Dam Breach Modelling and Downstream Risk Analysis

(For Arjo-Dedessa Dam)

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December, 2015

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School of Graduate Studies

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Dam Breach Modelling and Downstream Risk Analysis Using

(For Arjo-Dedessa Dam)

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This is to certify that the thesis prepared by Hayimanot Lejissa, entitled: Dam Breach Modeling and Downstream Risk Analysis for Arjo-Dedessa Dam and submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering under Hydraulic Engineering compiles with the regulations of the university and meets the accepted standards with respect to originality and quality

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Dr. Ing. Asie Kemal (Advisor)

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Date

This thesis is dedicated to my brothers for their endless love, encouragement and support, and for always being near me.

ABSTRACT

Analysis and simulation of embankment dam breach events and the resulting floods are critical to differentiating and reducing threats due to potential dam failures. Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at downstream critical locations.

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) can be to develop a dam failure model. HEC-GeoRAS was used to extract geometric information from a Digital Elevation Model (DEM) and then imported into HEC-RAS 4.1 where one dimensional Unsteady-flow simulation of the dam breach performed. The simulation results were mapped using the GIS extension tool HEC-GeoRAS on ArcMap. Inundation mapping of water surface profile results from dam failure models provides a level of the flood hazard and provides insight for emergency action plan.

The process for gathering and preparing data, estimating breach parameters, creating an unsteady-flow model in HEC-RAS, entry of dam breach parameters, performing a dam failure analysis for two dam failure scenarios, mapping of the flood inundation and emergency action plan are discussed in this paper.

Key Words: Dam Breach, Modelling, DEM, HEC-GeoRAS, HEC-RAS, Hydrograph, Inundation

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LISTS OF ACRONMYS

DAMBRK:	Dam Break
DEM:	Digital Elevation Model
DSO:	Dam Safety Office
EAP:	Emergency Action Plan
FEMA:	Federal Emergency Management Agency
FERC:	Federal Energy Regulatory Commission
FRL:	Full Reservoir Level
GIS:	Geographical Information System
HEC-HMS:	Hydrologic Engineering Centre Hydrologic Modelling System
HEC-RAS:	Hydrologic Engineering Centre River Analysis System
IDF:	Inflow Design Flood
MWIE:	Ministry of Water, Irrigation and Energy
MWL:	Maximum Water Level
NBCBN-RE:	Nile Basin Capacity Building Network for River Engineering
NMA:	National Meteorological Agency
NWS:	National Weather Service
OWWCE:	Oromia Water Works Construction Enterprise
OWWDSE:	Oromia Water Works Design and Supervision Enterprise
PMF:	Probable Maximum Flood
SDF:	Spillway Design Flood
TIN:	Triangulated Irregular Network
USACE:	United States Army Corps of Engineers
USBR:	United States Bureau of Reclamation

LIST OF VARIABLES

A_s	Surface area of the reservoir
B_{avg}	Average Breach Width
BFF	Breach Formation Factor ($H_w * V_{out}$)
C	Width of the dam crest in meter
C_b	Coefficient, which is a function of reservoir size
g	Acceleration due to gravity, which equals 9.81meter/sec2
H_b	Height of Breach in Meter
H_w	Maximum depth of water stored behind the breach
K_o	Failure mode factor
V_w	Reservoir Volume Stored Behind the dam
Q_p	Dam Breach Peak Discharge
T_f	Breach development time
V_{out}	Breach Outflow (m^3)
V_{er}	Volume of dam eroded in cubic meters during a breach.
W_b	Breach bottom width
Z_b	Side slopes of breach (Z_b Horizontal: 1 Vertical)
Z_d	Slopes of downstream face of the embankment (*Zd* Horizontal: 1 Vertical).
Z_u	Slope of the upstream face of the embankment (*Zu* Horizontal: 1 Vertical).
Z_3	Sum of the upstream and downstream embankment slopes, Zu + Zd
γ	Instantaneous flow reduction factor = $\frac{23.4A_s}{B_{avg}}$

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1. INTRODUCTION

1.1. Background

Dams are hydraulic structures used to store, control, divert water impounding it behind the upstream side of dam in a reservoir for different purposes, like hydropower generation, water supply, irrigation, navigation and transportation, etc. Although dams have many advantages, the risk that may happen due to the failure still exists. Dams can have a risk to downstream communities and properties if not designed, operated and maintained properly. Dams are classified as concrete dams or earthen dams depending on construction materials, large and small dams depending on water storage volume behind the dam and height of the dam.

