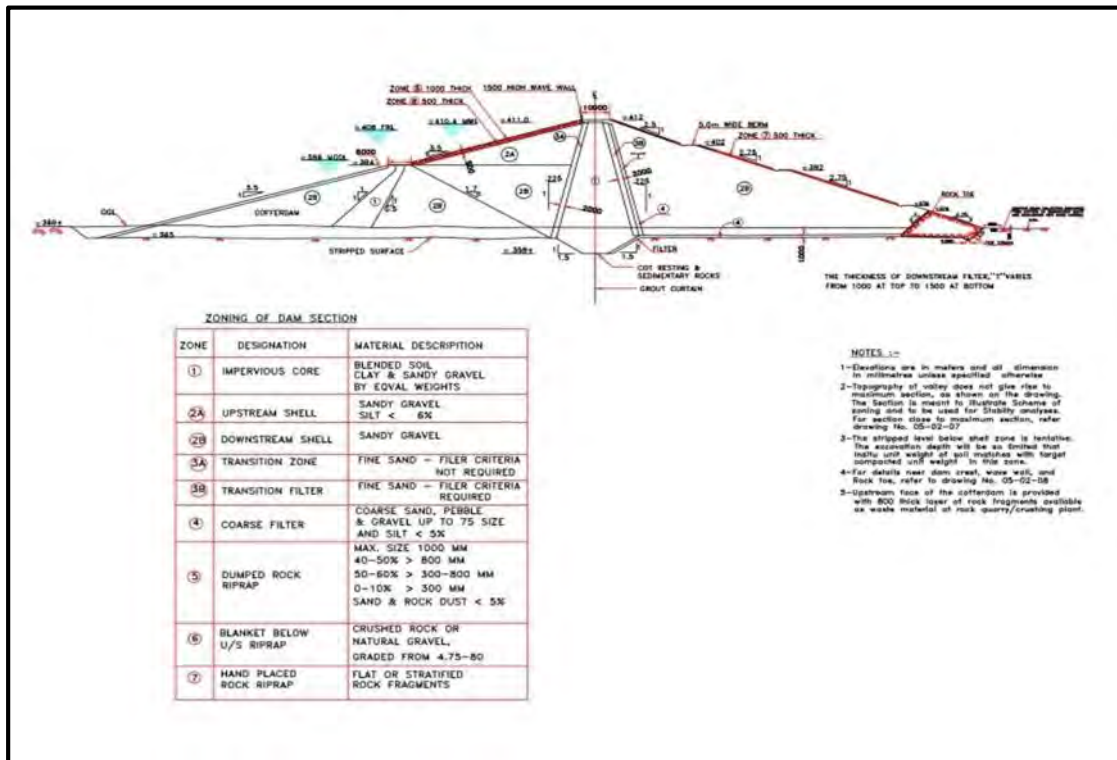


Figure 4-1 Tendaho dam section

4.3.2 Cross-Sectional Geometric Data Generation

Unfortunately, surveyed river cross-sectional geometric data at required location of Awash and Logia Rivers were not found at Ministry of Water, Irrigation and Electricity and at Water Works Design and Supervision Enterprise office as well. Interviews with WWDSE staffs (engineers) indicated that the cross-sections upstream of the Tendaho dam about 100m and downstream of the dam of about 100m were of their major concern for Tendaho dam design. Hence these x-sectional data have been collected and used as a reference data for x-sectional data generation from Digital Elevation Model (DEM) using HEC-GeoRAS software.

It is clear that, collected cross-sectional data for 200m reach length of River Awash at u/s and d/s of the dam were not adequate to model the whole reach of about 95.8 Km. Hence, to fill this data gap, an option to use generated x-sections from Digital Elevation Model (DEM) was made in this work. Hence, cross-section data of modeling reach and floodplain were generated by using Arc-GIS campaign software called HEC-GeoRAS. The following two basic topographic data sets were collected

and used as input data to HEC-GeoRAS model to generate reach and floodplain cross-section within study domain.

Secondary data: Tendaho dam and irrigation project layout map have been collected from Water Works Design and Supervision Enterprise (WWDSE). The map shows the location of Tendaho dam and reservoir, Awash and Logia Rivers, main towns (like Logia, Dubti and Assaita), Agricultural Villages (AV), Subheads, Headquarter, Tendaho Sugar Factory and other infrastructures which are relevant data for Tendaho dam break analysis and for dam break induced flood risk assessments. The following figure shows collected Tendaho dam and irrigation project layout. Furthermore, high resolution satellite images of the study area have been collected from WWDSE as to investigate detail information of the study domain as previously given in figure 4-1.

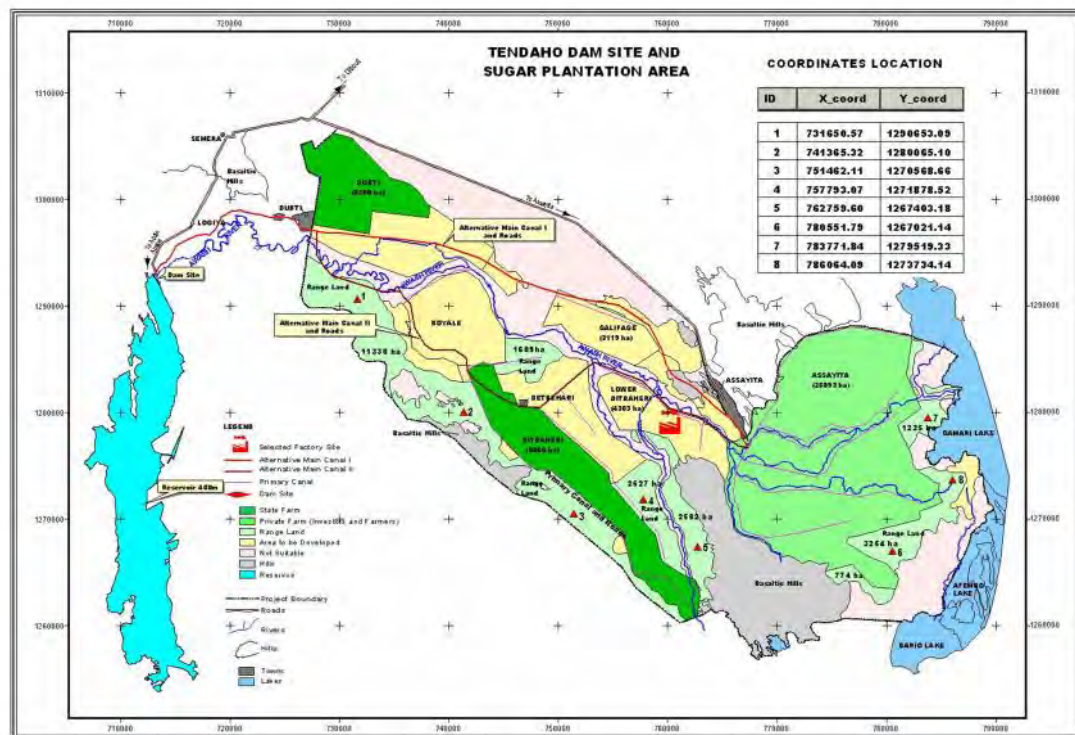


Figure 4-2 Layout map of Tendaho dam and irrigation project

Primary data: coordinate (X, Y) data at Control points (at corner points of irrigation command), hydraulic structures (i.e. dam, bridges), center of populated towns (i.e. Logia, Dubti and Assaita) and at Sugar Factory were taken by using hand GPS. In addition to hand GPS, coordinates (X, Y, and Z) data at said key locations have been further assessed using satellite remote sensing (Google earth software). Besides,

satellite image that shows background information is captured and stored in data set to visualize river networks during delineation and inundation mapping.

Table 4-1 Major infrastructures and settlements location

No	Infrastructure/settlement	Coordinates		
		X	Y	Z
1	Tendaho dam			
1.1	Dam Axis-1 (DA1)	713255	1292937	NA
1.2	Dam Axis-2 (DA2)	713435	1292657	NA
2	Spillway			
2.1	Spillway axis-1 (SA1)	712960	1292955	NA
2.2	Spillway axis-1 (SA2)	713004	1292947	NA
3	Logia town	715407	1296730	387
4	Awash-Logia Confluence	717926	1296285	386
5	Dubti bridge	732218	1297582	370
6	Dubti town	731206	1298346	370
7	Asaita town	769888	1280942	353
8	Sugar Factory-Assaita bridge	764851	1279646	353
9	Sugar Factory	757331	1278413	353

Before generating x-section data from DEM, Tendaho dam and irrigation command layout map which was previously in CAD format was geo-referenced (rectified) in a valid projected coordinate system (PCS) where Ethiopia and Tendaho dam and irrigation project are located by using Arc-GIS version 10 software. Steps of projection were: ADINDAN UTM Zone 37N. The satellite image of the study area is geo-referenced (digitized) based on rectified layout map of Tendaho dam and irrigation project. Then, GPS coordinates of control points (i.e. infrastructures and settlements listed above) are digitized on layout map.

Following digitizing (rectifying) relevant infrastructures and settlements on the map, channel and overbank geometries as well as hydraulic structures (i.e. dam, spillway

and bridges) geometries were generated from Digital Elevation Model (DEM of 30mx30m resolution) of Tendaho dam and irrigation project by using HEC-GeoRAS software. River reach and floodplain cross-sectional geometric data were generated in such a way that at selected location through a stream reach and at locations where changes occur in discharge, slope, shape, or roughness and at location of hydraulic structures. Considering flat slope and uniform flow of River Awash at downstream of Tendaho dam [36], a fewer number of x-section per Km were taken at key locations.

In this research, key locations are defined as places/points at which important infrastructures, settlements, towns (densely populated area), irrigation command area...etc. are located. These assumptions were valid assumptions and recommended by many researchers working in river hydraulics and dam safety especially for academic research which is data and finance limited research. Finally, relevant RAS Layers for stream centerline, cross-sectional cut line, bank line, flow path line and others are created and exported to HEC-RAS model.

Graphical description of study Domain River reaches, downstream floodplains, generated cross-sectional geometry of reaches and floodplains and relevant downstream settlements within study area are given in the following figure and table.

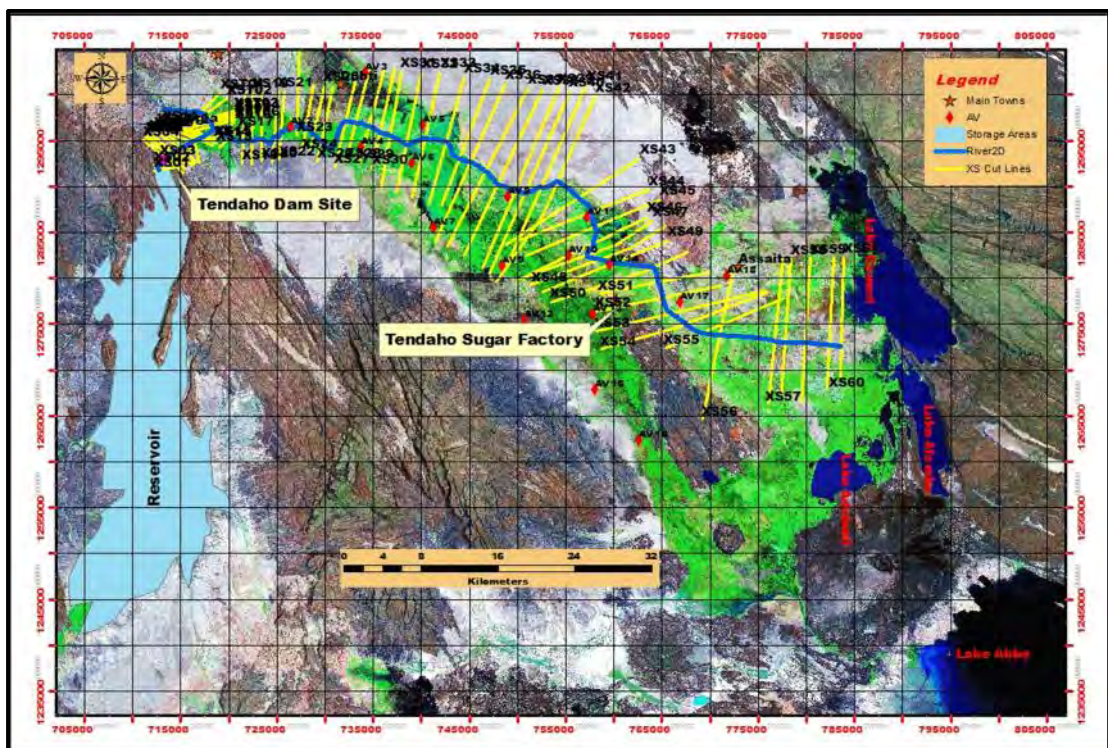


Figure 4-3 HEC-GeoRAS model setup for modeling domain

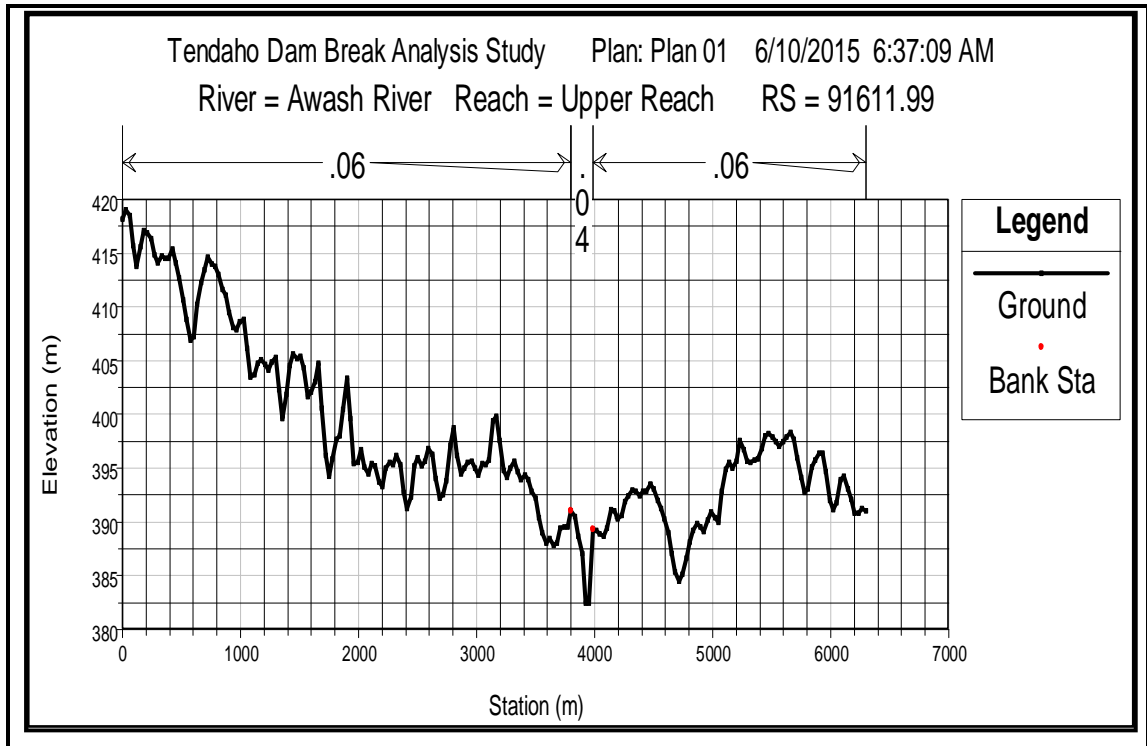


Figure 4-4 Sample generated river cross-section using HEC-GeoRAS

The calculated lengths of reaches are given in the following table.

Table 4-2 Generated data as to cross-sections for the modeling reach

Cross-section ID	River station from most d/s end of reach (m)	Cross-section ID	River station from most d/s end of reach (m)	Cross-section ID	River station from most d/s end of reach (m)
XS_01	95,756	XS_18	83,968.07	XS_41	40,608.00
XS_02	95,334.76	XS_19	82,813.79	XS_42	38,903.41
XS_03	94,887.95	XS_20	81,627.94	XS_43	36,180.73
XS_04	94,469.50	XS_21	79,808.52	XS_44	34,045.37
XS_05	94,025.55	XS_22	78,517.52	XS_45	32,334.25
XS_06	93,682.20	XS_23	76,157.93	XS_46	30,811.15
XS_07	93,338.84	XS_24	74,098.77	XS_47	29,156.00
XS_08	92,800.96	XS_25	72,214.06	XS_48	27,501.00
XS_09	92,263.07	XS_26	69,246.88	XS_49	26,457.39
XS_10	91,611.99	XS_27	65,914.25	XS_50	24,173.20
XS_11	90,960.90	XS_28	63,794.34	XS_51	22,544.63
XS_12	90,293.02	XS_29	61,173.38	XS_52	21,100.00
XS_13	89,625.13	XS_30	59,796.90	XS_53	20,762.56

Table 4-2 Generated data as to cross-sections for the modeling reach...cont'd.

XS_14	89,442.85	XS_31	58,701.32	XS_54	18,762.00
XS_15	89,260.57	XS_32	56,756.65	XS_55	16,711.43
XST_01	2,427.86	XS_33	54,763.20	XS_56	14,711.00
XST_02	1,829.11	XS_34	52,526.59	XS_57	12,711.00
XST_03	1,403.24	XS_35	50,453.35	XS_58	7,090.46
XST_04	1,122.35	XS_36	49,212.37	XS_59	6,041.09
XST_05	837.75	XS_37	47,279.45	XS_60	3,927.35
XST_06	448.08	XS_38	45,471.19	XS_61	1,382.58
XS_16	87,289.88	XS_39	43,332.27		
XS_17	86,077.21	XS_40	42,312.87		

4.3.3 The Catastrophic Inflow Hydrographs to the Tendaho Dam

According to hydrology study report of Tendaho dam WWDSE [35], the Inter Tropical Convergence Zone (ITCZ) influences the rainfall formation over the Awash basin. Two rainy seasons have been experienced on the most part of the Awash basin. Bi-modal type of rainfall dominates the Awash watershed. The main rainy season often extends from end of June through September and the small rainy season from end of February to middle of May. Thus, based on Tendaho dam hydrology study report [35], two seasons; winter (rainy season) and summer (dry season) will be considered for overtopping and piping failure simulation of subject dam.

The catastrophic probable maximum flood (PMF) inflows hydrograph to the Tendaho dam provided by WWDSE [35] have been given in the following table.

Table 4-3 Spillway size and routed peak flow for the 24 hour full PMF inflows

PMF over-topping check against the dam height at 412m				
Spillway crest elevation (m)	FRL (m)	MWL (m)	Peak inflow (m ³ /s)	Peak routed flow (m ³ /s)
402	408	411.1	6084	1849
400	409	411.0	6084	2441
400	409.5	411.2	6084	2539
400	410	411.5	6084	2639

Table 4-3 Spillway size and routed peak flow for the 24 hour full PMF inflows...cont'd

400	408	410.4	6084	2253
399	409	410.6	6084	2658
399	409.5	410.9	6084	2760
399	410	411.2	6084	2864

The detailed tabulated values of the catastrophic inflow hydrographs were measured on the hydrograph and were presented below.

Table 4-4 Detailed values of the catastrophic inflow hydrographs

Time (hr)	Full PMF inflow (m ³ /s)	Time (hr)	Full PMF inflow (m ³ /s)	Time (hr)	Full PMF inflow (m ³ /s)	Time (hr)	Full PMF inflow (m ³ /s)
1	318	61	3396	121	1432	181	429
2	319	62	3259	122	1419	182	419
3	319	63	3130	123	1404	183	409
4	318	64	3008	124	1390	184	400
5	318	65	2893	125	1375	185	391
6	317	66	2785	126	1360	186	382
7	316	67	2684	127	1344	187	374
8	315	68	2589	128	1328	188	365
9	313	69	2501	129	1312	189	357
10	315	70	2419	130	1295	190	349
11	326	71	2342	131	1278	191	341
12	353	72	2272	132	1260	192	333
13	409	73	2206	133	1243	193	326
14	502	74	2147	134	1224	194	318
15	638	75	2092	135	1206	195	311

Table 4-4 Detailed values of the catastrophic inflow hydrographs...cont'd

16	815	76	2041	136	1187	196	304
17	1027	77	1995	137	1168	197	297
18	1263	78	1954	138	1148	198	290
19	1515	79	1916	139	1128	199	283
20	1777	80	1882	140	1108	200	277
21	2044	81	1852	141	1087	201	271
22	2316	82	1825	142	1067	202	264
23	2594	83	1801	143	1046	203	258
24	2877	84	1780	144	1025	204	252
25	3164	85	1761	145	1004	205	246
26	3452	86	1745	146	983	206	241
27	3741	87	1730	147	962	207	235
28	4027	88	1716	148	941	208	230
29	4305	89	1704	149	920	209	224
30	4569	90	1694	150	899	210	219
31	4818	91	1684	151	879	211	214
32	5050	92	1675	152	858	212	209
33	5262	93	1667	153	838	213	205
34	5454	94	1660	154	819	214	200
35	5622	95	1653	155	799	215	195
36	5766	96	1647	156	780	216	191
37	5887	97	1641	157	762	217	186
38	5980	98	1635	158	744	218	182
39	6044	99	1629	159	726	219	178
40	6078	100	1623	160	708	220	174

Table 4-4 Detailed values of the catastrophic inflow hydrographs...cont'd

41	6084	101	1617	161	691	221	170
42	6062	102	1611	162	674	222	166
43	6015	103	1605	163	658	223	162
44	5947	104	1599	164	642	224	158
45	5858	105	1593	165	627	225	155
46	5754	106	1586	166	612	226	151
47	5635	107	1579	167	597	227	148
48	5502	108	1571	168	583	228	144
49	5358	109	1563	169	569	229	141
50	5203	110	1555	170	555	230	138
51	5039	111	1546	171	542	231	135
52	4870	112	1537	172	529	232	132
53	4696	113	1527	173	517	233	128
54	4522	114	1517	174	504	234	126
55	4349	115	1506	175	493	235	123
56	4178	116	1495	176	481	236	120
57	4010	117	1483	177	470	237	117
58	3848	118	1471	178	459	238	114
59	3691	119	1459	179	449	239	112
60	3540	120	1446	180	439	240	109

Based on above table 4-4, time step of the hydrograph is therefore constant as 1 hour and the peak inflow for full PMF equals to 6,084m³/s occurred at 41 hours. During case study dam failure simulation, the total duration of simulation was specified as 100 hours given that the total duration of hydrograph which is 240 hours, is unrealistically long as to be discussed later under modeling scenarios.

It is noteworthy to mention here that WWDSE [35] had designed the spillway to accommodate this catastrophic inflow hydrograph, which implies that even in case of an inflow of such great magnitude to the reservoir, the spillway can release the inflow, thereby preventing the rise of water level behind the reservoir. Although this could help to prevent the water behind the reservoir from overtopping the dam crest and hence failure of the dam itself, the water released through the spillway could still present a risk to the settlements downstream of the dam given the magnitude of the discharge. Furthermore, in case of failure of any of the spillway gates, the capacity of the spillway would be decreased significantly, making the occurrence of overtopping of dam crest and hence failure of the dam itself unavoidable.

WWDSE [35] assumed that during spilling of the PMF, the irrigation outlet tunnel flow full discharge of $78 \text{ m}^3/\text{s}$ as part of the flood and a constant discharge of $5 \text{ m}^3/\text{s}$ was released for downstream ecological. In this work, total discharge (constant) of $83 \text{ m}^3/\text{s}$ as a pilot flow in inline structure (dam) data editor was considered.

4.3.4 Rating Curves of Awash River

Ministry of Water, Irrigation and Electricity (MoWIE) has a hydrological station at Tendaho dam site on River Awash conducted periodic measurement of the flow. The gauged hydrological data provided by MoWIE spans a period from 1965 to 2007. The maximum observed flow depth on Awash River was 8.75m in 1974. These data were used for Manning roughness ‘n’ calibration for unsteady state flood routing at downstream region and detail data are given at Appendix-A of this paper.

4.3.5 Monthly Observed Flow Values of the Logia River

Ministry of Water, Irrigation and Electricity (MoWIE) has also provided flow data from 1988 to 2007 on Logia River. The maximum observed flow was $616.93 \text{ m}^3/\text{s}$ in 1988 which was specified as the maximum constant value of the gradually increasing lateral inflow to Awash River in the winter case scenarios. An average flow of $202 \text{ m}^3/\text{s}$ was also specified in the summer case scenarios. Detail data are given at Appendix-A of this paper.

4.3.6 Upstream and Downstream Boundary Conditions

There are several different types of boundary conditions available for unsteady dam break analysis. As per US Army Corps of Engineers, HEC-RAS User’s Manual [29],

the main boundary conditions and the possible place to use them are given in the following table.

Table 4-5 Summary of unsteady state flow boundary conditions

Boundary condition (BC)	Appropriate (possible) place to use BC
Flow hydrograph	U/s and d/s BC (commonly used as u/s BC)
Stage hydrograph	Either u/s or d/s BC
Rating curve	D/s BC
Normal depth	Only for d/s BC (for an open ended reach) In this case Manning's equation is used to calculate stage for each computed flows
Lateral inflow hydrograph	Internal BC (usually uniform lateral inflow is used between user specified x-sections)
Ground water (GW) interflow	Applied to a reach/storage area where GW outflow is appreciable
A time series of gate opening (for spillway)	Data entered for inline gated spillway

In this work, Tendaho dam and normal depth (i.e. open ended) boundary conditions were specified as upstream and downstream boundary conditions for modeling domain respectively. Uniform lateral inflows of $616.93\text{m}^3/\text{s}$ and $202\text{m}^3/\text{s}$ were used at Logia River cross-section-2427.862 (i.e. at u/s end) for winter and summer season scenarios respectively. Moreover, user defined time series of spillway gate openings from 9m-0m were used for both winter and summer case scenarios which will be given under section-4.4.14 of modeling scenarios.

4.3.7 Manning Roughness

The Author is unable to access any official Manning roughness values for the modeling reaches, therefore had to use arbitrarily but, judicially chosen Manning roughness values based on Chow (1959). HEC-RAS User's Manual [29] suggested to use a compiled "n" values for streams and floodplains found in open channel hydraulics, Chow (1959).

Chow [5] presents a table showing indicatory ranges of Manning roughness values for several conditions. Two of those basic categories, which are of particular interest as

far as this thesis work is considered, are the categories titled as “Minor Streams” and “Major Streams”. Chow [5] defines “Major Streams” as streams with surface width at flood stage more than 100 ft (i.e. ~30.48 m.).

In Tendaho dam’s case study, the smallest top-width at flood stage is quite greater than 30.48m. Hence, the cross-sections along the entire reach were assumed to be larger than 30.48 m and the stream was evaluated as “Major Stream” while determining the Manning roughness values. Currently it was observed that there is some vegetation (cane plantation) on the banks and on the floodplains. Based on these assumptions, the Manning roughness value ranges suggested by Chow (1959) for main channels and the floodplains in case of “Minor Streams” were identified as follows:

Minor streams (surface width at flood stage less than 100 feet)

🌿 Some weeds, light brush on banks.....0.035-0.050

Flood plains (adjacent to natural streams)

🌿 Light brush and trees:

a) Winter.....0.050-0.060

b) Summer.....0.060-0.080

Chow [5], however, suggests that roughness coefficient for major streams (i.e. surface width at flood stage more than 100 feet) is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks and adds that values of “n” may be somewhat reduced. In light of all these, Manning roughness values falling within the ranges given above were selected for the main channel and the floodplains.

In HEC-RAS model, the user can divide the reach into sub-reaches in terms of Manning roughness values. However, in the case study of Tendaho dam, the total reach was divided into just one “Manning Reach”. The Manning roughness values corresponding to each cross-section has to be specified to the HEC-RAS model to simulate the unsteady flow. Accordingly, the Author has used Manning roughness “n” values of 0.04 for main channel and 0.06 for left and right floodplains.

4.3.8 Population at Flood Risk

As per WWDSE [36], the Tendaho dam and irrigation project is to be implemented in Zone-one of Afar Regional state which comprises of four Woredas namely: Mille, Dubti, Afambo and Assaita. The Tendaho dam and reservoir are found in Mille and Dubti Woredas, whereas the proposed irrigation commands are found in Dubti, Afambo and Asasaita Woredas.

The Projected population data of 2015 (Hamle 2007) for four main towns which are located at downstream of the case study dam and reservoir was collected from Federal Democratic Republic of Ethiopia Central Statistical Agency (2013) Population Projection Report for all Regions at Wereda Level from 2014-2017. Other settlements' data due to Tendaho Sugar Development Project have been collected from Sugar Corporation of Ethiopia. Projected population data collected from CSA are given in the following table.

Table 4-6 Projected population data of main towns

Wereda (town)	Value of population		Total
	Male	Female	
Dubti Woreda (Dubti + Logia towns)	29,365	25,393	54,758
Afambo town	810	561	1,371
Assaita town	14,015	12,651	26,666
Total Population	44,190	38,605	82,795

Thus, total number of urban residents within the study domain and are at Tendaho dam break flood risk is 82,795 from which 44,190 are male and 38,605 are female.

4.3.9 Topographic Maps of the Modeling Domain

As mentioned so far, surveyed topographic maps for modeling domain at required detail, areal cover and location are not found both at Water Works Design and Supervision Enterprise (WWDSE) and at Ethiopian Mapping Agency (EMA). Due to this data limitation, cross-sectional geometric data for modeling reach and floodplain were generated from DEM and used as model input.

Moreover, Tendaho dam break induced flood inundation map was prepared by using the same DEM data collected from WWDSE following completion of modeling. The maps indicating the inundated areas will be given under chapter-5 results and discussion.

4.4 Internal Data Analysis

Detailed analyses of breach parameters for both overtopping and piping failure cases of Tendaho dam are given in the following subsequent section. Breach parameters predictions are made based on empirical equations discussed above under section-3.7.

4.4.1 Breach Parameters for Overtopping Tendaho Dam Failure Case

The following empirical equations and recommendations are used to predict the breach parameters for subject dam failure. However, final values of breach parameters will be selected based on recommendations, sensitivity test and physical existence of calculated breach values.

1. Breach width (B)

 USBR (1988)

$B (m) = 3h_w$, where, B is the average breach width, h_w is the hydraulic depth of water above breach invert at time of failure

For a h_w of 37.16 m (i.e. dam is assumed to fail at a water elevation of 412.16 m and the terminal breach bottom elevation is 375 m), the above equation yields B as 111.48 m.

 Von Thun Gillette (1990)

$B (m) = 2.5h_w + C_b$, where, B is the average breach width, h_w is the hydraulic depth of water above breach invert at time of failure, and C_b is the offset factor and is function of reservoir size, which was previously given in table 3-2.

The Tendaho dam has a reservoir size of $2.53 \times 10^9 \text{ m}^3$ at dam crest level (412m), which is greater than $1.23 \times 10^7 \text{ m}^3$; therefore, C_b for the Tendaho dam's case is 54.9 m. For a h_w of 37.16 m (i.e. dam is assumed to fail at a water elevation of 412.16 m and the terminal breach bottom elevation is 375 m) and C_b of 54.9 m, the above equation yields B as 147.8 m. Given that the invert elevation of the breach bottom is 375 m and the crest elevation of the dam is 412 m, the average breach width of

147.8m computed by the above equation must be located just between the two, at 393.21 meters elevation.

🚧 Froehlich (1995b)

$B \text{ (m)} = 0.1803 * K_o * V_w^{0.32} * h_b^{0.19}$, where, $V_w \text{ (m}^3\text{)} =$ volume of breached water, $h_b \text{ (m)}$ = height of breach

For a K_o of 1.4, V_w of $2.53 * 10^9 \text{ m}^3$ and h_b of 37.16 m, the above equation yields B as 512.2 m.

🚧 Pandya and Jitaji (2013)

As per Pandya and Jitaji (2013), an average breach width of earth-fill dam can be expressed by its height as: $B \text{ (m)} = 4 * h_d$, where, h_d , is height of the dam in meter.

The Tendaho dam has a height of 44m which was previously given under section-4.3.1: geometrical and technical data of the Tendaho dam. Therefore, for h_d of 44m, the above equation yields B as 176m.

2. Breach full formation (development) time (T_f)

🚧 USBR (1988)

$$T_f \text{ (hr)} = 0.011 * B$$

For a B of 111.48m, the above equation yields T_f as 1.23 hr.

🚧 MacDonald and Langridge-Monopolis (1984)

$$V_{erc} = 0.0261 * (V_w * h_w)^{0.769}, \text{ for earth-fill dam failure}$$

$T_f \text{ (hr)} = 0.0179 * (V_{er})^{0.364}$; Where, V_{er} is volume of eroded materials, h_w is the hydraulic depth of water above breach invert at time of failure, V_w is the volume of water above breach invert elevation at the time of breach (m^3) and T_f is breach development time.

For water elevation of 412.16 m and the terminal breach bottom elevation of 375m, the values of h_w and V_w are 37.16m and $2.53 * 10^9 \text{ m}^3$, and the above equation yields V_{er} and T_f as 7,161,902.52 m^3 and 5.6 hours respectively.

🚧 Von Thun Gillette (1990)

$$T_f \text{ (hr)} = 0.0209 * h_w + 0.25, \text{ for erosion resistant dam}$$

Where, h_w (m) = the depth of water at the dam at the time of failure,

Tendaho dam is considered as erosion resistant dam because appropriate provision of berms and rock ripraps were made by the engineer [36]. For a h_w of 37.16m, the above equation yields T_f as 1.03 hr.

🚧 Froehlich (1995b)

$$T_f \text{ (hr)} = 0.00254 * (V_w)^{0.53} * h_b^{-0.9}$$

For a V_w of $2.53 * 10^9 \text{ m}^3$ and h_w of 37.16m the above equation yields T_f as 9.45 hours.

🚧 Pandya and Jitaji (2013)

They suggest an average breach formation time of 2 hours for earth-fill dam failure.

3. Breach side slope (Z)

Froehlich (1995b) suggests breach side slope (Z) of 1 (1H: 1V) for overtopping earthen dam failure case. On other hand, Pandya and Jitaji (2013) suggest the same mean breach side slope for earth-fill dam failure by assuming trapezoidal shape of breach.

4.4.2 Breach Parameters for Piping Tendaho Dam Failure Case

Same to overtopping failure case of Tendaho dam, breach parameters for piping failure case are predicted in the following way.

1. Breach width (B)

🚧 USBR (1988)

B (m) = $3h_w$, where, B is the average breach width, h_w is the hydraulic depth of water above breach invert at time of failure in meter.

In piping failure scenarios, the dam was assumed to fail at a critical WSEL of 412 m and assumed terminal breach elevation (h_b) is 375m, similar to the overtopping failure case. For a h_w of 37m (i.e. dam is assumed to fail at a water elevation of 412m and the terminal breach bottom elevation is 375m), the above equation yields B as 111m.

🚧 Von Thun Gillette (1990)

B (m) = $2.5h_w + C_b$; Where, B is the average breach width, h_w is the hydraulic depth of water above breach invert at time of failure, and C_b is the offset factor and is function of reservoir size.

Tendaho dam has a reservoir size of $2.52 \times 10^9 \text{ m}^3$ at dam crest level (412m), which is greater than $1.23 \times 10^7 \text{ m}^3$; therefore, C_b for the Tendaho dam's case is 54.9 m. For a h_w of 37m (i.e. dam is assumed to fail at a water elevation of 412m and the terminal breach bottom elevation is 375m) and C_b of 54.9m, the above equation yields B as 147.4m.

🚧 Froehlich (1995b)

$B \text{ (m)} = 0.1803 * K_o * V_w^{0.32} * h_b^{0.19}$; where, $V_w \text{ (m}^3\text{)} =$ volume of breached water, $h_b \text{ (m)} =$ height of breach and $K_o=1$ (for piping). For V_w of $2.52 \times 10^9 \text{ m}^3$ and h_b of 37m, the above equation yields B as 365.1m.

🚧 Pandya and Jitaji (2013)

Pandya and Jitaji (2013) suggest an average breach width for earth-fill dam can be expressed by its height as: $B \text{ (m)} = 4 * h_d$, where, h_d , is height of the dam in meter. For Tendaho dam height of 44m, the above equation yields B as 176m, which is similar to overtopping Tendaho dam failure case.

2. Breach full formation (development) time (T_f)

🚧 USBR (1988)

$$T_f \text{ (hr)} = 0.011 * B$$

For a breach width (B) of 111m, the above equation yields T_f as 1.22hr.

🚧 MacDonald and Langridge-Monopolis (1984)

$$V_{er} = 0.0261 * (V_w * h_w)^{0.769}, \text{ for earth-fill dam failure}$$

$T_f \text{ (hr)} = 0.0179 * (V_{er})^{0.364}$, where V_{er} is volume of eroded materials (m^3), h_w is the hydraulic depth of water above breach invert at time of failure (m), V_w is the volume of water above breach invert elevation at the time of breach (m^3) and T_f is breach development time (hr).

For water elevation of 412m and the terminal breach bottom elevation of 375 m, the values of h_w and V_w are 37m and $2.52 \times 10^9 \text{ m}^3$, and the above equation yields V_{er} and T_f as 7,116,470.43 m^3 and 5.59 hours respectively.

🚧 Von Thun Gillette (1990)

T_f (hr) = 0.0209* h_w + 0.25, for erosion resistant dam, where, h_w (m) = the depth of water at the dam at the time of failure,

For a h_w of 37m, the above equation yields T_f as 1.02 hours.

🚧 Froehlich (1995b)

$$T_f \text{ (hr)} = 0.00254 * (V_w)^{0.53} * h_b^{-0.9}$$

For a V_w of $2.52 * 10^9 \text{ m}^3$ and h_w of 37m the above equation yields T_f as 9.46 hours.

🚧 Pandya and Jitaji (2013)

Pandya and Jitaji [19] suggested an average formation time of 2 hours for earth-fill dam failure.

3. Breach side slope (Z)

🚧 Froehlich (1995b)

Froehlich (1995b) suggested breach side slope of 0.7 (0.7H: 1V) for piping earthen dam failure case.

🚧 Pandya and Jitaji (2013)

Pandya and Jitaji (2013) suggested mean breach side slope (Z) of 1 (1H: 1V) for earth-fill dam failure, which is similar to overtopping failure case.

The last input required as to breach geometry in piping failure case was the center line of the breach, which was specified as 393.5m to the HEC-RAS model.

4.4.3 Breach Parameter Sensitivity

After an analysis of the breach parameters (breach width, B and time of failure, T_f) effect on the peak breach discharge (Q_p), Fread et al. (1998) concluded that it can be generalized that for large reservoirs Q_p is quite sensitive to B and rather insensitive to T_f , while for very small reservoirs Q_p is somewhat insensitive to B and fairly sensitive to T_f , where the reservoir volume (V_r), dam height (h_d) and the reservoir surface area (A_s) of the dam they analyzed were 250,000 acre-ft, 260 ft and 2,000 acres respectively, which they categorize as a dam with a moderately large reservoir.

Given that the top water surface area (A_s), height of dam (h_d), and total volume of reservoir (V_r) for Tendaho dam are $160.6 \times 10^6 \text{ m}^2$, 44m and $1.86 \times 10^9 \text{ m}^3$, respectively, which roughly convert to English units as 39,674 acres, 144.4ft and 1,508,149 acre-ft.

Hence, based on this analysis, Tendaho dam can be regarded as a dam having a large reservoir and it can be deduced that the peak discharge (Q_p) through the dam will mostly be influenced by the average breach width parameter (B).

4.4.4 Breach Parameter Validation

For overtopping dam failures, time elapsed from initiation of breach formation on the upstream face of the dam till full-formation of the breach may be in the range of a few minutes to usually less than an hour, depending on the height of the dam, the type of materials used in construction, the extent of compaction of the materials, and the magnitude and duration of the overtopping flow of the escaping water, and add that the time of failure is usually considerably longer for piping failures than an overtopping failure since the upstream face is slowly being eroded in the very early phase of the piping development [Fread et al, 1998; cited in Firat Bag (11)]. For this reason, Fread et al (1998) suggested to use average breach width and breach formation time predicted by using Froehlich (1995) equation. Besides, Fread et al (1998) set a range for estimated average breach bottom width (B) as; $0.5h_d \leq B \leq 8h_d$, where h_d is the height of the dam. The height and crest length of the Tendaho dam are 44 m and 412 m (exclusive of the spillway length), respectively. Hence, the lower and upper limits of this range yield minimum and maximum average breach bottom widths (B) of 22m and 352m, respectively.

Based on this rule for average breach width by Fread et al (1998), estimated average breach width values should not yield unrealistic results, i.e. the resultant breach width should be no more and most likely less than the maximum crest length of 412m provided that the local material around the dam withstands the force of escaping water and is not eroded, confining the maximum limits of the breach geometry to the dam geometry. Under this assumption, a breach bottom width of 412 m would not be possible given that the length of the dam body at lower elevation is less than 412 m.

It is suggested to check whether or not the breach bottom width fits into the actual boundaries of the dam, i.e. the breach bottom width at the terminal breach elevation does not extend outside the dam body. The breach width parameter is based on the breach height therefore; the breach height (hence, the terminal breach bottom elevation) has to be arbitrarily but judiciously chosen so that the maximum breach

bottom width does not extend beyond the actual boundaries of the Tendaho dam cross-section previously given under geometrical and technical data of the dam.

After sensitivity testing and based on the above rule set by Fread et al (1998), the following values of predicted breach parameters are selected and finally used within HEC-RAS model for subject dam failure simulation.

Table 4-7 Predicted breach parameters for overtopping Tendaho dam failure case

Breach parameter	Predicted value	Selected equation for subject dam
Terminal breach bottom width, B (m)	176	Pandya and Jitaji (2013)
Terminal breach height, h_b (m)	37	Judged by the modeler
Breach full formation time, T_f (hr)	1.03	Von Thunand Gillette (1990)
Side slope of breach Z (ZH:1V)	1:1	Froehlich (1995b), Pandya and Jitaji (2013)

Table 4-8 Predicted breach parameters for piping Tendaho dam failure case

Breach parameter	Predicted value	Selected equation for subject dam
Terminal breach bottom width, B (m)	176	Pandya and Jitaji (2013)
Terminal breach height, h_b (m)	37	Judged by the modeler
Breach full formation time, T_f (hr)	2	Pandya and Jitaji (2013)
Side slope of breach Z (ZH:1V)	0.7:1	Froehlich (1995b)

4.4.5 Automatic Manning “n” Calibration

The HEC-RAS Version 4.1 model has a built-in feature called “Automatic Manning n Calibration” that calculates the Manning roughness values for a given reach. The automatic calibration procedure computes adjustments to existing flow roughness factors to follow observed stages. The flow roughness factor must already be set in the geometry or plan and the observed stage must already be set in the unsteady flow data before implementing this calibration procedure.