All dams, regardless of their design or construction, have increased forces applied to them during extreme events which increase the potential risk of failure (Ahmad Asnaashari, 2014). Therefore, a dam breach analysis is usually conducted to determine the ultimate discharge from a hypothetical breach of a dam under such events. The outcome is a breach hydrograph from dam failure with a flood wave immediately downstream of the dam, which is routed throughout the river system to determine the flood arrival time, peak flow, and the depth of flow at downstream locations.

Mapping of inundation areas (i.e. areas flooded by the flood wave) is used for: estimating the potential consequences of a dam breach; confirming the classification; and emergency planning purposes.

Generally, modeling of a potential embankment-breaching failure involves solving two problems, i.e., determining the outflow from the breach and routing the computed flood wave through the downstream flood plain (Chaiyuth Chinnarasri, 2007)

This paper discusses the dam failure analysis of a medium earth and rock-fill dam. As a case study, Arjo Dedessa Rock-fill Dam which is located about 360 km from Addis Ababa and is accessible via Bedele, is proposed. Dam site is located 1.5 km upstream from the confluence of Dedessa and Wama Rivers near Arjo in Jima zone. (O.W.W.D.S.E, 2013)

This dam can be classified as high hazard structure since the size of number of population is significant and property found at downstream of the dam is very large. Due to this classification, modeling the dam breach and analyzing the effect of dam breach outflow hydrograph on downstream floodplain for this dam is necessary.

1.2. Statement of the Problem

A well-designed, constructed, and operated dam can reduce flood risk in areas downstream by temporarily impounding flood waters and attenuating the observed peak flood flows in exposed low lying areas, even if the dam is not specifically designed for flood mitigation. However, impounding water behind a dam also creates risk to downstream areas because of the potential for uncontrolled release of the reservoir pool caused by dam failure which could result in a peak flow discharge that greatly exceeds any possible natural flood event. There are several potential causes of dam failure including hydrologic, hydraulic, geologic, seismic, mechanical, and operational (FEMA, 2013).

One of the most common causes of dam failures is the inability to safely pass flood flows. Failures caused by hydrologic conditions can range from sudden failure, with complete breaching or collapse of the dam, to gradual failure, with progressive erosion and partial breaching (FEMA, 2013)

Dam failures may occur due to a variety of causes such as significant hydrologic and non-hydrologic events. If a dam breach occurs, an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area and affect the population and damages the economic resources. Nowadays, there are many embankment dams existing, under construction and planned to be constructed in Ethiopia. Therefore, in addition to proper design and construction method, the dam failure and risk analysis must be undertaken for the sake of safety.
Arjo- Dedessa Irrigation Project Dam, a rock-fill embankment dam consists of impervious clay core is now under construction. This dam is proposed to impound the water within the reservoir of capacity 1924.6 million meter cube at its full pool level. The main purpose if this dam is to retain, control and release this water for downstream huge irrigation fields without the disturbance of the downstream natural river flow system. Unlike its benefits, its safety also should be considered for the consequence in case the failure may exist during and after the construction time. At the downstream of Arjo-Dedessa Dam, there is a huge irrigation field of 88,000 hectares along

Dedessa River in the left and right side at a distance of about 40 km downstream of the dam. Additionally, two bridges which are located at 40 km and 90 km downstream on Nekemte Bedele and Nekemt Gimbi asphalted roads. A small town called Arjo-Gudatu and Dedessa Lodge are also located at about 90 km downstream. Even if the spillway of the proposed dam was designed to pass the inflow design flood, the dam breach modelling and downstream risk analysis for the worst hydrologic and non-hydrologic are important for the safety of the dam and to prevent the life loss and property damages may happen in case the failure may occur.