In this work, selection of Manning roughness n values for main channels and floodplains was made based on Chow (1959) recommendation. However, unsteady state flow analysis outputs especially water surface elevation (WSEL) is highly affected by flow roughness. To coup up this variation, HEC-RAS User's Manual [29] recommends to calibrate flow roughness based on actual measured stages as HEC-RAS has automatic Manning roughness n , calibration option.

Hence, the Author has used measured stage data at upper reach of Awash River during unsteady state flood routing so that the HEC-RAS software will calibrate Manning roughness automatically based on measured stages, and the resulting values of water surface elevations at each x-section are reliable.

4.4.6 Initial Conditions

In addition to boundary conditions, a modeler must establish initial conditions of the system at beginning of unsteady flow simulation. As per HEC-RAS User's Manual [29], the initial conditions consist of flow and stage information at each of the x-section, as well as elevations for any storage areas defined in the system.

HEC-RAS User's Manual [29] suggests selecting arbitrary values of initial water surface elevation in such a way that its value will be between the possible maximum and minimum reservoir operation level and to use an initial flow value equivalent to base flow hydrograph of the modeling reach at each cross-sections.

Accordingly, the Author has used an initial water surface elevation value of 400m (which is between 409m/FRL and 396m/MDDL) for Tendaho dam reservoir; and initial flows of $100 \text{ m}^3/\text{s}$ for upper Awash Reach, $30 \text{ m}^3/\text{s}$ for Logia River (tributary) and total flow $130 \text{ m}^3/\text{s}$ for lower Awash Reach. Hence, all the remaining water surface elevations and flows at each x-section of modeling reaches are then calculated by the HEC-RAS software.

4.4.7 Failure Initiation Criteria

As per HEC-RAS User's Manual [29], there are two alternatives to the user as failure initiation criteria. To that end, the user can either specify a time at which the dam failure will commence or a certain critical height of water above the crest elevation of dam, which will indicate commencement of dam failure if the elevation of water trapped behind the reservoir, exceeds it. In this study, the option of specifying a

critical height of water above the dam crest elevation, which will be referred to as “critical WSEL” hereinafter, was preferred.

A literature review and needs assessment study conducted by Wahl [25] includes useful information in this regard. The study makes a quotation from Singh and Snorrason’s studies in 1982 as to the critical WSEL. It is quoted therein that: “Singh and Snorrason (1982) provided the first quantitative guidance on breach width. They also found that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.16-0.61 meters (0.5-2 feet).

Based on this, the overtopping depth for the purpose of case study of Tendaho dam was taken as the lower range as 0.16m so as to ensure conservativeness of the analyses. That is to say, the breach formation would commence when the WSEL behind the reservoir reached 412.16 m. As for the piping failure cases, the failure initiation criteria was selected as a critical WSEL of 412 m, which is equal to Tendaho dam’s crest elevation.

4.4.8 Contraction\Expansion Coefficients

The HEC-RAS integrates the expansion and contraction slope, S_e , to the Saint-Venant equations so as to take into account their effect on the flow. Where the change in river x-section is small, and flow is subcritical, the coefficient of contraction and expansion are typically 0.1 and 0.3 respectively [29]. When the change in effective x-sectional area is abrupt such at bridges, contraction and expansion coefficients of 0.3 and 0.5 are often used.

Thus, contraction and expansion coefficients of 0.1 and 0.3 for river x-sections of no constriction and 0.3 and 0.5 at bridge constriction respectively are used for modeling reach in this study.

4.4.9 Weir and Orifice Coefficients

As previously mentioned under section-2.6.2, the value of weir coefficient (C_w) for overtopping dam failure case is usually around 2.6-2.8 and for piping failure varies from 2.6-3.1.

Accordingly, the Author has used the weir coefficients of 2.6 and 3.0 for overtopping and piping Tendaho dam failure cases respectively within HEC-RAS model. Default

pipng coefficient or orifice coefficient (C_p) of 0.5 was also used for piping failure simulation within HEC-RAS model.

4.4.10 Lateral Structure Modeling

The HEC-RAS model allows the user to specify levees to account for interaction between the levees and the main channel. Since there is no levee within study domain, this option could not be used.

4.4.11 Friction Slope for Cross-Section

A detailed friction slope calculation equation used by HEC-RAS model for unsteady state dam break flood routing was previously given under Section-3.6.1. A default (average friction slope method for x-section) was used for case study dam break unsteady flood routing.

4.4.12 Low Flow Filter Option

The HEC-RAS Model has a built-in safety feature called “low flow filter” to prevent the flow values of any hydrograph from going below the initial flow values at $t=0$, thereby preventing the retention of critical errors in depth and flow in the vicinity of a rapidly rising wave front such as associated with dam-break waves or any sudden discharge releases from reservoirs. This further prevents occurrence of errors due to calculated water surface elevations lower than the streambed invert elevation.

Hence, in this work, the low flow filter was turned on within HEC-RAS model so as to prevent computational errors and increase stability of iterations.

4.4.13 Breach Progressions

A detailed review of breach progression was previously made under section-2.6.2. In this work, linear breach progression was selected within HEC-RAS model for both overtopping and piping Tendaho dam failure cases.

4.4.14 Modeling Scenarios

To investigate a potential risk of Tendaho dam failure induced flood to the settlements in the vicinity of Awash and Logia Rivers, the modeling domain was tested for a set of scenarios under overtopping and piping failures and the failure scenarios are given in the following tables.

Table 4-9 Scenario definitions for overtopping Tendaho dam failure simulation

Scenario ID	Season	Lateral flow from Logia River (m ³ /s)	Spillway gates opening height, h (m)	Type of failure	Failure criteria (m)
S1	Summer	Uniform flow of Q _{max} = 202 m ³ /s	9 (completely open)	Overtopping	412.16
S2	Summer	Uniform flow of Q _{max} = 202 m ³ /s	6	Overtopping	412.16
S3	Summer	Uniform flow of Q _{max} = 202 m ³ /s	3	Overtopping	412.16
S4	Summer	Uniform flow of Q _{max} = 202 m ³ /s	0 (completely closed)	Overtopping	412.16
Manning halved (S4m)	Summer	Uniform flow of Q _{max} = 202 m ³ /s	0 (completely closed)	Overtopping	412.16
W1	Winter	Uniform flow of Q _{max} = 616.93 m ³ /s	9 (completely open)	Overtopping	412.16
W2	Winter	Uniform flow of Q _{max} = 616.93 m ³ /s	6	Overtopping	412.16
W3	Winter	Uniform flow of Q _{max} = 616.93 m ³ /s	3	Overtopping	412.16
W4	Winter	Uniform flow of Q _{max} = 616.93 m ³ /s	0 (completely closed)	Overtopping	412.16

Table 4-10 Scenario definitions for piping Tendaho dam failure simulation

Scenario ID	Season	Lateral Flow from Logia River (m ³ /s)	Spillway gates opening height, h (m)	Type of failure	Failure criteria (m)
S4P	Summer	Uniform flow of Q _{max} = 202 m ³ /s	0 (completely closed)	Piping	412
W4P	Winter	Uniform flow of Q _{max} = 616.93 m ³ /s	0 (completely closed)	Piping	412

Based on table 4-9 and table 4-10, the only difference between the summer and winter scenarios is the value of the lateral inflow from Logia River. The values for the lateral inflow from Logia River were obtained from the daily observed flow data collected at Ministry of Water, Irrigation and Energy department of hydrology, where the

recorded maximum flow and the annual average flow were found $616.93 \text{ m}^3/\text{s}$ and $202 \text{ m}^3/\text{s}$ respectively.

In both winter and summer scenarios, a catastrophic full PMF inflow hydrograph of $6,084 \text{ m}^3/\text{s}$ peak discharge of 240 hours duration was used as the inflow hydrographs at the upstream end (i.e. Tendaho dam) of the modeling reach. Details of catastrophic inflow hydrograph were previously given in table-4.4. Given that the total durations of the hydrographs are impractically long, the modeling was performed only for duration of 100 hours, which also covered the peak magnitudes of $6,084 \text{ m}^3/\text{s}$ occurring at 41 hours.

4.4.15 HEC-RAS Model Setup

This study is largely based on two main modeling routines employed mainly by HEC-RAS software.

1. Dam breach hydrograph simulation under a set of pre-defined scenarios
2. The use of the full hydrodynamic shallow water equation to solve unsteady flood wave propagation resulting from dam failure scenarios and thus, identifying settlements at flood risk by use of inundation map at downstream region.

In this work, two cross-sections are used inside of the reservoir; therefore, HEC-RAS will perform full unsteady flow routing through the reservoir pool and downstream of the dam. The routing reach is hydraulically connected to the reservoir (storage area) with the first (most upstream) x-section. This x-section water surface is forced to the elevation of the water surface in the storage area during the unsteady flow routing. The second x-section in the pool area is required as a bounding x-section for the inline structure (the dam). Finally, the HEC-RAS model is set up to simulate the subject dam breach which will provide dam break peak flood out flows and maximum WSELs at each river cross-sections for each predefined eleven scenarios.

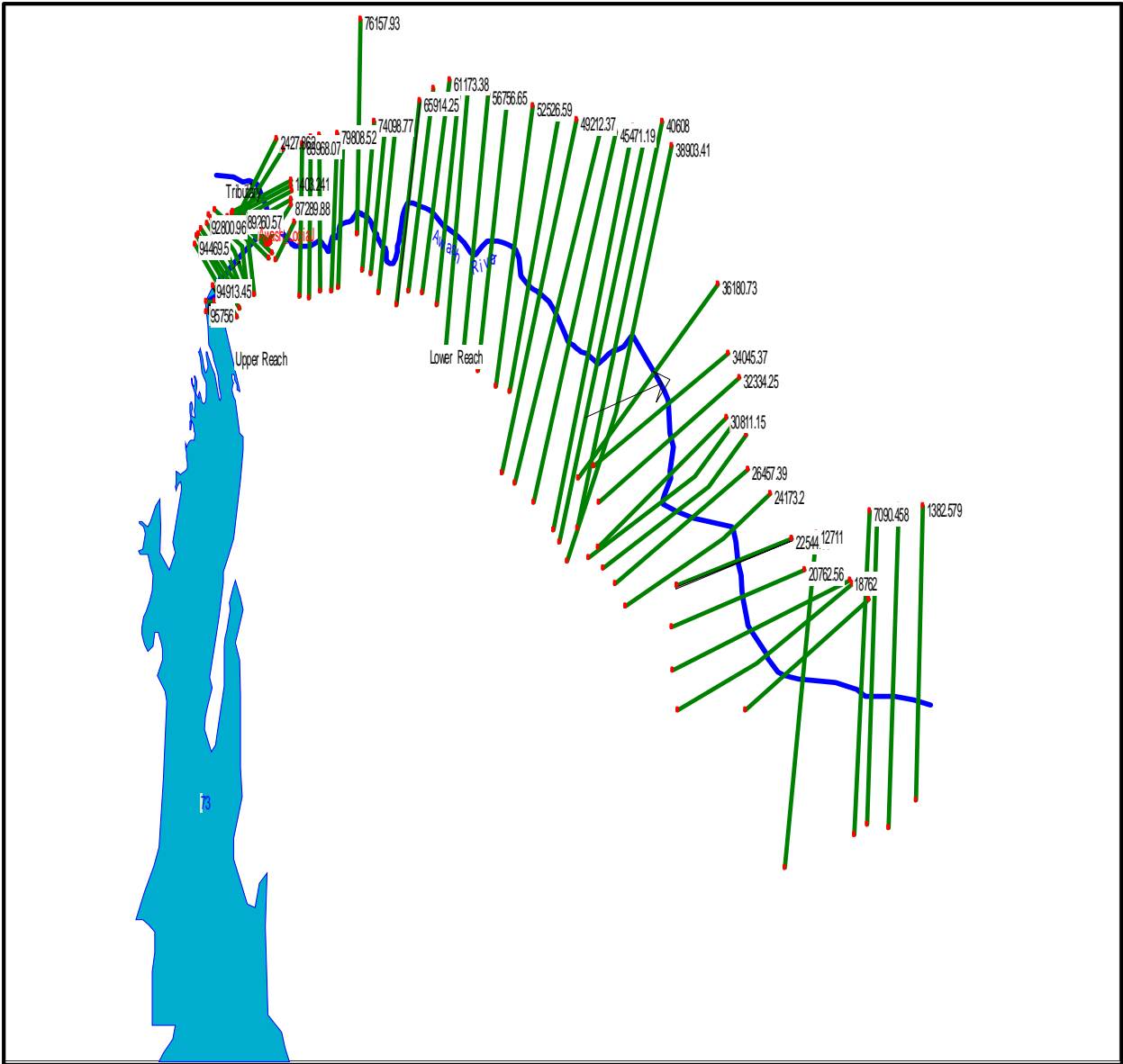


Figure 4-5 HEC-RAS model setup for modeling domain

CHAPTER FIVE

5. RESULTS AND DISCUSSION

5.1 Simulation in HEC-RAS Model

Following completion of Tendaho dam breach simulation in HEC-RAS model for each defined eleven scenarios, the following outputs were acquired:

1. Discharge hydrographs at each river station
2. WSELs hydrographs at each river station
3. Times when maximum flow and WSEL occur at each river station
4. Graphical x-sectional views showing maximum WSELs, water depths and widths at each river station
5. Discharge profile for the entire reach
6. Water surface profile for the entire reach

However, in this chapter, sample HEC-RAS simulation results at key river stations will be given and discussed. The discussions will be made referring detail location map as previously given under section-3.3: modeling domain in fig 3-3. Furthermore, standard HEC-RAS model outputs at selected River stations are attached at Appendix-C of this paper.

Comparisons of the HEC-RAS simulation results at River stations-87289.88 and at River station-72214.06 for all predefined eleven scenarios are given in the following tables.

Table 5-1 Sample HEC-RAS simulation results at RS-87289.88 for all scenarios

Scenario ID	Max. Discharge (m ³ /s)	Time of Max. Discharge (hrs)	Max. WSEL (m)	Time of Max WSEL(hrs)
S1	5,347.42	88.00	388.84	87.50
S2	5,531.76	86.00	388.91	85.50
S3	70,302.84	83.50	396.70	84.50
S4	75,766.97	57.00	397.05	57.50
S4m	125,632.10	56.50	395.84	57.00

Table 5-1 Sample HEC-RAS simulation results at RS-87289.88 for all scenarios...cont'd

W1	5,760.44	88.00	388.98	87.50
W2	5,947.08	86.00	389.04	85.50
W3	73,329.60	83.50	397.01	84.50
W4	77,892.61	57.00	397.24	57.50
S4P	65,840.21	56.00	396.03	56.50
W4P	66,345.68	56.00	396.23	56.50

Table 5-2 Sample HEC-RAS simulation results at RS-72214.06 for all scenarios

Scenario ID	Max. Discharge (m ³ /s)	Time of Max. Discharge (hrs)	Max. WSEL (m)	Time of Max WSEL(hrs)
S1	5,276.96	91.00	374.44	90.50
S2	5,501.60	89.00	374.49	88.50
S3	65,834.49	86.50	380.10	86.00
S4	66,696.75	59.50	380.18	59.50
S4m	104,003.10	58.00	378.98	58.00
W1	5,692.11	91.00	374.54	90.50
W2	5,915.73	89.00	374.59	88.50
W3	68,915.66	86.00	380.28	86.00
W4	69,562.20	59.50	380.34	59.50
S4P	55,283.73	58.50	379.54	58.50
W4P	60,784.03	59.00	379.82	58.50

As per HEC-RAS simulation results in table 5-1 and table 5-2, the failure naturally occurs earlier as the spillway gate openings are decreased, which is because the spillway gates are unable to discharge the incoming flow, thereby causing faster WSEL rise inside the reservoir under those circumstances. Hence, the more the spillway gate openings are decreased, the sooner the WSEL behind the reservoir reaches the “critical WSEL”, which triggers the failure of dam and formation of the breach, hence release of the water volume trapped behind the dam.

Another issue that is noteworthy to mention is that in the “spillway gates openings of 9m and 6m” case (Scenarios S1 and S2 and W1-W2); there is still a high rise of WSEL at each cross-section although the dam does not fail and breach is not formed. For instance, the maximum WSEL at RS-89260.57, 69246.88, 22544.63 and 20762.56 for scenario S1 are 388.84m, 372.43m, 354.69m and 353.48m respectively, which are still enough to inundate some settlements in the vicinity and to risk lives. Therefore, it is evident that completely opening (9m) the spillway gates should not be considered as an action while devising an Emergency Action Plan (EAP).

A third issue that comes out of the outputs in table-5.1 and table-5.2 is that the change of 50% in Manning roughness values did not have significant effect on the computed maximum WSELs, but on their time of occurrence. For RS-87289.88 for instance (table-5.1), a decrease of 50% in Manning roughness values resulted in only some 0.3% of decrease in maximum WSEL and a 0.5hour decrease in its time of occurrence which has significance in terms of arrival time of the flood to the settlement at downstream of the dam. The 50% decrease in Manning roughness values further caused some 66% of increase in the maximum discharge in comparison to the first case, but 0.5 hour decrease in its time of occurrence which is same as that of maximum WSEL time. These percentages, certainly, do not yield any strict correlations but still present rough indicators as to the effect of change in Manning roughness values to maximum WSELs and discharges computed. Given that the WSELs occurring at each cross-section are of the essence in the analysis, it can be deduced that the computed WSELs are close to the actual values and the judicious selection of the Manning roughness values do not create any problems in terms of accuracy of the outputs, as far as the WSELs are considered. However, accurate Manning roughness data should be sought to obtain more precise flood arrival times so as to devise sound emergency action plans, where flood arrival times will be of crucial importance as well.

A fourth issue that also comes out of the results in table-5.1 and table-5.2 is that the overtopping and piping mode of failures did not have significant difference on the computed maximum WSELs. For instance in table-5.1, a piping mode of failure has resulted in only some 0.3% of decrease in maximum WSEL and one hour decrease in its time of occurrence in comparison to overtopping mode of failure for both summer and winter cases. Furthermore, piping mode of failure has resulted some 13% of

decrease in the maximum discharge in comparison to the first case and one hour decrease in its time of occurrence which is the same value of difference as that of maximum WSEL time of occurrence. These percentages, certainly, do not yield any strict correlations but still present rough indicators as to the effect of overtopping and piping mode of failures values to maximum WSELs and discharges computed. Given that the WSELs occurring at each cross-section are of the essence in the analysis, it can be deduced that the computed WSELs for both mode of failures are close to each other; selection of the overtopping mode of failure for Emergency Action Plan (EAP) do not create any problems in terms of appropriate mode of failure selection for Tendaho dam failure simulation case, as far as the WSELs are considered. However, accurate Manning roughness data is still required for more precise flood arrival times so as to devise sound emergency action plans as stated in the above paragraph.

Furthermore, it is evident that the change in value of lateral flow from Logia River does not have any significant effect either on the maximum WSELs or on the maximum discharges, given the insignificance of the lateral flow value in comparison to the observed discharges in the order of $10,000\text{m}^3/\text{s}$.

It appears that Scenario W4 yields the largest discharge for RS-87289.88. As for RS-72214.06, Scenario W4 yields highest discharge and WSEL values. Therefore, the output data for Scenario W4 was used while preparing the inundation map for the reach. The map was prepared by plotting the maximum water surface width at each cross-section for Scenario W4 on their places on the Digital Elevation Model (DEM) whilst accounting for the respective maximum WSELs as well, as shown in the following figure.

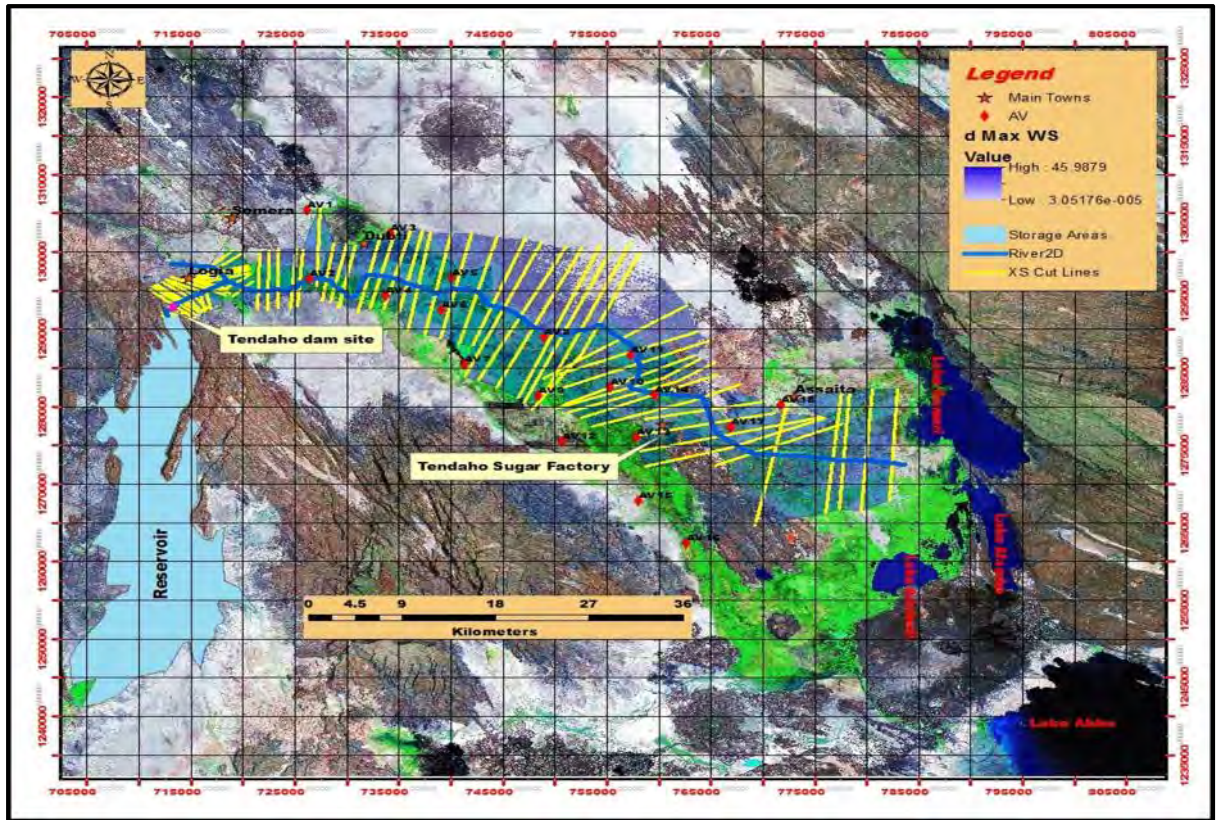


Figure 5-1 Tendaho dam break flood inundation map

The maximum WSELs and flood surface widths for Scenario W4 at each River cross-section, used while preparing the inundation map in above fig 5-1, is given in the following table.

Table 5-3 Maximum WSELs and flood surface widths results at each river station

Cross-Section ID	River station from d/s end/junction (m)	Max WSEL (m)	Max Surface Width (m)
XS_01	95,756	432.99	1,676.95
XS_02	95,334.76	432.97	542.00
XS_03	94,887.95	410.55	503.54
XS_04	94,469.50	405.56	4,376.53
XS_05	94,025.55	405.30	4,325.31
XS_06	93,682.20	404.89	4,625.79
XS_07	93,338.84	403.91	4,541.61
XS_08	92,800.96	403.21	3,852.04

Table 5-3 Maximum WSELs and flood surface widths results at each RS...cont'd

XS_09	92,263.07	402.52	3,809.03
XS_10	91,611.99	402.02	4,684.27
XS_11	90,960.90	401.53	4,626.23
XS_12	90,293.02	401.19	4,191.47
XS_13	89,625.13	401.12	4,178.77
XS_14	89,442.85	400.96	2,927.46
XS_15	89,260.57	397.24	2,927.46
XST_01	2,427.86	397.24	4,191.61
XST_02	1,829.11	397.24	4,938.20
XST_03	1,403.24	397.24	4,211.92
XST_04	1,122.35	397.24	4,326.36
XST_05	837.75	397.24	5,427.78
XST_06	448.08	397.24	4,739.98
XS_16	87,289.88	397.24	3,156.24
XS_17	86,077.21	395.39	2,688.21
XS_18	83,968.07	393.34	5,471.60
XS_19	82,813.79	392.59	5,934.18
XS_20	81,627.94	391.86	4,761.85
XS_21	79,808.52	390.17	5,179.18
XS_22	78,517.52	386.31	4,871.62
XS_23	76,157.93	382.64	8,952.95
XS_24	74,098.77	381.50	5,604.02
XS_25	72,214.06	380.34	4,996.19
XS_26	69,246.88	377.92	4,786.00
XS_27	65,914.25	376.06	8,114.90

Table 5-3 Maximum WSELs and flood surface widths results at each RS...cont'd

XS_28	63,794.34	374.91	8,281.98
XS_29	61,173.38	374.22	9,402.98
XS_30	59,796.90	373.85	10,549.13
XS_31	58,701.32	373.55	12,699.60
XS_32	56,756.65	372.86	12,087.99
XS_33	54,763.20	372.24	12,615.62
XS_34	52,526.59	371.69	12,711.47
XS_35	50,453.35	370.65	10,599.99
XS_36	49,212.37	370.01	13,344.25
XS_37	47,279.45	369.28	12,984.59
XS_38	45,471.19	368.48	12,712.13
XS_39	43,332.27	367.75	13,374.90
XS_40	42,312.87	367.55	13,335.92
XS_41	40,608.00	366.44	12,740.89
XS_42	38,903.41	365.23	13,711.10
XS_43	36,180.73	364.21	12,874.38
XS_44	34,045.37	363.39	12,542.25
XS_45	32,334.25	362.70	10,102.14
XS_46	30,811.15	362.20	13,568.74
XS_47	29,156.00	361.26	10,363.76
XS_48	27,501.00	360.88	10,084.88
XS_49	26,457.39	360.15	7,991.60
XS_50	24,173.20	358.66	10,054.72
XS_51	22,544.63	358.13	9,167.74
XS_52	21,100.00	357.70	9,057.28

Table 5-3 Maximum WSELs and flood surface widths results at each RS...cont'd

XS_53	20,762.56	356.73	11,691.24
XS_54	18,762.00	355.32	14,062.13
XS_55	16,711.43	354.89	12,842.91
XS_56	14,711.00	351.76	10,215.00
XS_57	12,711.00	348.94	14,774.91
XS_58	7,090.46	347.58	13,450.10
XS_59	6,041.09	347.11	12,898.10
XS_60	3,927.35	346.36	14,449.65
XS_61	1,382.58	345.37	13,320.59

For dam break simulation in Scenario W4, the closest (in terms of time of occurrence of flooding WSEL) settlement under inundation risk in case of Tendaho dam is Logia town to which the flooding WSEL (not the flood wave) reaches at $t = 5$ hours, as given below in table 5-7. This implies that there would be ample time to warn the settlement for evacuation in case of failure of the Tendaho dam provided that the warning is issued before a certain reservoir elevation; the maximum operation level (FRL) of 409m for instance, is exceeded.

Another issue with crucial importance in terms of inundation and life risk is that the settlements in the vicinity of the modeling reach may still be under risk of inundation even before the dam has failed, because of the rise in WSEL of the respective Awash due to the high magnitude of water released by the spillway gates. For instance, for Winter case, in the “spillway gates completely open Scenario (W1)”, the discharge released through the spillway is about $5,197 \text{ m}^3/\text{s}$ in magnitude for a certain duration, which is enough to inundate Logia town which is very close to or nearby the streambed. Furthermore, there might be people working on sugar cane plantation lands located on the floodplains at that time as well as people working at Sugar Factory or other industrial facilities/plants located close to the streambed or on the floodplain, whose lives might be under risk. Therefore, an emergency evacuation warning issuance may still be necessary when the dam does not fail but the spillway releases a discharge of significant magnitude to downstream to prevent failure of the

dam itself. For such a purpose, a critical WSEL for each cross-section and the critical spillway discharge that can cause such a WSEL should be identified, and a secondary warning time should be calculated for pre-dam failure conditions. That is to say, the emergency action plans should not solely be based on the failure of the dam itself.

The four main settlements that are located downstream of the dam and are also located close to the streambed are Logia town in the vicinity of Awash RS-89260.57, Dubti town in the vicinity of Awash RS-69246.88, Assaita town and Tendaho Sugar Factory in the vicinity of Awash RS-22544.63. For Scenarios W1-W4, the time when the WSEL in the vicinity reach these settlements' elevations and the settlements' details are given in the following tables.

Table 5-4 Summary analysis of Scenario W1 for four settlements located close to streambed

Name of settlement (town)	Settlement Elev. (m)	Distance to dam (Km)	Max WSEL (m)	Max flood depth (m)	Max flooding width (m)	Time when WSEL reaches settlement's elev. (hr)	Time of initiation of breach formation (hr)
Logia	387	5.65	388.98	2.05	1,726.14	9.00	NA
Dubti	370	25.67	372.52	1.63	2,947.11	17.00	
Assaita	353	72.37	354.78	2.46	6,451.57	35.00	
Sugar Factory	353	72.37	354.78	2.46	6,451.57	35.00	

Table 5-5 Summary analysis of Scenario W2 for four settlements located close to streambed

Name of settlement (town)	Settlement Elev. (m)	Distance to dam (Km)	Max WSEL (m)	Max flood depth (m)	Max flooding width (m)	Time when WSEL reaches settlement's elev. (hr)	Time of initiation of breach formation (hr)
Logia	387	5.65	389.04	2.08	1,749.32	9.00	NA
Dubti	370	25.67	372.57	1.66	2,989.73	17.00	
Assaita	353	72.37	354.88	2.50	6,641.48	35.00	
Sugar Factory	353	72.37	354.88	2.50	6,641.48	35.00	

Table 5-6 Summary analysis of Scenario W3 for four settlements located close to streambed

Name of settlement (town)	Settlement Elev. (m)	Distance to dam (Km)	Max WSEL (m)	Max flood depth (m)	Max flooding width (m)	Time when WSEL reaches settlement's elev. (hr)	Time of initiation of breach formation (hr)
Logia	387	5.65	396.99	8.13	2,927.46	8.00	80.50
Dubti	370	25.67	377.96	5.67	4,795.48	15.00	
Assaita	353	72.37	358.25	4.88	9,198.32	20.00	
Sugar Factory	353	72.37	358.25	4.88	9,198.32	20.00	

Table 5-7 Summary analysis of Scenario W4 for four settlements located close to streambed

Name of settlement (town)	Settlement Elev. (m)	Distance to dam (Km)	Max WSEL (m)	Max flood depth (m)	Max flooding width (m)	Time when WSEL reaches settlement's elev. (hr)	Time of initiation of breach formation (hr)
Logia	387	5.65	397.24	8.38	2,927.46	5.00	54.00
Dubti	370	25.67	377.92	5.63	4,786.00	9.00	
Assaita	353	72.37	358.13	4.77	9,167.74	28.00	
Sugar Factory	353	72.37	358.13	4.77	9,167.74	28.00	

It can be deduced from table 5-4 to table 5-7 that, through an analysis oriented around the occurrence of dam failure, these four settlements seem to have indefinite time for issuance of a warning as the dam does not fail in Scenarios W1-W2. It is, however, clear that times of WSEL to reach the elevation of these settlements are known as presented in above tables. Therefore, an evacuation warning should be issued before that times are elapsed for each scenarios W1-W2. For instance, from the above table 5-4 to table 5-5 it is seen that it takes 9, 17, 35 and 35 hours respectively for the WSEL to reach the elevations of these four settlements in Scenarios W1-W2. Thus, an evacuation warning should be issued before that time has elapsed, and preferably well before 9 hours in scenarios W1-W2.

Furthermore from above table 5-6 and table 5-7, it is observed that dam failure occurs after the time when flooding WSEL reaches all the four settlements' elevation. Based

on table 5-6 and table 5-7, Scenario W4 can be considered to yield the most favorable dam failure times and flood reaching times as far as the four settlements are concerned given that the scenario W4 not only yields a high time of dam failure but also the highest flooding WSEL arrival times at furthest and densely populated Assaita town and Tendaho Sugar Factory, hence allowing for longer evacuation time.

An emergency action plan devised based on the gate opening configuration in Scenario W4 (three gates completely closed case) would allow about 5 hours of time before the WSEL reaches at Logia settlement's elevation. Hence, in case of such an emergency action plan, the issuance of evacuation warning may be decided based on the progress of the inflow and should take place well before 5 hours for the subject reach (in the case of Logia town), giving the evacuees ample time. Once the residents in the settlements are safely evacuated, the spillway gates can be opened completely so as to secure the dam against overtopping failure.

Based on the numerical outputs of scenario W4, the reservoir WSEL at 5 hours appears to be 408 m, an elevation lower than the maximum operating level of 409 m as well as the overtopping failure criterion of 412.16 m. Therefore, the safety of the dam can still be ensured by opening the spillway gates completely after 5 hours. Given that the specified inflow hydrograph peaks at 41 hours with peak discharge of about $6,084\text{m}^3/\text{s}$ and that the spillway at its maximum capacity can release that discharge, the reservoir WSEL would certainly decrease after 37 hours, thereby securing the dam. The water thus released would however still pose some risk to the properties close to the streambed such as factories, agricultural lands, roads, administrative buildings, residential areas, environmentally sensitive areas etc. Nevertheless, the life risk would be minimized by such an approach although property damage might be inevitable. The 5 hours of time given would further allow the officials to take necessary measures in the settlements such as closing the roads, ensuring electricity and food supplies, securing official documents, reinforcing security measures in place, and take further measures such as deploying levees to partially obstruct or divert the potential flood.

Finally, the excel plots of peak discharge hydrographs and maximum WSEL hydrographs at four densely populated settlements for the worst case scenario (W4) simulation within HEC-RAS model are given in the following figures.

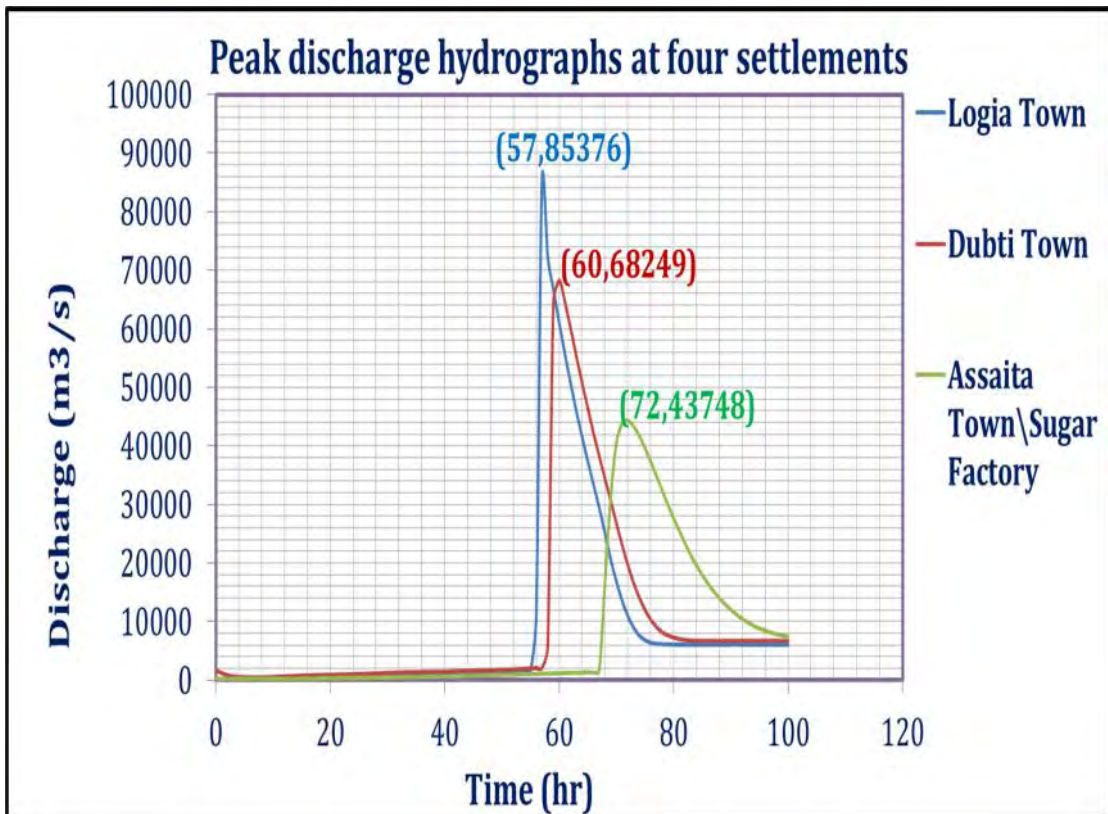


Figure 5-2 Peak discharge hydrographs at four settlements

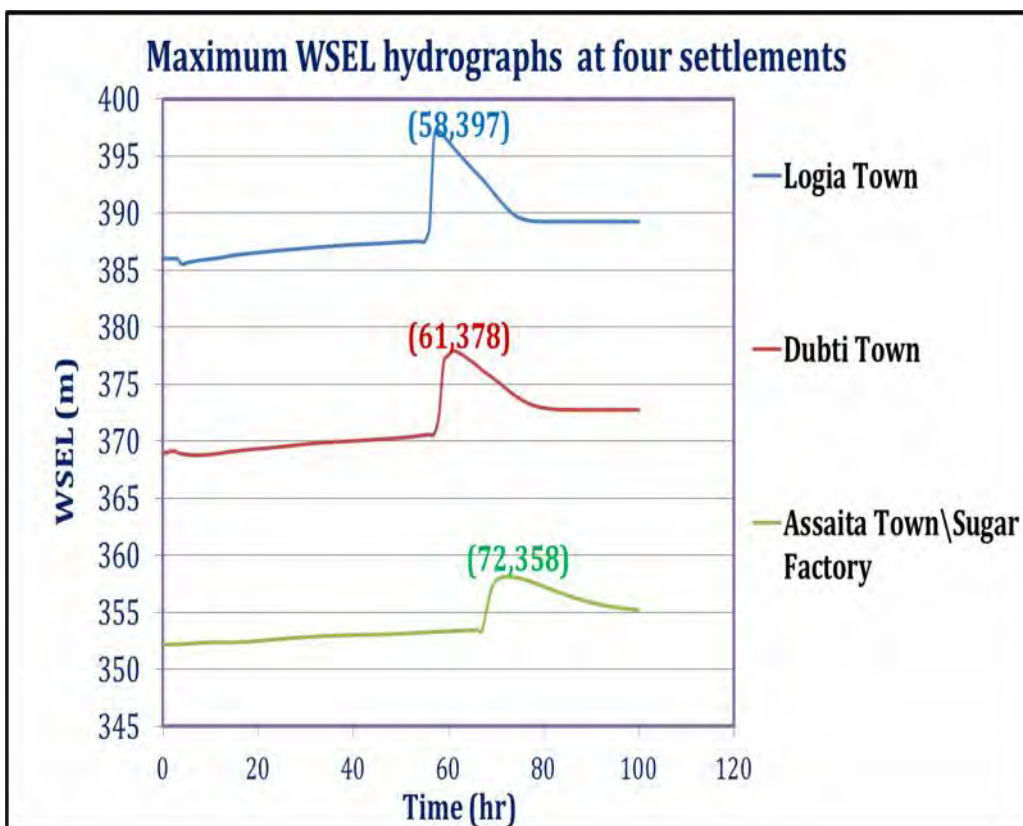


Figure 5-3 Maximum WSEL hydrographs at four settlements

The risk assessment of the Tendaho dam failure induced floods will further be delineated under the following subsequent section.

5.2 Risk Assessment

Using the inundated area plot and the data as to maximum WSEL occurring at each river cross-section, settlements under risk of inundation and loss of life are identified. Lists of these settlements showing their populations as well as the flood severity ratings and available warning times with respect to time of dam failure for worst case Scenario W4 are given in the following table. Flood severity ratings and available warning time prediction are made based on previously given equation (3.23). Please note that the following assumptions were made while preparing table 5-8:

1. The mean annual discharge ($Q_{2.33}$) for settlements located between River station-94469.5 to 89260.57 was assumed as $83\text{m}^3/\text{s}$ because during subject dam break simulation, Tendaho dam was assumed to release a total discharge of $83\text{m}^3/\text{s}$ to downstream region.
2. The mean annual discharge ($Q_{2.33}$) for settlements located between River station-89260.57 to 1382.579, on the other hand, was assumed as $83\text{m}^3/\text{s} + 202\text{m}^3/\text{s}$ where the second term was added to account for the tributary flow from Logia River after River station-89260.57, which was assumed to have a mean annual average flow value of $202\text{m}^3/\text{s}$ based on monthly peak flow observations data provided by Ministry of Water, Irrigation and Electricity (MoWIE) at Logia River station.
3. The peak discharge caused by dam failure (Q_{df}) for each settlement was taken as the average of the peak discharges at upstream and downstream cross-sections bounding the respective reach, based on HEC-RAS outputs.
4. The maximum flooding width (W_{df}) for each settlement was taken as the average of the maximum water surface widths at upstream and downstream cross-sections bounding the respective reach, based on HEC-RAS outputs.
5. Similarly, the available warning time was taken as the average of time of peak discharge for the upstream and downstream cross-sections bounding the respective reach less the time of initiation of breach formation, assuming that flood warning is issued immediately before the failure of the dam.