Thus, for Arjo-Dedessa rock-fill dam, two dam breach scenarios were proposed to analyse the effect of the outflow breach hydrographs on the downstream floodplains. These scenarios are hydrologic (overtopping) and non-hydrologic (Sunny Day) or Piping failure.

The problem that needs to be solved in this study paper is the proper modeling of Arjo-Dedessa Dam breach and the effect of the breach outflow flood on the downstream flood plain. Therefore, for the proposed dam breach modeling, prediction of breach outflow hydrograph and computer simulation to evaluate the dam failure and its impact to downstream area, which is the first phase of dam breach analysis, is discussed for dam failure scenarios.

1.3. Objective

The main objective of this study is to model the Arjo-Dedessa Rock-fill Embankment Dam breach and analyze the risk level (consequence) due to the dam breach/failure on the downstream floodplain.

The specific objectives are:

- To predict dam breach parameters using three breach parameter estimation methods
- To compare and discuss the results
- To select the critical one for Arjo-Dedessa rock-fill dam breach modeling case
- To calculate peak outflows for overtopping and piping failure approaches
- To route the model outflow hydrographs through the downstream floodplain.
- To prepare floodplain inundation maps and
- To prepare an emergency action plan (EAP) to give a response in order to prevent or reduce the damages that may occur on downstream due to the dam failure.

1.4. Organization of the Thesis

This thesis is organized in seven chapters. Chapter one deals with the general introduction (i.e. background, statement of the problem and objectives of the study. Chapter two discusses the literature review on embankment dam breach modeling methods, breach characteristics and parameter estimation, and hydraulic modeling for the analysis of downstream routing the outflow hydrograph from dam breach, different researcher's literatures on dam breach modeling and outflow hydrograph routing, mapping flood inundation area, etc. . Chapter three discusses the methodology and steps to be taken in this paper in order to process the study well. In this section, study area, data collection, hydraulic model development, etc. are discussed. In Chapter four, dam breach parameter estimation and model setup are discussed. In this part, dam breach parameters, breach development time, peak breach outflow discharges are estimated and the river (Dedessa) is represented in hydraulic model (HEC-RAS). Flood inundation area delineation mapping also presented in this chapter. Chapter five covers the results of the modeling and the discussions on the output results. In chapter six, the emergency action plan preparation is presented. Finally in chapter seven, conclusions and recommendations are presented.

2. LITERATURE REVIEW

2.1. Introduction

In this chapter different literatures works including recent scientific journals, guidelines and books related to embankment dam breach modeling, breach parameter estimation methods, are reviewed widely according to the recent and updated different federal agencies dam breach guidelines and personal studies of other researchers in this study area.

A dam size categorization strategy was developed by the U.S. Army Corps of Engineers (USACE) for implementing Public Law 92-367 (National Dam Inspection Act) (Dams Sector, Homeland Security, 2011) and provides a useful measure for describing the size of the dams that have failed. Size classification may be determined by either storage or height, whichever gives the larger size category. In general, as a dam increases in size, the peak dam failure outflow, flood depths, and river distance experiencing dangerous flooding increase.

2.2. History of Dam Breach

As dams pose a serious threat to residents, businesses, infrastructure, landowners, crops, etc. downstream of them, it has always been important to analyze the causes and results of dam failure. There are currently about 80,000 dams listed in the U.S. national inventory (Steininger, 2014). 81% of these are earthen dams, and 1,595 are considered a significant hazard to a city downstream. Dam failures have proven to be quite deadly, destructive, and costly.

The history of water defence and water retention structures coexists with the history of their failures. Around the world thousands of dams have been constructed over many centuries. But also, hundreds of dams have failed and every year many dikes breach due to high flows in the rivers, sea storm surges, etc. often leading to catastrophic consequences. By far the world's worst dam disaster occurred in Henan province in China, in August 1975, when the Banqiao Dam and the Shimantan Dam failed catastrophically due to the overtopping caused by torrential rains. Approximately 85,000 people died from flooding and many more died during subsequent epidemics and starvation; millions of residents lost their homes (Qing, 1997).

In the Netherlands, in February 1953, a high–tide storm caused the highest water levels observed up to date and breached the dikes in more than 450 places, causing the death of nearly 1,900 people as well as enormous economic damage (Gerritsen, 2005).
According to the data from all U.S. dam failures, the majority of dam failure fatalities have been caused by dams having a size category of intermediate or large. The failure of four dams in the late 1800s and early 1900s: Mill River Dam, South Fork Dam, Walnut Grove Dam, and St. Francis Dam; all having a size category of intermediate or large, caused more than 2,800 fatalities. The failure of dams with a size category of small has caused comparatively few fatalities (Dams Sector, Homeland Security, 2011) In case of Africa, specifically east Africa, according to the Nile Basin Capacity Building Network for River Engineering (Kamal Eldin Bashar M. K., 2005) team assessment on an inventory and performances of micro dams in Sudan, Uganda and Ethiopia, a number of micro dams constructed in three countries were analyzed. As shown from their assessment results, most of the micro dams were constructed for the purpose of water supply and small numbers of these micro dams were constructed for the purpose of recharging groundwater. Among some of the causes of failure for these micro dams, most of them were faced severe siltation problems and some were failed due to spillway failure. Seepage of water beneath the dam or from the reservoir was also a problem.

In Ethiopia, traditional small scale irrigation schemes have existed centuries ago, particularly in the eastern, central, north western parts of the country (Kamal Eldin Bashar M. K., 2005) for the irrigation and water supply purposes. According to the Nile Basin Capacity Building Network for River Engineering study, the diversion structures were constructed of wood, stones, and grass with earth. They were often washed away during high river flows and have to be reconstructed each year. Modern small scale irrigation using micro-dams were given great emphasis after major droughts have stricken the country in 1973/74 and 1984/8).

From the Ministry of Water Resources technical team site visit report in 2003, on micro and medium dams constructed in 1970's and 1990's in Amhara and Tigray regions, the common problems for dam failure were overtopping due to inadequate spillway capacity flood estimation problem; seepage through foundation, abutments, and reservoir area site selection problem;

cracking or structural failure- geotechnical problem; sedimentation design problem and lack of watershed management; less inflow in the reservoir-hydrological analysis problem; and lack of proper maintenance and rehabilitation work (Kamal Eldin Bashar M. K., 2005).

Many embankment dams are constructed in Ethiopia most of which are used for irrigation purpose. However, their capacity reduces frequently before their design life time due to a number of reasons. The main causes of capacity reduction are Hydrological, Structural and Hydraulic failure of which hydraulic failures contributes 58% in Amhara region (Mekonnen, 208)

The probability of dam breach (failure) in current time is low. However, the consequence the dam breach in case it happens may be catastrophic and has multi-dimensional damages on the downstream areas (Andrew Charles, 2011)

2.3. Hazard Classification and Breach Failure Scenarios of Dams

The hazard potential classification of a dam, along with its size (height and capacity) classification, is used by United States, State agencies to regulate dam design and dam breach modelling. FEMA guidance recommends a three-step rating system that defines low-, significant-, and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure. (FEMA, 2013)

The two primary dam breach study approaches used in today’s dam breach analysis are an event-based approach and a risk-based approach. The event-based approach has been traditionally the most widely used for dam breach analysis. For the event-based approach, both a non-hydrologic “fair weather failure,” also referred to as a “sunny day failure,” and a specific hydrologic failure event, such as the Probable Maximum Flood (PMF), are usually established based on a dam’s hazard potential classification.

A risk-based approach which is not covered in this paper is commonly used for dam design purposes to establish the Spillway Design Flood (SDF) or Inflow Design Flood (IDF) for a dam.

2.4. Dam Breach Mechanism and Breach Parameters Estimation

A key element for calculating a dam breach hydrograph for a specific dam involves estimating the dam breach parameters for dam breach modelling related to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation. The shape of the peak breach outflow hydrograph is influenced by the storage in the impoundment at the time of breach, reservoir inflow at the time of breach, size of the dam, and most importantly, the dam material type's , and/ or mode of assumed failure.