Table 5-8 Settlements under flood risk and their flood severity type

Name of the Settlement	DV (m ² /s)	Flood Severity	Time of dam failure (hr)	Warning time available (hr)
Logia town	29	Medium	54.0	3.0
Sub-HQ (AV-2)	15	Medium	54.0	4.0
Secondary School	15	Medium	54.0	4.0
Health Center	15	Medium	54.0	4.0
AV-1	10	Medium	54.0	4.5
Elementary School	10	Medium	54.0	4.5
Health post	10	Medium	54.0	4.5
Dubti town	10	Medium	54.0	6.3
AV-3	8	Medium	54.0	7.0
Elementary School	8	Medium	54.0	7.0
Health post	8	Medium	54.0	7.0
AV-4	7	Medium	54.0	7.3
Elementary School	7	Medium	54.0	7.3
Health post	7	Medium	54.0	7.3
Sub-HQ (AV-5)	5	Medium	54.0	8.8
Secondary School	5	Medium	54.0	8.8
Health Center	5	Medium	54.0	8.8
Sub-HQ (AV-6)	4.3	Low	54.0	9.5
Secondary School	4.3	Low	54.0	9.5
Health Center	4.3	Low	54.0	9.5
AV-7	4.5	Low	54.0	10.3
Elementary School	4.5	Low	54.0	10.3
Health post	4.5	Low	54.0	10.3
AV-8	3.9	Low	54.0	11.8
Elementary School	3.9	Low	54.0	11.8
Health post	3.9	Low	54.0	11.8
AV-9	3.8	Low	54.0	12.5
Elementary School	3.8	Low	54.0	12.5
Health post	3.8	Low	54.0	12.5
Assaita town	4.8	Medium	54.0	18.0
Sugar Factory	4.8	Medium	54.0	18.0
Health post	4.8	Medium	54.0	18.0
Secondary School	4.8	Medium	54.0	18.0
Health Center	4.8	Medium	54.0	18.0
Hospital	4.8	Medium	54.0	18.0
Police station	4.8	Medium	54.0	18.0
Clinic	4.8	Medium	54.0	18.0
Afambo town	3.3	Low	54.0	19.3
AV-15	3.5	Low	54.0	21.8
Elementary School	3.5	Low	54.0	21.8
Health post	3.5	Low	54.0	21.8
AV-16	3.1	Low	54.0	23.3
Elem. School	3.1	Low	54.0	23.3
Health post	3.1	Low	54.0	23.3

Based on the severity types identified for each settlement under flooding risk in table 5-8, the expected numbers of fatalities are calculated using the suggested coefficients previously given under section-3.8 in table 3-4 for each of the following four cases.

1. The people under risk have precise understanding of the flood warning.
2. The people under risk have vague understanding of the flood warning.
3. The warning is issued not immediately before the failure of the dam but 2 hours later.
4. A combination of cases 2 and case 3.

The number of fatalities thus calculated is given below in table 5-9. Furthermore detail calculation is given at Appendix-D of this paper.

Table 5-9 Number of predicted fatalities in the settlements under Tendaho dam break flood risk for four cases

Name of the Settlement	Population	of Fatalities for case-1	of Fatalities for case-2	of Fatalities for case-3	of Fatalities for case-4
Logia town	36,505	366	1,096	731	1,461
Sub-HQ (AV-2)	1,750	18	53	18	53
Secondary School	1,540	16	47	16	47
Health Center	30	1	1	1	1
AV-1	1,206	13	37	13	37
Elementary School	520	6	16	6	16
Health post	10	1	1	1	1
Dubti town	18,253	183	548	183	548
AV-3	1,206	13	37	13	37
Elementary School	520	6	16	6	16
Health post	10	1	1	1	1
AV-4	1,105	12	34	12	34
Elementary School	520	6	16	6	16
Health post	10	1	1	1	1
AV-5	1,750	18	53	18	53
Secondary School	1,540	16	47	16	47
Health Center	10	1	1	1	1
AV-6	1,750	1	1	1	1
Secondary School	1,540	1	1	1	1
Health Center	10	0	0	0	0
AV-7	1,105	1	1	1	1
Elementary School	520	1	1	1	1
Health post	10	0	0	0	0

Table 5-9 Number of predicted fatalities in the settlements under Tendaho dam break flood risk for four cases...cont'd

AV-8	1,105	1	1	1	1
Elementary School	520	1	1	1	1
Health post	10	0	0	0	0
AV-9	1,206	1	1	1	1
Elementary School	520	1	1	1	1
Health post	10	0	0	0	0
Assaita town	26,666	267	800	267	800
Sugar Factory	3,264	33	98	33	98
Health post	10	1	1	1	1
Secondary School	1,540	16	47	16	47
Health Center	10	1	1	1	1
Hospital	120	1	4	1	4
Police station	50	1	2	1	2
Clinic	20	1	1	1	1
Afambo town	1,371	1	1	1	1
AV-15	1,206	1	1	1	1
Elementary School	520	1	1	1	1
Health post	10	0	0	0	0
AV-16	1,206	1	1	1	1
Elementary School	520	1	1	1	1
Health post	10	0	0	0	0
Total Number of Fatalities		1,012	2,972	1,377	3,337

Based on table 5-9, understanding of the flood warning plays a major role in terms of minimizing the potential fatalities given that the number of fatalities increase by 1,960 (194%) in case of Tendaho dam failure when the understanding of the flood warning is assumed to be vague.

Furthermore, the time of issuance of warning does not seem to have significant effect on the number of fatalities, which is understandable as the settlements at 2 hours distance to the flood wave for all settlements (villages) except Logia town. However, for the nearest settlement, Logia town the number of fatalities increase by 365 (100%) when warning issued 2 hours after failure of the Tendaho dam for understanding of the flood warning is assumed to be precise and by 365 (33%) when warning issued 2 hours after failure of the Tendaho dam for understanding of the flood warning is assumed to be vague. This is because, a high percentage of life loss resulting from dam failure occurs in the first 15 mi (25 km) downstream from a dam that has failed

and loss of life, as a percentage of people at risk, becomes very small more than 15 mi (25 km) downstream from a dam for two main reasons [37]:

1. First these downstream areas receive warning that usually is much better than the warning, if any, issued in areas nearer the dam
2. Second, the energy exhibited by the flood is lessened, the flood rises at a slower rate and the leading edge of the flooding usually moves at a slower rate in these downstream areas

CHAPTER SIX

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusion

Through this thesis work, the potential flow conditions at downstream in the event of probable failure of the Tendaho dam, located in Afar Region of Ethiopia, were analyzed under eleven different scenarios using the Hydrologic Engineering Center River Analysis System (HEC-RAS V4.1) software developed by the US Army Corps of Engineers (USACE, 2010).

The variables in the scenarios defined for analyses were mainly related with the parameters for the potential breach formed on the said dam when it fails, and the spillway gate openings. Eight of the scenarios were geared towards an analysis of the overtopping failure of the dam. Two of the scenarios were to analyze the piping failure of the dam, where the breach parameters were different than those of overtopping scenarios.

The last scenario had different Manning roughness values than the foregoing ten so as to investigate the sensitivity of the analyses to Manning roughness value, as the Author, due to lack of measured data, had to use fictitious Manning roughness values for the cross-sections in the reach based on Chow (1959). Outputs sought were the maximum WSELs and the peak discharges at each cross-section station used to define the modeling reach, and the time thereof. Overtopping failure scenarios yielded more severe flow conditions downstream of Tendaho dam as the cross-sectional area of the breach in overtopping failure scenarios was greater than that of in piping failure scenarios. The worst case scenario among the overtopping scenarios was found to be Scenario W4, where all three spillway gates were completely closed. An inundation was therefore prepared for the subject reach based on the maximum WSEL and water surface width outputs of Scenario W4.

The inundation map was later used to identify the settlements that are under inundation risk as per Scenario W4. The settlements thus identified were then analyzed for potential loss of life in the event of failure of the dam, as per the method suggested by Graham

(1999). It was found that several neighborhoods at downstream of the dam, being located on a flat plain, would face risk of inundation if Tendaho dam were to fail. It was found that again the nearby town to the dam, Logia, being densely populated settlement, would experience the highest number of fatality even though the flood wave would lose its strength when it spread out to flat plain Assaita town is located on. Fortunately, it would take about 18 hours for the flood wave from Tendaho dam to arrive at Assaita town, which would be ample time to take additional flood mitigation measures in the vicinity of the settlements and/or evacuate the areas under risk.

Next to Logia town, the nearby town to Tendaho dam is Dubti town, being densely populated, would experience the second number of fatality. It would take about 3 and 6.3 hours for the flood wave from Tendaho dam to arrive at Logia and Dubti towns respectively. The most distant one is Agricultural Village sixteen (AV16) where the resulting dam failure induced flood wave would reach in about 23.3 hours. In the event of failure of Tendaho dam, these settlements (villages) would require a swiftly conducted evacuation, which would require prior studies and training as to management and implementation thereof, respectively.

The best scenario in terms of managing the high magnitude of inflow (with a peak discharge of about 6,084 m³/s and total duration of 240 hours) whilst preventing the failure of the dam and critical inundation of the settlements downstream, on the other hand, was found to be Scenario W4, where all three spillway gates were completely closed. Hence, Scenario W4 would offer the most favorable conditions in the occurrence of such an inflow to the dam, and should be used while preparing a management plan for the dam and the emergency action plans.

As for Scenario S4m, which had the same configuration with Scenario S4 except for the halved Manning roughness values and which was used to identify the sensitivity of the outputs to Manning roughness value; it was found that a 50% reduction in Manning roughness values did not have significant effect on the computed maximum WSELs but, on their time of occurrence. For River station 87289.88 (table 5-1), a decrease of 50% in Manning roughness values resulted in only some 0.3% of decrease in maximum WSEL but a 0.5 hour decrease in its time of occurrence, which has significance in terms of

arrival time of the flood to the settlement at downstream of the dam. The 50% decrease in Manning roughness values further caused some 66% of increase in the maximum discharge in comparison to the first case, with a 0.5 hour decrease in its time of occurrence. These variations do not result in significant changes in terms of maximum WSELs but time of occurrence thereof and the peak discharges at each cross-section. Therefore, it was deduced that judicious selection of Manning roughness values would not have any significant effect while preparing inundation maps but, emergency action plans and evacuation plans. Therefore, in cases where measured Manning roughness data is not available, a certain safety time margin in terms of hours might be provided for the computed flood wave arrival times by repeating the same sensitivity test for a range of possible Manning roughness values.

6.2 Recommendation

The analyses indicated that there would be the need to:

- 🚧 Prepare emergency action plan for the dam.
- 🚧 Reinforce the existing flood protection structures (for irrigation commands) and construct new ones for downstream settlements
- 🚧 Commission an observation station farther upstream of the dam for real time feedback to the dam staff.
- 🚧 Conduct a socio-economic and environmental impact and risk assessment of the dam.
- 🚧 Conduct training geared towards implementation of the said emergency action plan.

6.2.1 Emergency Action Plan

A comprehensive emergency action plan should be prepared that indicates evacuation means, evacuation routes, safe and easily accessible gathering locations, people responsible of managing the evacuation in each village, sub-head, head quarter, the duties and responsibilities of each party including the dam personnel, state Authorities such as the governorship, sub-governorship, municipalities, village headmen, gendarmerie, military forces, and other state institutions responsible of the subject dam. To prevent off-scene losses, it should be beneficial to construct shelters for the evacuees to take refuge at

the gathering points with separate phone lines and reserve supplies such as medicines, food, water, clothing and heating. In addition to risk to life and property, emergency action plans should further take into account potential damages to any environmentally sensitive areas such as the Lake Gamari, Lake Afambo and Lake Abbe which are located at the downstream most end of the modeling reach in case of the Tendaho dam (see fig 5-1). Hence, another potential consequence of the failure of the Tendaho dam would be severe damage to the bio-diversity and habitat in Lake Gamari, Lake Afambo and Lake Abbe, which could not be mended by monetary allocations but nature itself.

6.2.2 Dykes, Diversion Channels and Flood Detention Basin

There are already some dykes(38 Km) built and under construction (10 Km) along right side of Awash River starting about 25 Km downstream of the Tendaho dam. The dykes are well below Agricultural Village (AV2) and their purpose is to protect sugar cane plantation from natural flooding. The elevations of these dykes are still not enough to protect sugar cane command from the dam break flood since dam break WSEL is quite larger than natural flood level. However, there are no built dykes to protect the settlements (towns) within study domain from dam break flooding. Due to limitation of data at WWDSE office, dykes are not taken into consideration as to improve the cross-section data and inputting them to the model. However, the existence of the dykes, the flood wave after the dam failure still manages to penetrate further inland to the command. This is understandable given that the dykes have most probably been designed and placed to mitigate or prevent the threat of run-off flood.

Therefore, the existing dykes should be improved to cope up with the threat of a dam failure flood and/or further bank protection measures should be devised to be implemented in the event of dam failure. The event-time measures could be reinforcing the banks and the existing dykes by placing sand-bags or dumping sand, implementing the emergency action plan for evacuation. Pre-event measures could be improving the existing dykes and construction new dykes at critical locations along the river, constructing additional flood detention basins and regulators at suitable locations along the modeling reach, dredging emergency release channels to divert the flow to less-populated areas in case the WSEL exceeds a certain critical value etc. In this sense, a

diversion channel could be built upstream of Logia town, which is the most populated settlement in the modeling reach, so as to move the flood wave away from the town through flat plains towards Lake Abbe whilst accounting for the additional risks it can pose to the population nearby and the habitat in the vicinity of Abbe Lake (see fig 5-1). This approach would further require acquisition of some additional lands that the proposed diversion channel would overlap; hence would have some economic and social impacts on the affected landowners and their dependents even in the absence of dam failure and flooding and hence would require additional mitigate measures against socio-economic impacts such an option would impose. Therefore, that approach would necessitate a comprehensive cost-benefit analysis.

Furthermore, there is no existing flood detention basin to mitigate the flood, thereby decreasing the observed maximum WSELs and the peak discharges. Therefore, there is a need to build new flood detention basins along the reach provided that they withstand the impact of a dam failure induced flood.

Placement of rip-raps and spurs and dredging the streambed at critical locations along the modeling reach could further be considered as additional measures to regulate the flow after a detailed engineering survey as well as an assessment of the potential socio-economic and environmental impacts of such measures.

6.2.3 Observation Station U/s of Tendaho Dam

Certainly, there is further need for an observation station farther upstream of the dam to take real-time measurements of the incoming flow, which should simultaneously forward the measurements to an emergency response station at the dam for real-time forecasting of the probable flow conditions downstream of the dam and response of the WSEL in the reservoir, thereby providing insight for the issuance of evacuation warnings for specific settlements at the downstream and deciding on the width of the spillway gate openings, hence the magnitude of the discharge released through the gates. In such a manner, emergency response team might act based on pre-specified criteria in order to minimize potential losses and prevent the failure of the dam at the same time.

6.2.4 Flood Warning Communication Channels

The understanding of the flood warning by the residents of the settlements under risk of the essence in terms of reducing the number of possible fatalities due to a dam-failure induced flood. This particularly has crucial importance when the rural settlements, where the villagers are usually in fields (sometimes distant to the village centers) till late hours and the means of communication might somewhat be deficient, are considered. Hence, in addition to the broadcast warning messages through media, there would be further need for audio-visual warning means to ensure sound communication of the warning. In this regard, as a first action, the emergency action team might try to contact the village headmen to announce the warning in the village centers. As a second venue, the gendarmerie, security and military forces should immediately be contacted given that they can be deployed and mobilized on short notice. There would be the need for some sort of audio warning, such as a siren placed in the vicinity, in the settlements so as to ensure that the warning is received by all, in case the village headmen cannot be contacted.

6.2.5 Risk Assessment Study

The assessment of the risks imposed by a potential dam failure should be conducted through a risk assessment study geared towards identifying all social, economic and environmental risks and should further address the proposed measures, the agencies responsible for implementing the measures, stipulate for timeframes for implementation thereof, and identify the duties and responsibilities of each agency or party involved.

6.2.6 Training

The residents of the settlements that have been identified to be under flood risk by using the inundation map prepared in case the dam fails should be given prior training as to the said audio-visual warnings and their role in implementation of the emergency action plan so as to increase people's awareness in this regard.

6.3 Recommendation for Further Research

✚ Cross-sectional geometry measurements for the reaches are not easy to access. Cross-section measurements data at required River station were not available at dam owner

(MoWIE) and engineer (WWDSE) offices. To fill this data gap cross-section geometry were generated from low resolution DEM (30mx30m details) by using HEC-GeoRAS software. There would, however, be further need to improve the generated cross-sections by taking cross-section measurements on reach or by generating cross-sectional geometry from high resolution DEM for the study area. This is because cross-sectional data generated from low resolution DEM (30m) might not be of sufficient span-width or maximum elevation required for the analyses. The topographic maps should be produced by actual topographic surveying or generated from finer DEM if possible.

- ✚ Analyses by using other dam break analysis software like MIKE11 software, which might be used for comparison of outputs.
- ✚ Reaches with measured Manning roughness values and/or observed hydrographs at several gauging stations along the subject reach should be preferred for analyses. The observed hydrographs can be used to calibrate the subject reach in terms of Manning roughness values.

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APPENDIX

Appendix-A Measured hydrological data

Appendix-B Physical characteristics of Tendaho dam and reservoir

Appendix-C Standard graphic outputs of HEC-RAS model

Appendix-D Predicted number of fatalities at downstream region

Appendix-A Measured hydrological data

Table A-1 Maximum monthly flow data at Logia River hydrometric station

Year	Flow (m ³ /s)	Year	Flow (m ³ /s)
1988	30.206	1998	225.054
1989	63.524	1999	171.968
1990	40.471	2000	258.616
1991	616.93	2001	239.636
1992	159.094	2002	258.616
1993	125.369	2003	247.127
1994	151.951	2004	228.650
1995	181.756	2005	225.054
1996	258.616	2006	98.343
1997	157.059	2007	303.368

Table A-2 Water level and discharge data of Awash River at Tendaho hydrometric station

Year	Water Level (m)	Discharge (m ³ /s)	Year	Water Level (m)	Discharge (m ³ /s)	Year	Water Level (m)	Discharge (m ³ /s)
1965	3.63	368.10	1980	6.96	879.23	1995	5.44	633.54
1966	2.54	226.29	1981	6.96	879.23	1996	2.81	259.91
1967	3.94	411.14	1982	6.06	731.50	1997	6.00	721.88
1968	4.88	547.95	1983	4.92	553.97	1998	6.33	775.16
1969	4.48	488.63	1984	3.94	411.14	1999	6.31	771.90
1970	4.70	521.06	1985	6.78	849.19	2000	8.00	1,057.34
1971	4.34	468.25	1986	5.55	650.68	2001	6.48	799.66
1972	5.75	682.11	1987	7.89	1,038.15	2002	7.50	970.76
1973	6.94	875.88	1988	5.99	720.28	2003	7.16	912.89
1974	8.75	1,190.31	1989	5.46	636.65	2004	7.36	946.83
1975	6.95	877.55	1990	7.49	969.05	2005	7.09	901.07
1976	3.24	315.56	1991	5.71	675.80	2006	8.04	1,064.34

1977	7.48	967.34	1992	7.20	919.65	2007	6.74	842.54
1978	6.93	874.20	1993	7.60	987.94			
1979	5.99	720.28	1994	5.66	667.93			

Appendix-B Physical characteristics of Tendaho dam and reservoir

Table B-1: Technical specification of Tendaho dam

Tendaho dam		
1	Dam body	
1.1	Purpose	Irrigation
1.2	Type	Earth-fill
1.3	Height from toe	44m
1.4	Height from deepest foundation	53m
1.5	Top of wave wall	413.5m.a.m.s.l.
1.6	Crest elevation	412m.a.m.s.l.
1.7	Dam toe elevation	368m.a.m.s.l.
1.8	Crest length (exclusive of spillway length)	412m
1.9	Crest width	10m
1.10	U/s slope	3.5:1 m/m
1.11	D/s slope	2.5:1-2.57:1 m/m
2	Reservoir	
2.1	Maximum Flood Level (MWL)	409.3m.a.m.s.l.
2.2	Full Retention Level (FRL)	409m.a.m.s.l.
2.3	Minimum operating/Minimum Draw Down Level (MDDL)	396m.a.m.s.l.
2.4	Storage capacity at FRL (Live and Dead storage)	1.86x10 ⁹ m ³

2.5	Live/Effective storage at zero year	1.42x10 ⁹ m ³
2.6	Irrigation intake structure of irrigation outlet centerline level	391m.a.m.s.l.
2.7	General river bed level (OGL)	369.5m.a.m.s.l.
3	Spillway	
3.1	Type	Ogee, Radial gated
3.2	Total length (m)	226
3.3	Full PMF peak routed flow	2,441 m ³ /s
3.5	Full PMF peak inflow	6,084 m ³ /s
3.6	Crest elevation	400m.a.m.s.l.
3.7	Clear crest length (3 bays of 10.5m)	31.5m
3.8	Gate dimensions	10.5mx9m (3-gates)
3.9	Top level of gates	409m.a.m.s.l.
3.10	Bed level of approach channel	396m.a.m.s.l.
3.11	Type of energy dissipater	Hydraulic jump
4	Dam outlet (intake)	
4.1	Purpose	Irrigation
4.2	Outlet (center line) elevation	391m.a.m.s.l.
4.3	Invert level of tunnel outlet	388m.a.m.s.l.
4.4	Slide gate dimension/rectangular tunnel (Wxh)	4m (w) x 5m (h)
4.5	Designed Velocity of flow	4m/s
4.6	Capacity at FSL	78 m ³ /s
4.7	Gross irrigation command	60,000ha
5	Sugar Factory	

5.1	Capacity	26,000 TCD
5.2	Sugar cane production	7.02x10 ⁶ tones/year
5.3	Maximum period of operation	270 days per year

Table B-2 Tendaho dam reservoir elevation storage-area data

Elevation (m)	Area (Km ²)	Volume (MMC)	Elevation (m)	Area (Km ²)	Volume (MMC)
374	0.001	0.000	394	55.263	237.302
375	0.026	0.014	395	64.472	297.169
376	0.102	0.078	396	73.705	366.257
377	0.220	0.238	397	81.100	443.660
378	0.244	0.470	398	86.969	527.695
379	0.388	0.786	399	92.590	617.475
380	0.799	1.380	400	97.868	712.704
381	1.673	2.617	401	106.375	814.826
382	2.684	4.795	402	112.357	924.192
383	3.131	7.702	403	118.592	1039.667
384	6.834	12.685	404	124.768	1161.346
385	8.918	20.561	405	130.950	1289.206
386	12.718	31.378	406	133.725	1421.543
387	15.111	45.293	407	144.291	1560.552
388	17.649	61.673	408	149.860	1707.627
389	20.160	80.577	409	154.567	1859.841
390	22.014	101.664	410	160.598	2017.423
391	23.713	124.528	411	166.060	2180.752
392	25.362	149.066	412	174.503	2351.033
393	47.924	185.709	413	185.889	2710.472