Arjo-Dedessa dam has two outlet sluices (canal outlets) for the downstream irrigation purpose. For left and right abutments, extensive and intensive grouting was done and also the shattered and weathered material above the shear plane on the left and right abutment of the dam were removed and the area provided with consolidation grouting and well compacted.

Therefore, there is no fear of failure cases along the two sides of abutments and at the foundation below the bed level of Dedessa River at dam axis. Thus, it was assumed that the centerline of the breach can be set to the center line of the main channel invert at the bottom of downstream side of embankment dam axis in overtopping failure mode. But, for piping mode of failure in this study, the appropriate and logical point for failure location is at an elevation of 1339 meter(27m above bottom of the dam), where two left and right bank outlets are found. Due to the construction method problem and compaction difficulty at and around the outlet pipes in embankment dams, the degree of failure in embankment dam at outlets is high.

A number of methods are available for estimating breach parameters for use in dam breach studies. Since the selection of the breach parameters is specific to each dam, guidance is provided describing methods currently applied by dam safety professionals. The two major embankment dam breach mechanisms are overtopping and piping mode failure (FEMA P-946 /JULY 2013)

2.4.1. Overtopping Failure of Embankment Dam

Generally, overtopping failures of embankment dams (earthen/rock-fill) typically begin with head-cutting at the downstream toe and advance upstream until the erosion reaches the dam crest and reservoir surface. Once the reservoir is connected to the progressing breach, down-cutting of the embankment and lateral erosion occur until the breach expands to its final dimensions as shown in figure (2.1) below.

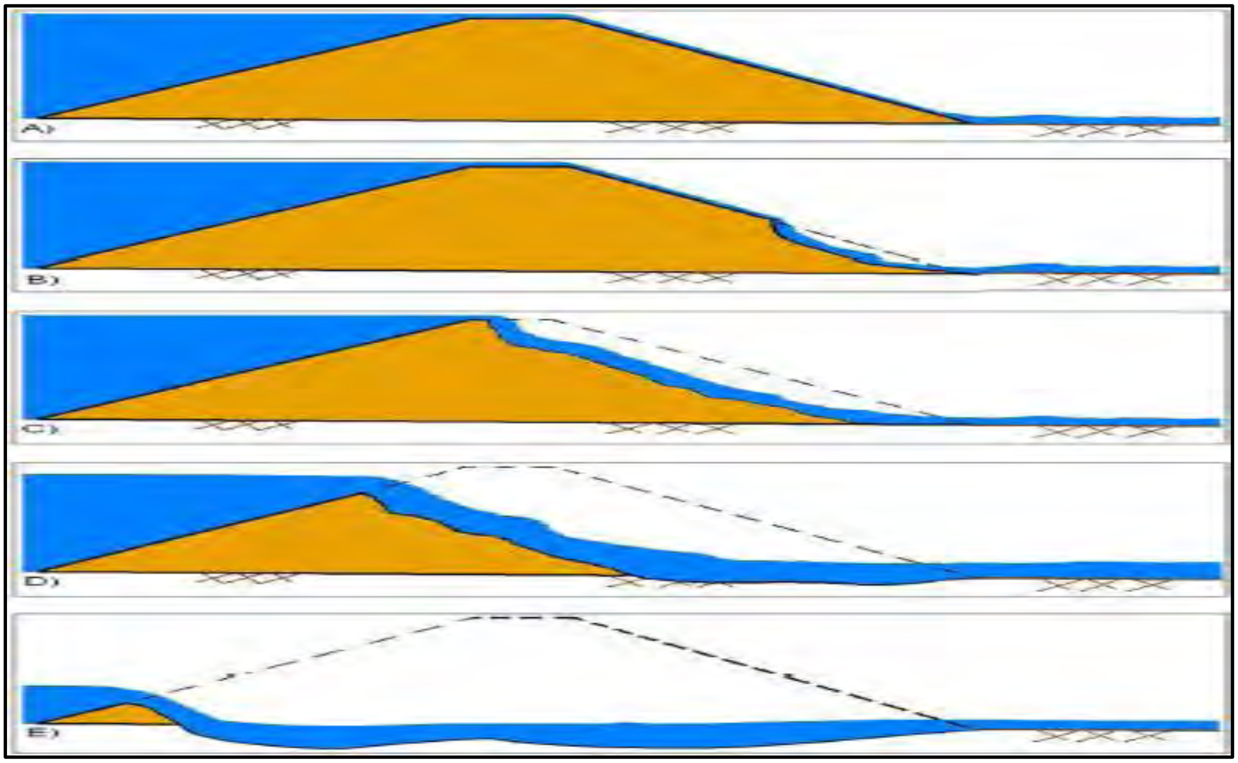


Figure 2-1: Breach Process of Overtopping (HEC-2014)

The breach is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent.

If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel. A generalized trapezoidal breach progression is illustrated in figure 2.2 below.

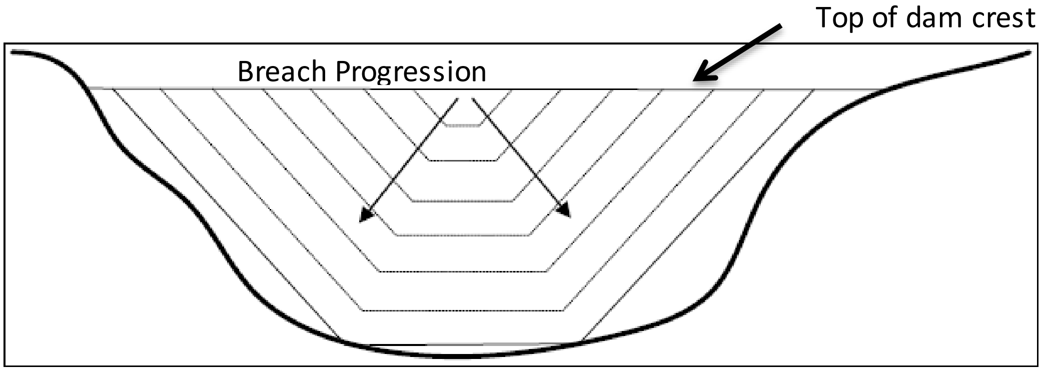
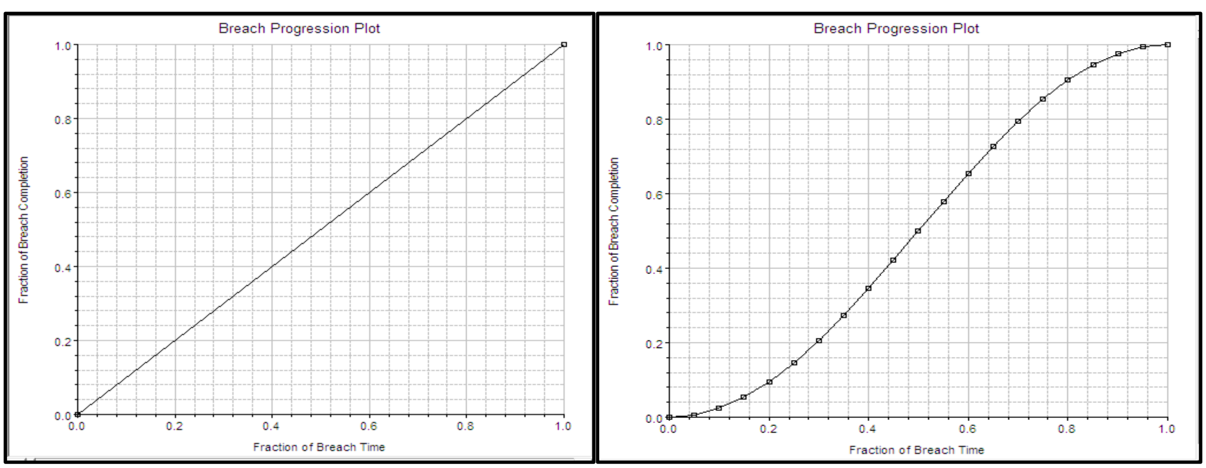


Figure 2-2: Overtopping trapezoidal breach progression, source: Gee (2009)

The breach may stop growing when the reservoir has emptied and there is no more water to erode the dam or the dam has completely eroded to the bottom of the reservoir or has reached bed rock (Gee, 2010) .The breach progression may be modelled as either a linear progression or a sine wave progression:

- **Linear progression:** rate of erosion remains the same for the duration of erosion development.