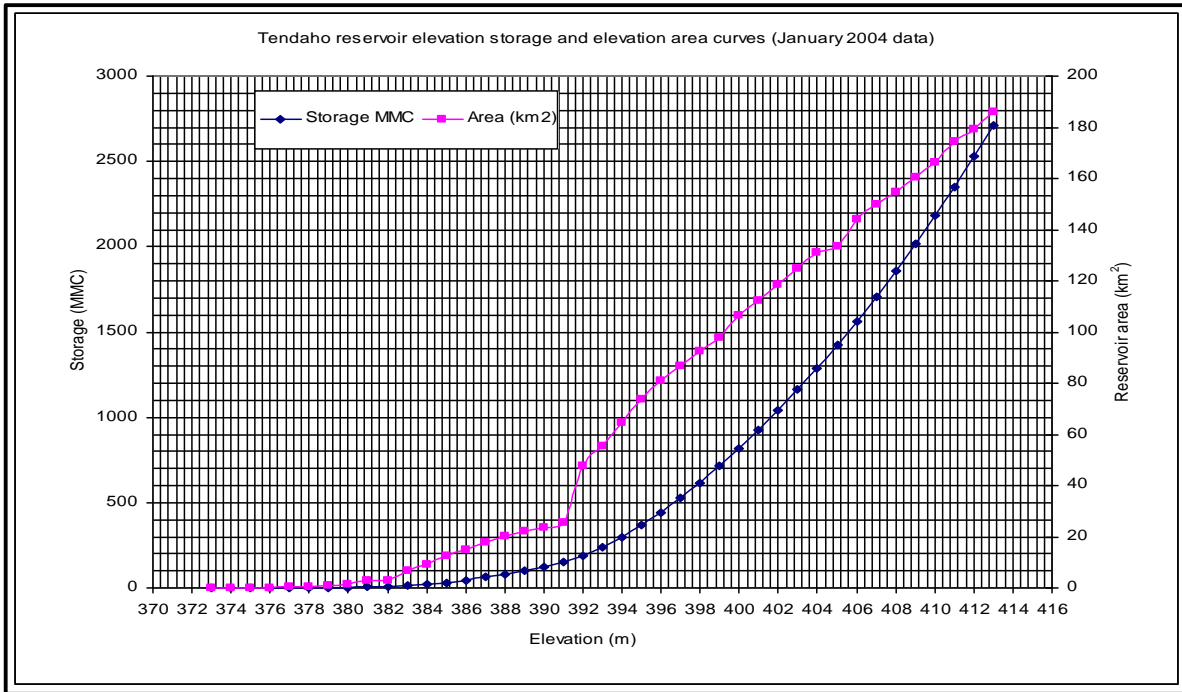


Figure B-1 Tendaho dam reservoir elevation storage-area curves

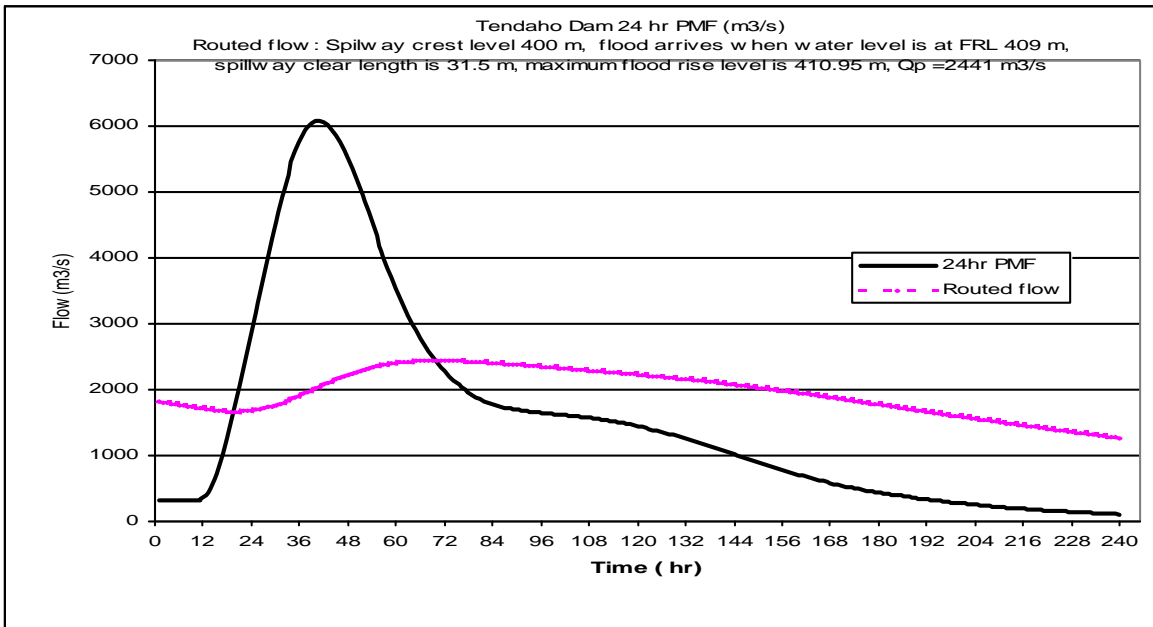


Figure B-2 Full PMF catastrophic inflows into the Tendaho dam reservoir and outflows over the 31.5m long clear span spillway

Appendix-C Standard Graphic Outputs of HEC-RAS Model

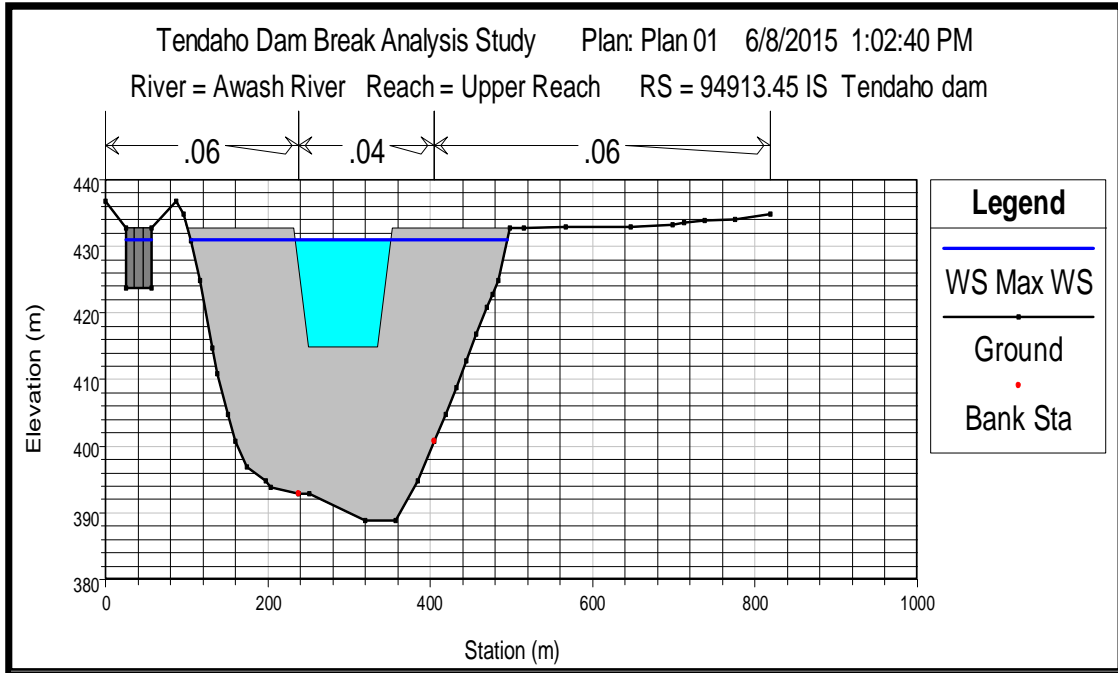


Figure C-1 Tendaho dam overtopping failure (breached) for Scenario W4

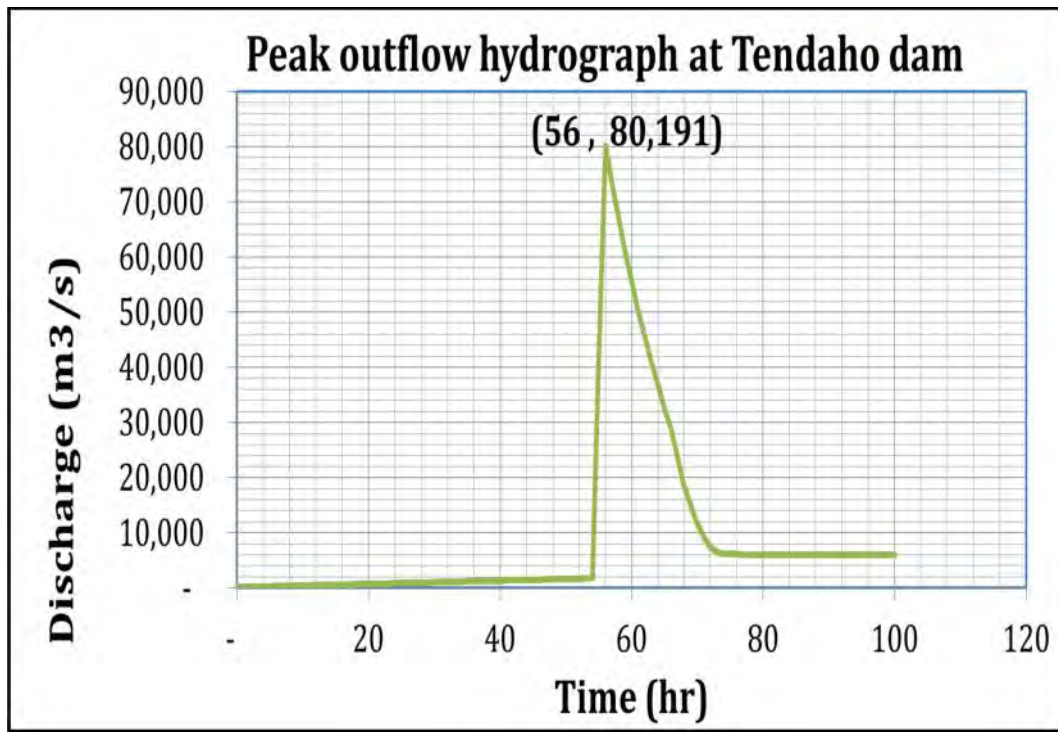


Figure C-2 Breached peak outflow hydrograph at Tendaho dam for scenario W4

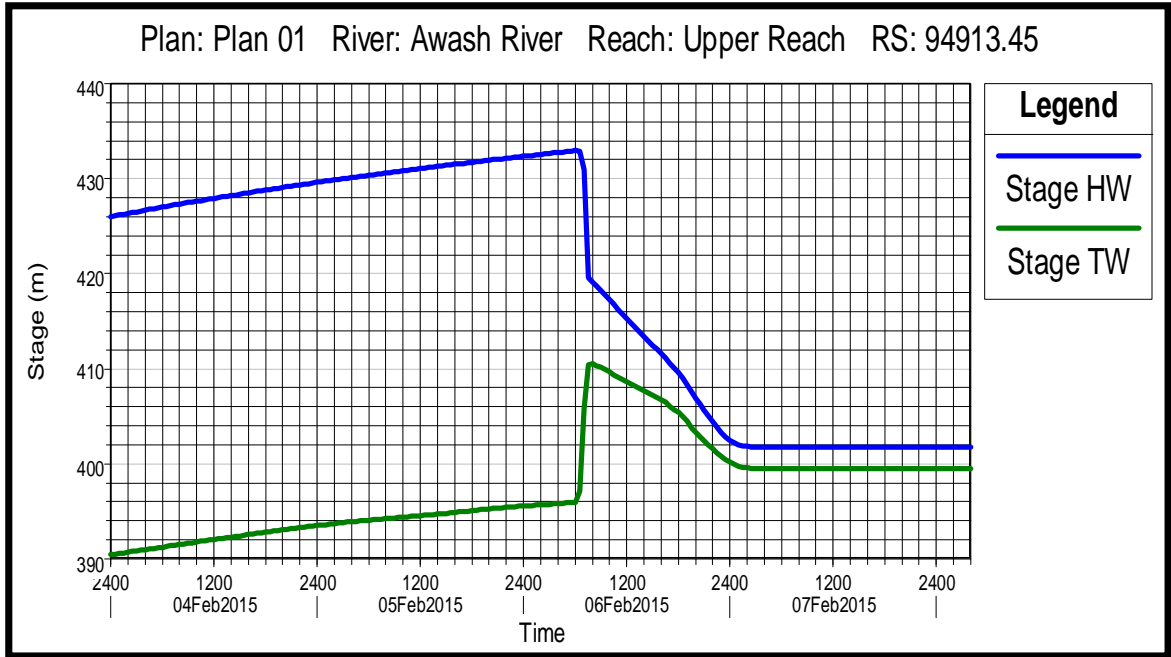


Figure C-3 WSEL hydrograph at Tendaho dam for Scenario W4

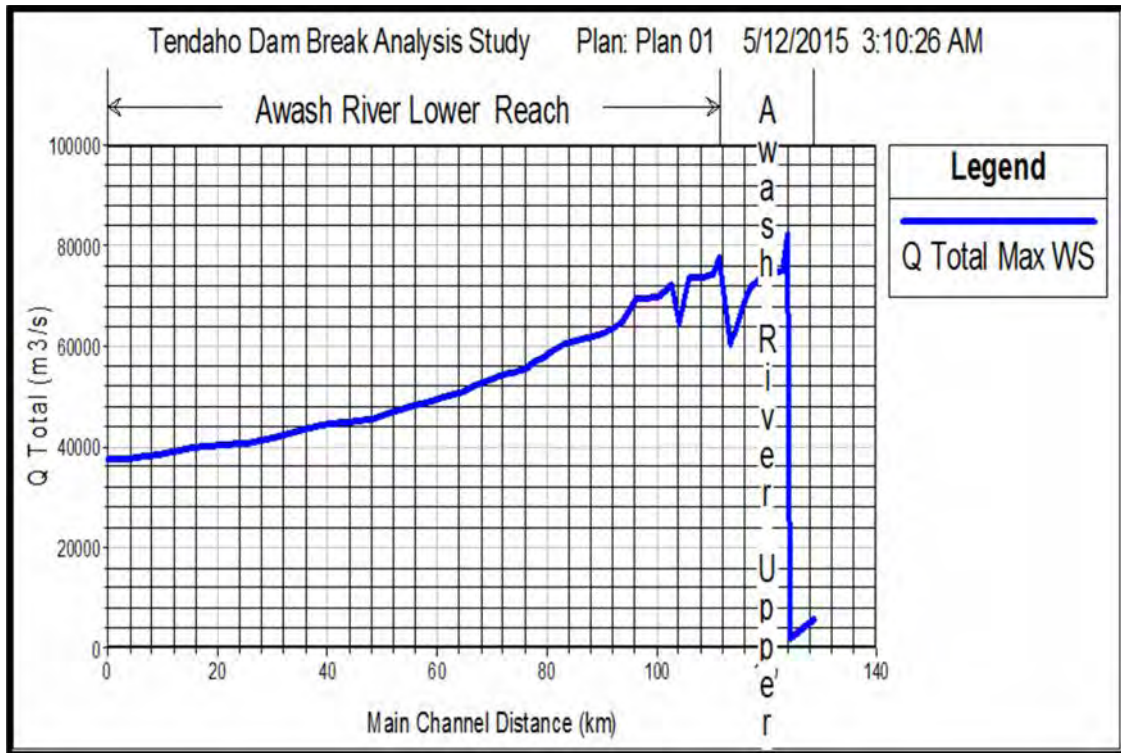


Figure C-4 Spatial variation of discharge profile for the entire reach for scenario W4

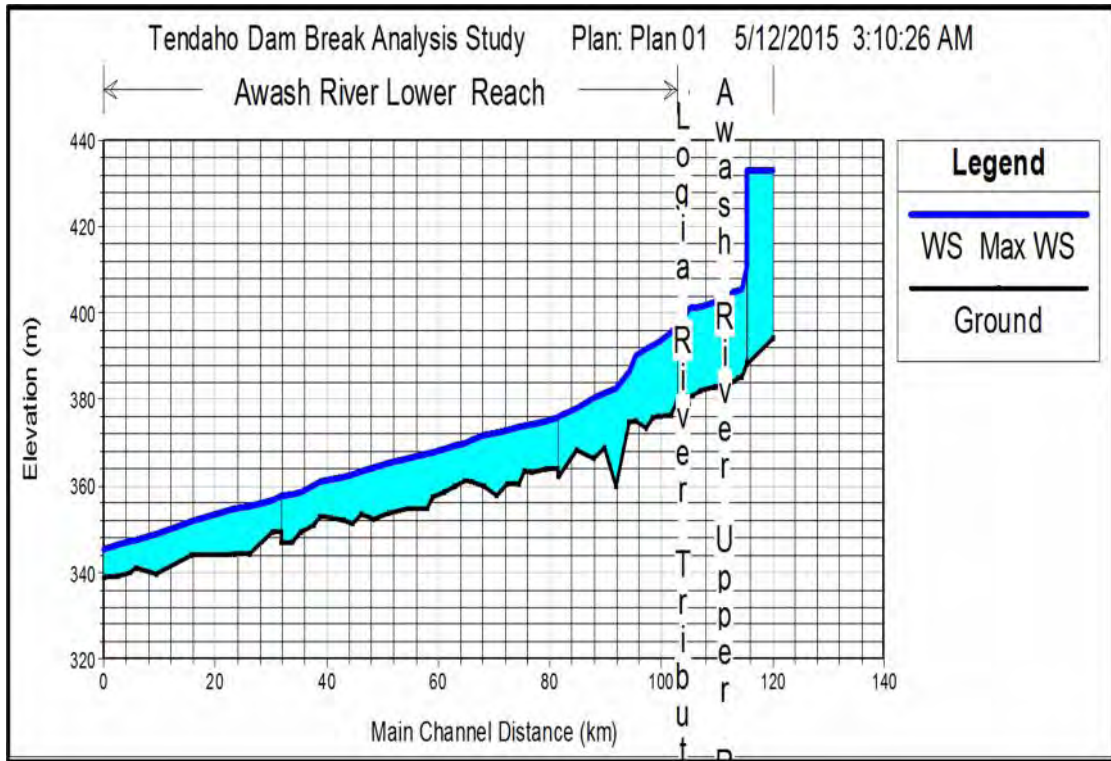


Figure C-5 Spatial variation of max.WS profile for the entire reach for scenario W4

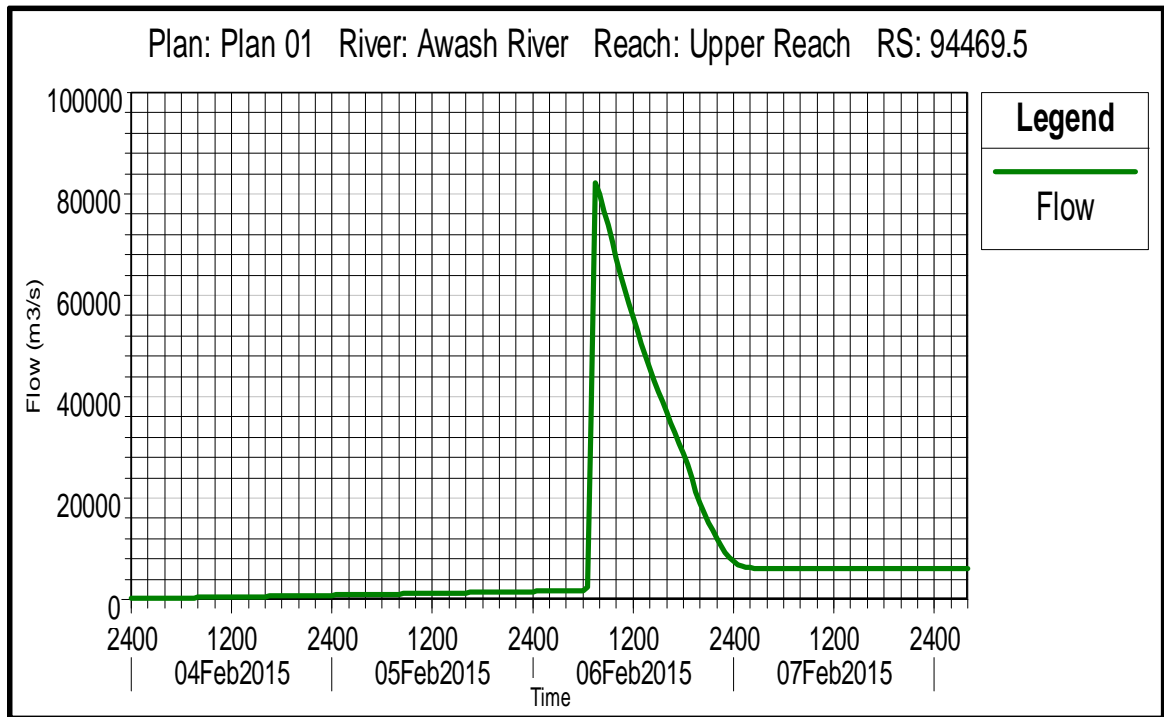


Figure C-6 Discharge hydrograph at RS-94469.5 for Scenario W4

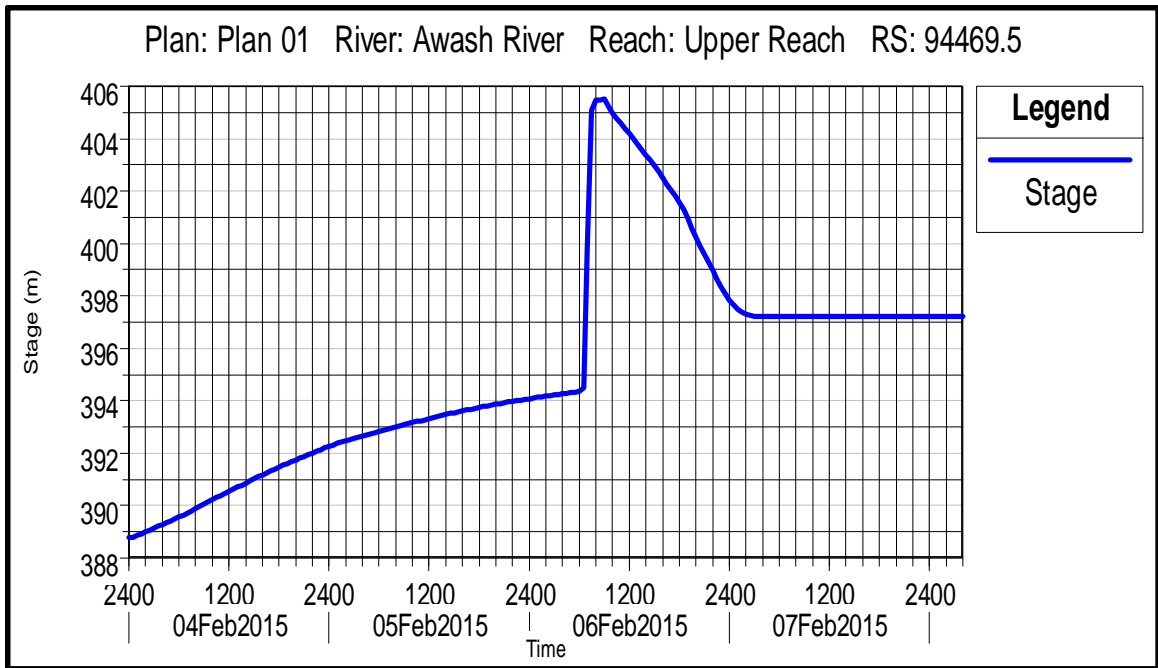


Figure C-7 WSEL hydrograph at RS-94469.5 for Scenario W4

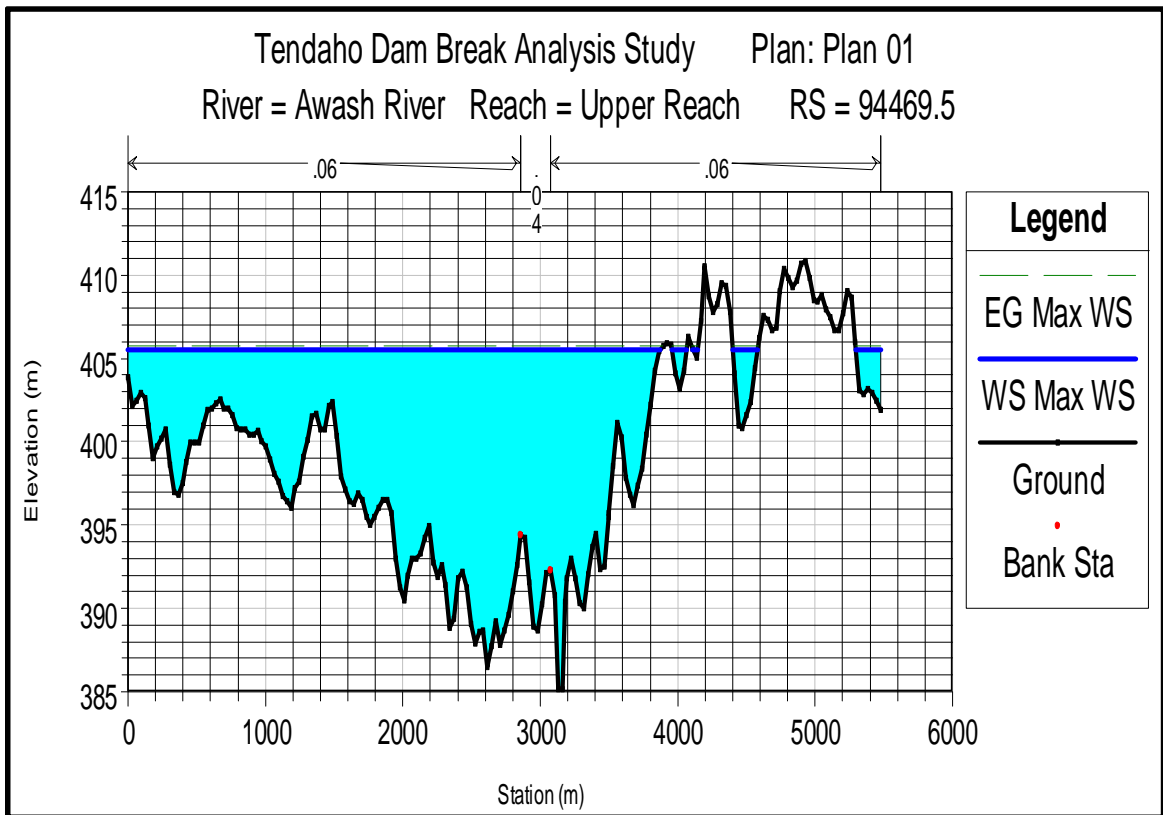


Figure C-8 Max WS, water depths and widths at RS-94469.5 for Scenario W4

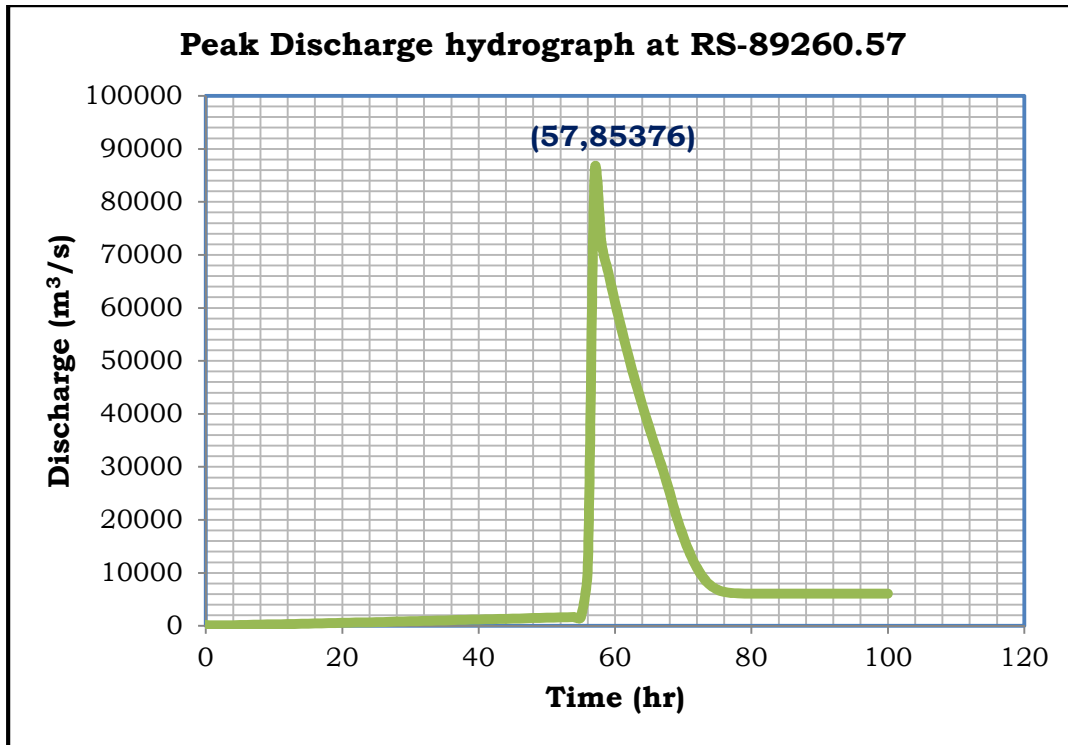


Figure C-9 Peak discharge hydrograph at Logia town for Scenario W4

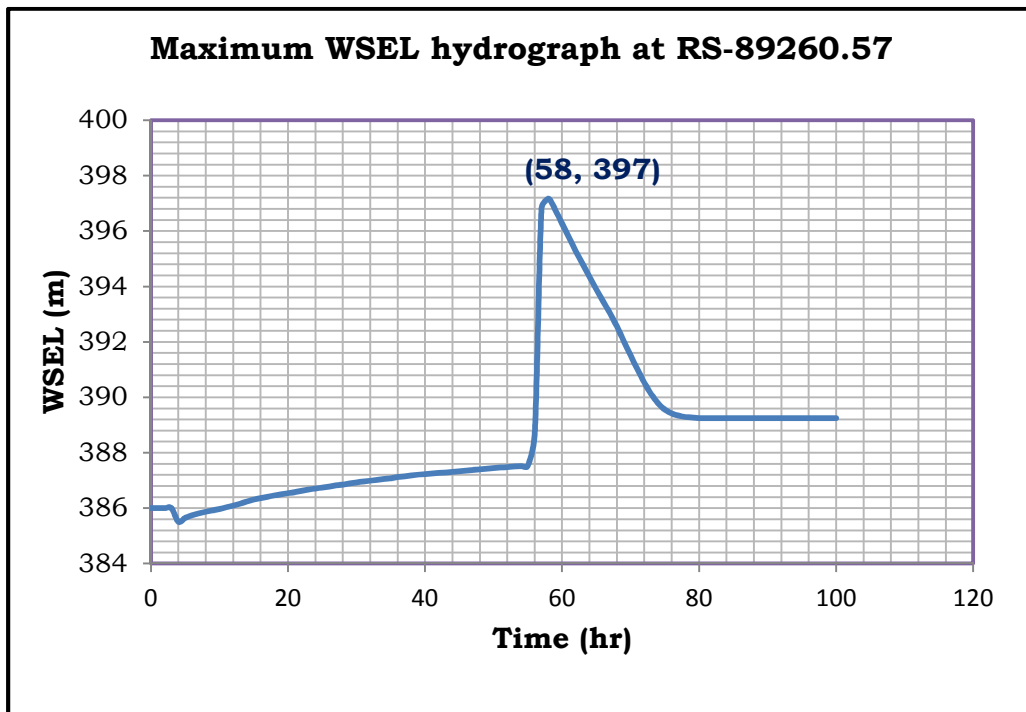


Figure C-10 Maximum WSEL hydrograph at Logia town for Scenario W4

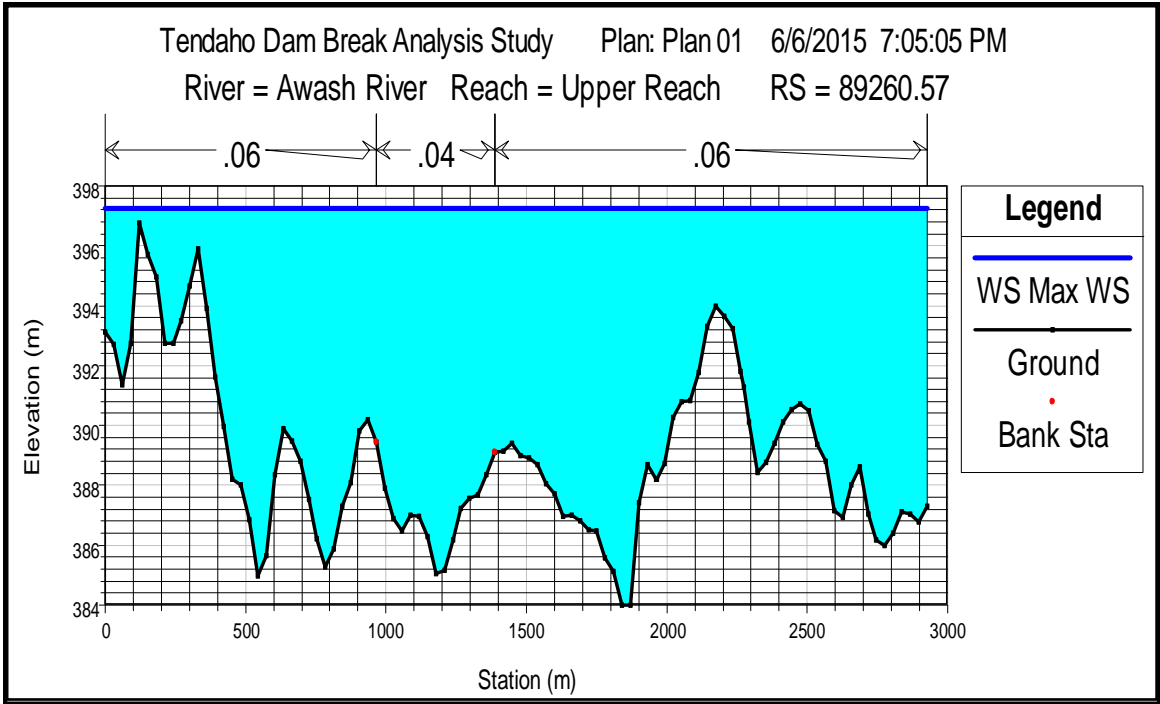


Figure C-11 Max WS, water depths and widths at Logia town for Scenario W4

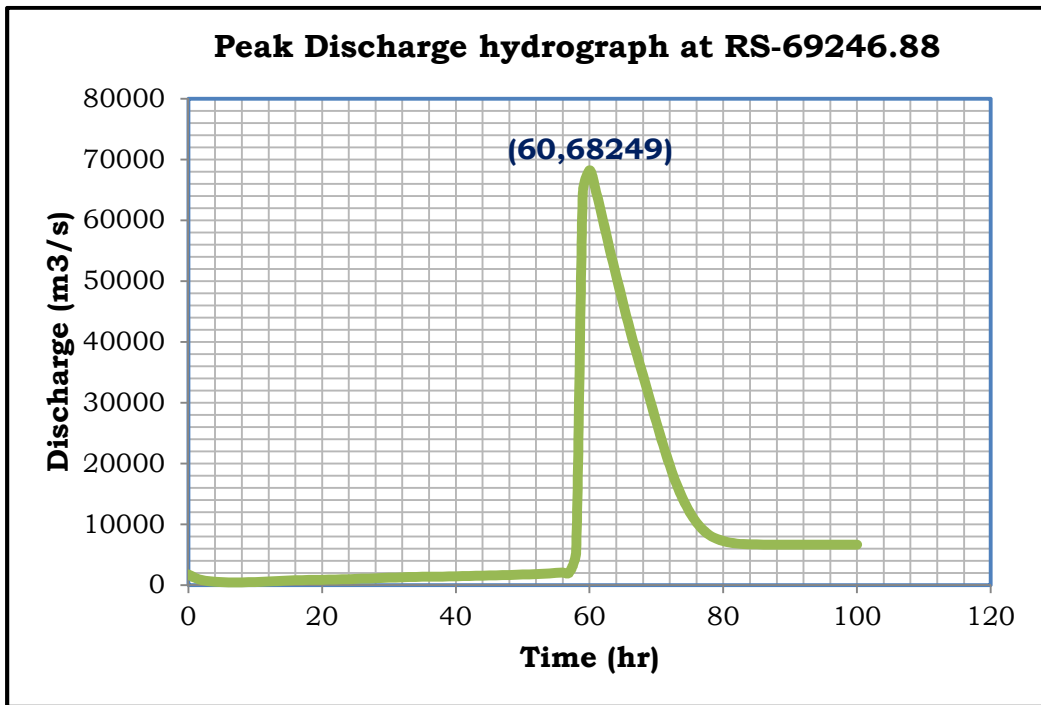


Figure C-12 Peak discharge hydrograph at Dubti town for Scenario W4

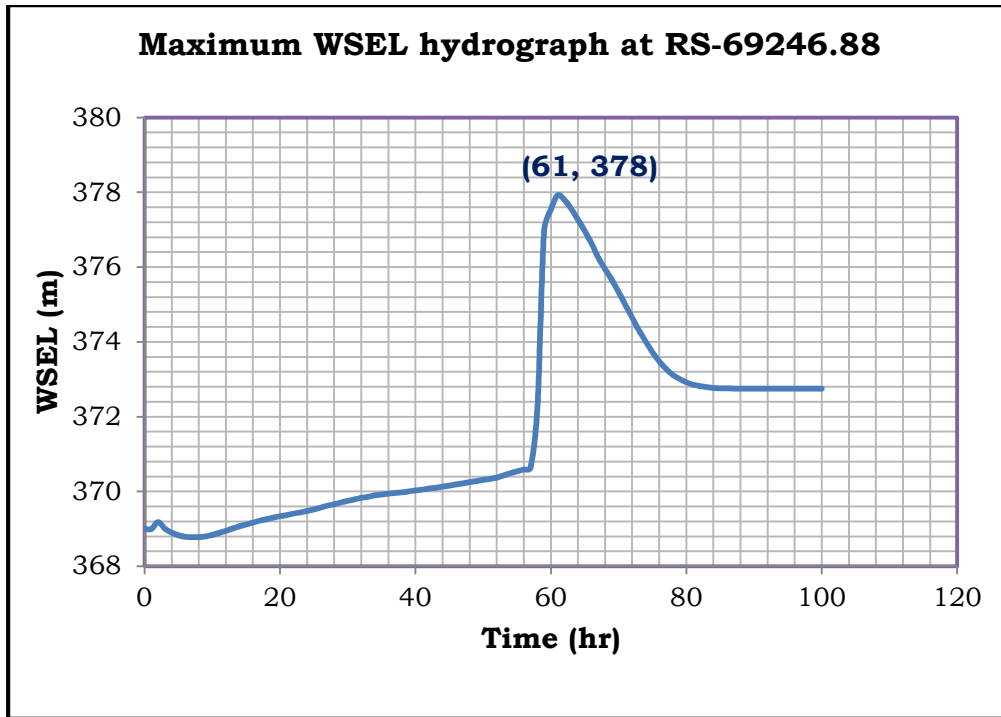


Figure C-13 Maximum WSEL hydrograph at Dubti town for Scenario W4

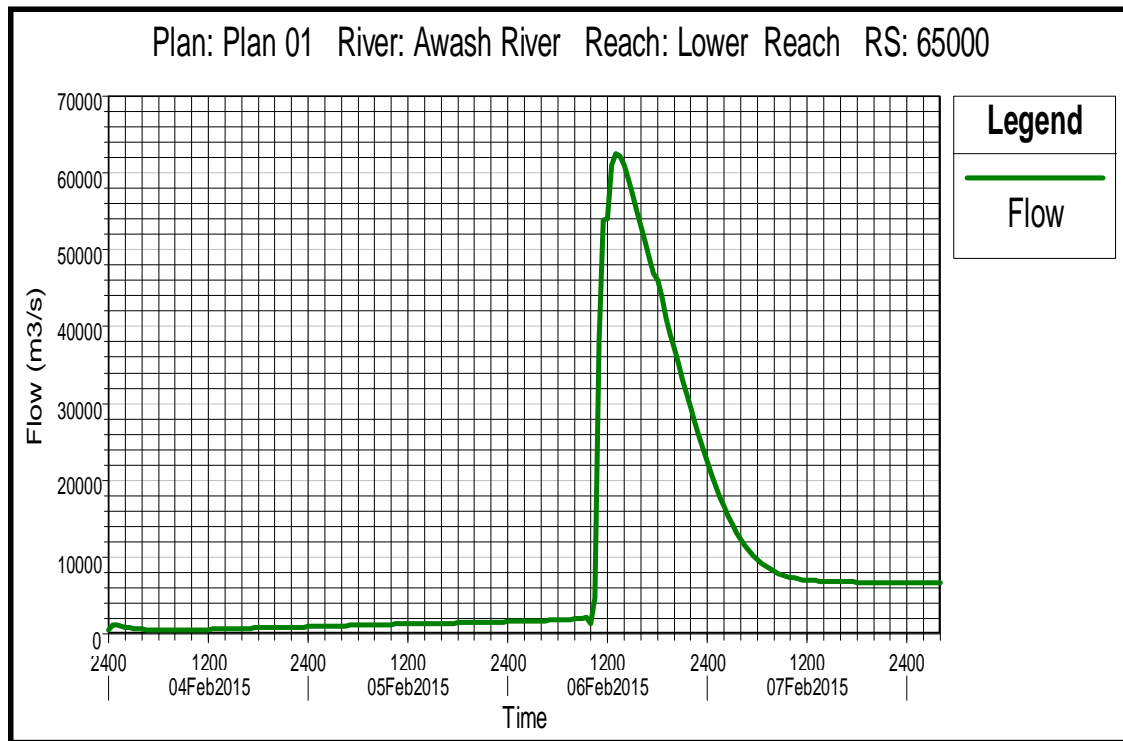


Figure C-14 Peak discharge hydrograph at Dubti Bridge for Scenario W4

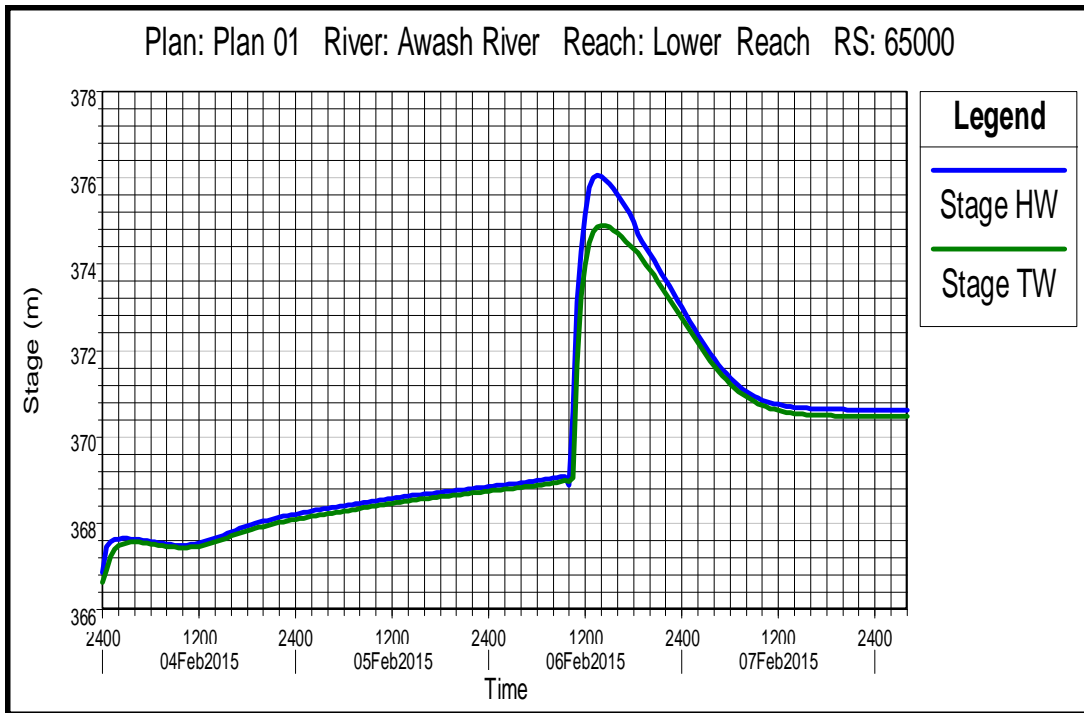


Figure C-15 Maximum WSEL hydrograph at Dubti Bridge for Scenario W4

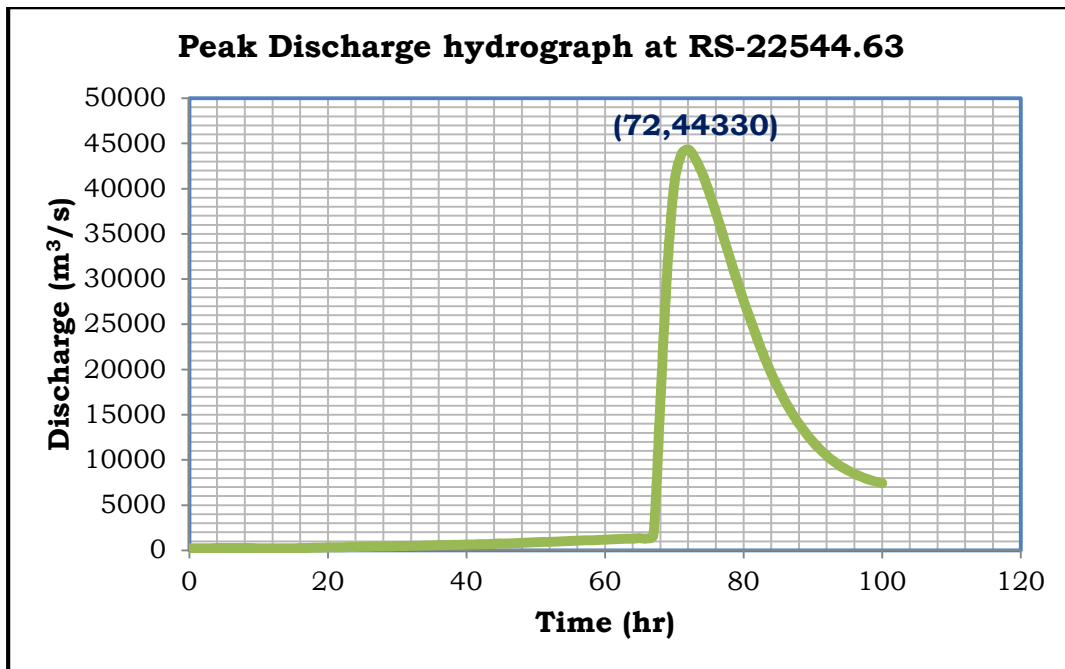


Figure C-16 Peak discharge hydrograph at Assaita town (Sugar Factory) for Scenario W4

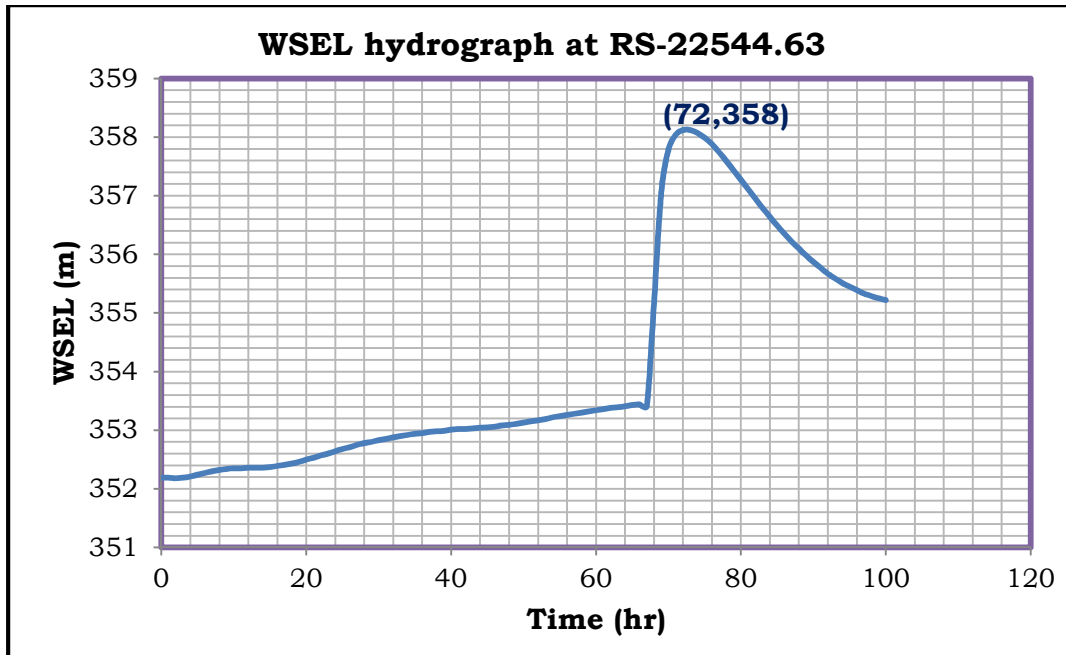


Figure C-17 Maximum WSEL hydrograph at Assaita town (Sugar Factory) for Scenario W4

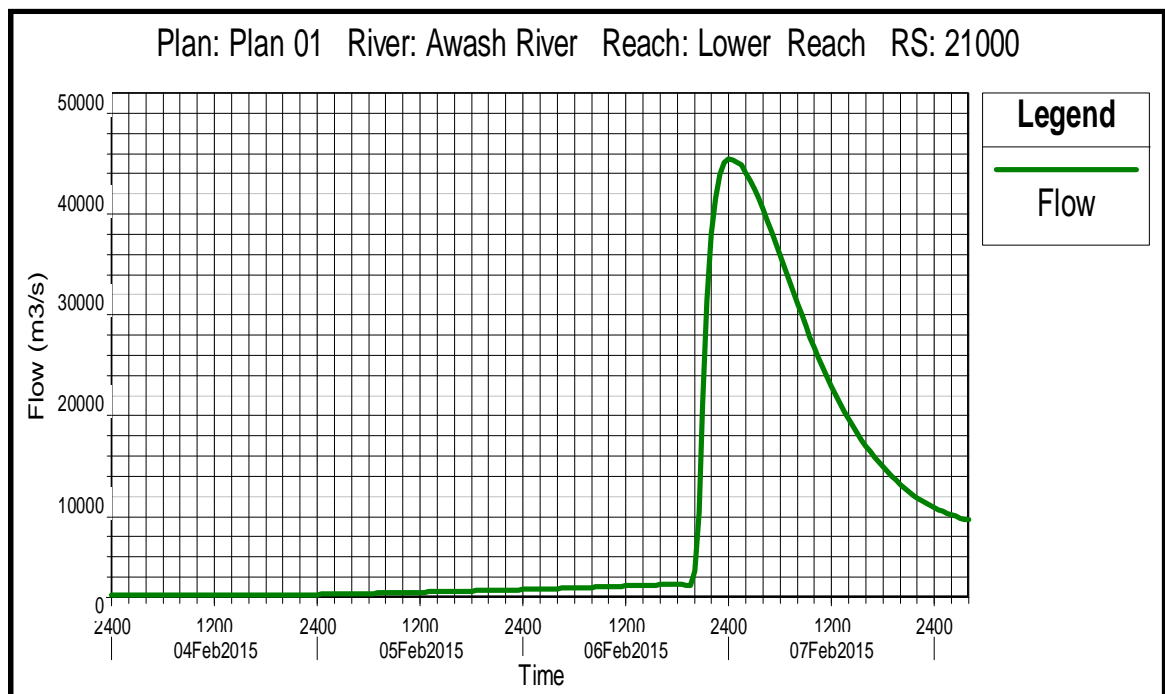


Figure C-18 Peak discharge hydrograph at Assaita Bridge for Scenario W4

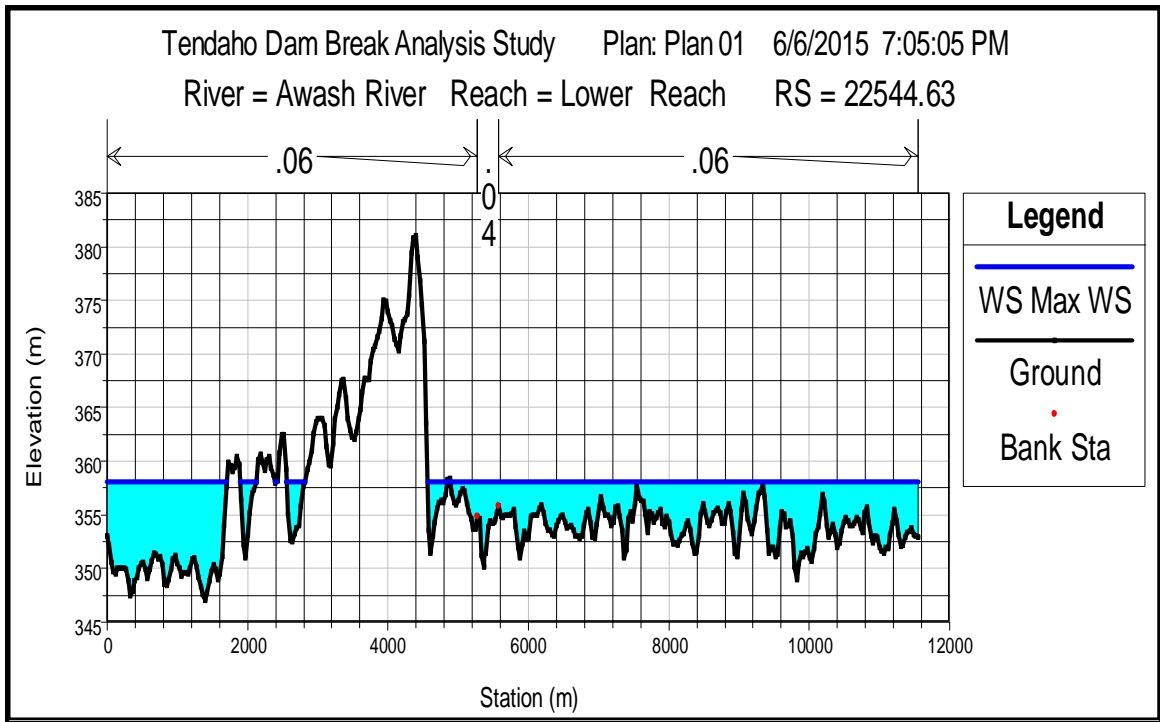


Figure C-19 Max WS, water depths and widths at Assaita town (Sugar Factory) for Scenario W4

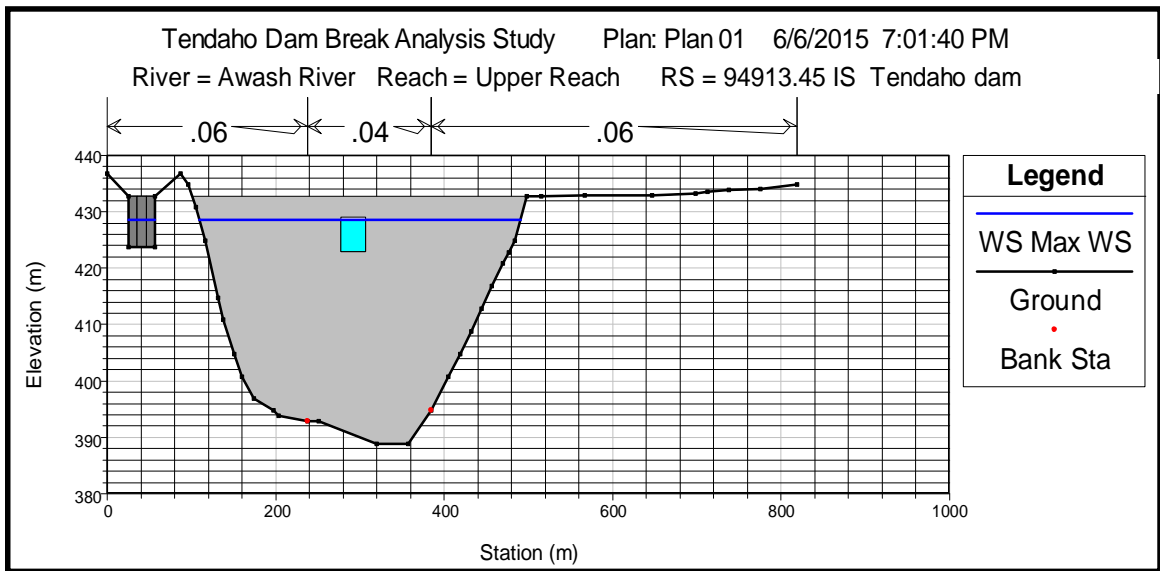


Figure C-20 Tendaho dam piping failure simulation in Scenario W4P

Appendix-D Predicted number of fatalities at downstream region

			Case-1 Precise understanding of warning			Case-2 Vague understanding of warning			Case-3 Warning issued 2 hours after failure of dam & precise understanding of warning			Case-4 Warning issued 2 hours after failure of dam & vague understanding of warning	
Settlement	Reach (u/s-d/s cross-section)	Population	Warning time available (hr)	Suggested Fatality rate	of Fatalities (Rounded up)	Warning time available (hr)	Suggested Fatality rate	of Fatalities (Rounded up)	Warning time available (hr)	Suggested Fatality rate	of Fatalities (Rounded up)	Suggested Fatality rate	of Fatalities (Rounded up)
Logia Town	14-15	36,505.3	3.0	0.01	366	3.0	0.03	1,096	1.0	0.02	731	0.04	1,461
Sub-HQ (AV-2)	21-22	1,750.0	4.0	0.01	18	4.0	0.03	53	2.0	0.01	18	0.03	53
Secondary School	21-22	1,540.0	4.0	0.01	16	4.0	0.03	47	2.0	0.01	16	0.03	47
Health Center	21-22	30.0	4.0	0.01	1	4.0	0.03	1	2.0	0.01	1	0.03	1
AV-1	22-23	1,206.0	4.5	0.01	13	4.5	0.03	37	2.5	0.01	13	0.03	37
Elementary School	22-23	520.0	4.5	0.01	6	4.5	0.03	16	2.5	0.01	6	0.03	16
Health post	22-23	10.0	4.5	0.01	1	4.5	0.03	1	2.5	0.01	1	0.03	1
Dubti Town	26-27	18,252.7	6.3	0.01	183	6.3	0.03	548	4.3	0.01	183	0.03	548
AV-3	27-28	1,206.0	7.0	0.01	13	7.0	0.03	37	5.0	0.01	13	0.03	37
Elementary School	27-28	520.0	7.0	0.01	6	7.0	0.03	16	5.0	0.01	6	0.03	16
Health post	27-28	10.0	7.0	0.01	1	7.0	0.03	1	5.0	0.01	1	0.03	1
AV-4	28-29	1,105.0	7.3	0.01	12	7.3	0.03	34	5.3	0.01	12	0.03	34
Elementary School	28-29	520.0	7.3	0.01	6	7.3	0.03	16	5.3	0.01	6	0.03	16
Health post	28-29	10.0	7.3	0.01	1	7.3	0.03	1	5.3	0.01	1	0.03	1
Sub-HQ (AV-5)	32-33	1,750.0	8.8	0.01	18	8.8	0.03	53	6.8	0.01	18	0.03	53
Secondary School	32-33	1,540.0	8.8	0.01	16	8.8	0.03	47	6.8	0.01	16	0.03	47
Health Center	32-33	30.0	8.8	0.01	1	8.8	0.03	1	6.8	0.01	1	0.03	1
Sub-HQ (AV-6)	33-34	1,750.0	9.5	0.0002	1	9.5	0.0003	1	7.5	0.0002	1	0.0003	1
Secondary School	33-34	1,540.0	9.5	0.0002	1	9.5	0.0003	1	7.5	0.0002	1	0.0003	1

Health Center	33-34	30.0	9.5	0.0002	0	9.5	0.0003	0	7.5	0.0002	0	0.0003	0
AV-7	35-36	1,105.0	10.3	0.0002	1	10.3	0.0003	1	8.3	0.0002	1	0.0003	1
Elementary School	35-36	520.0	10.3	0.0002	1	10.3	0.0003	1	8.3	0.0002	1	0.0003	1
Health post	35-36	10.0	10.3	0.0002	0	10.3	0.0003	0	8.3	0.0002	0	0.0003	0
AV-8	38-39	1,105.0	11.8	0.0002	1	11.8	0.0003	1	9.8	0.0002	1	0.0003	1
Elementary School	38-39	520.0	11.8	0.0002	1	11.8	0.0003	1	9.8	0.0002	1	0.0003	1
Health post	38-39	10.0	11.8	0.0002	0	11.8	0.0003	0	9.8	0.0002	0	0.0003	0
AV-9	40-41	1,206.0	12.5	0.0002	1	12.5	0.0003	1	10.5	0.0002	1	0.0003	1
Elementary School	40-41	520.0	12.5	0.0002	1	12.5	0.0003	1	10.5	0.0002	1	0.0003	1
Health post	40-41	10.0	12.5	0.0002	0	12.5	0.0003	0	10.5	0.0002	0	0.0003	0
Assaita Town	51-52	26,666.0	18.0	0.01	267	18.0	0.03	800	16.0	0.01	267	0.03	800
Sugar Factory	51-52	3,264.0	18.0	0.01	33	18.0	0.03	98	16.0	0.01	33	0.03	98
Health post	51-52	10.0	18.0	0.01	1	18.0	0.03	1	16.0	0.01	1	0.03	1
Secondary School	51-52	1,540.0	18.0	0.01	16	18.0	0.03	47	16.0	0.01	16	0.03	47
Health Center	51-52	30.0	18.0	0.01	1	18.0	0.03	1	16.0	0.01	1	0.03	1
Hospital	51-52	120.0	18.0	0.01	1	18.0	0.03	4	16.0	0.01	1	0.03	4
Police station	51-52	50.0	18.0	0.01	1	18.0	0.03	2	16.0	0.01	1	0.03	2
Clinic	51-52	20.0	18.0	0.01	1	18.0	0.03	1	16.0	0.01	1	0.03	1
Afambo Town	53-54	1,371.0	19.3	0.0002	1	19.3	0.0003	1	17.3	0.0002	1	0.0003	1
AV-15	55-56	1,206.0	21.8	0.0002	1	21.8	0.0003	1	19.8	0.0002	1	0.0003	1
Elementary School	55-56	520.0	21.8	0.0002	1	21.8	0.0003	1	19.8	0.0002	1	0.0003	1
Health post	55-56	10.0	21.8	0.0002	0	21.8	0.0003	0	19.8	0.0002	0	0.0003	0
AV-16	56-57	1,206.0	23.3	0.0002	1	23.3	0.0003	1	21.3	0.0002	1	0.0003	1
Elementary School	56-57	520.0	23.3	0.0002	1	23.3	0.0003	1	21.3	0.0002	1	0.0003	1
Health post	56-57	10.0	23.3	0.0002	0	23.3	0.0003	0	21.3	0.0002	0	0.0003	0
Total number of fatalities					1,012			2,972			1,377		3,337