- **Sine wave progression:** breach grows very slowly at the beginning and end of development and rapidly in between.



(a) (b)

Figure 2-3: Breach Progression, (a) linear, (b) sine wave, USACE HEC-RAS (4.1.0)

In a study by the State of Colorado Department of Natural Resources (State of Colorado, 2010), no significant difference were found between linear and sine wave progression models when comparing one overtopping case study in HEC-Hydrologic Modelling System (HMS) and HEC-RAS (2010) (Brunner G. W., 2010). Both progressions should be evaluated and the progression with the more conservative results should be utilized. (FEMA P-946 /JULY 2013)

2.4.2. Piping / Internal Erosion Failures

Piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid (Brunner G. , 2014). Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse.

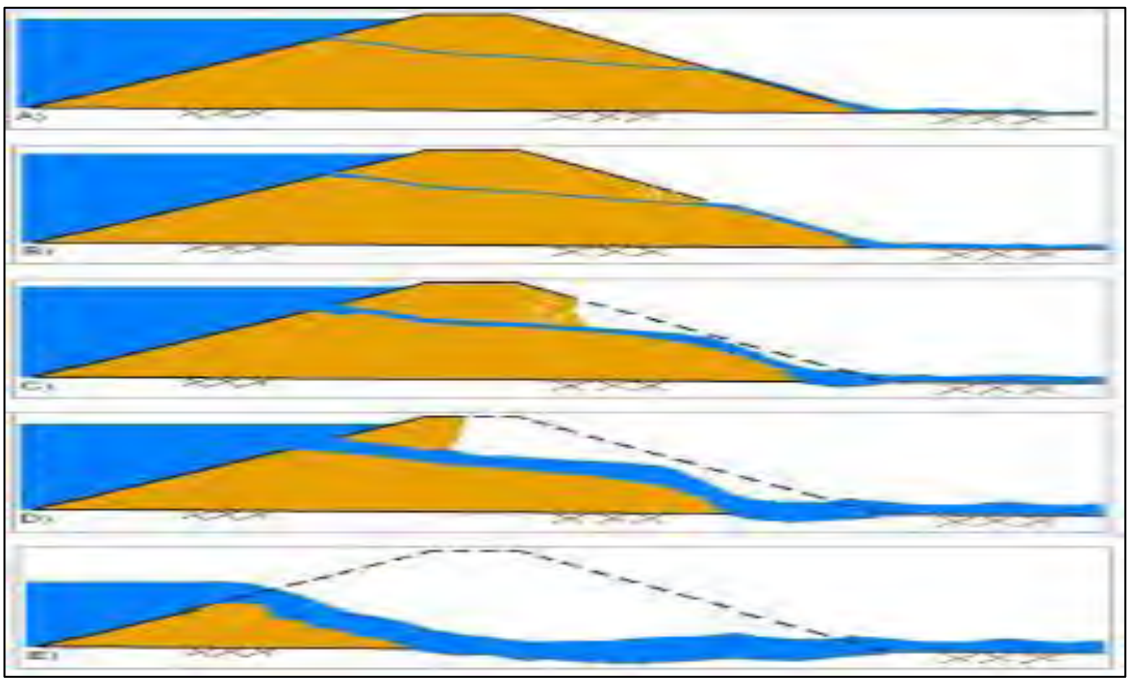


Figure 2-4: Breach Process for Piping Mode of Failure (HEC-2014)

Piping failures are typically modelled in two phases, before and after the dam crest collapses. Water flow through the piping hole is modelled as orifice flow before the dam crest collapses and as weir flow after the dam crest collapses. For small dams constructed from cohesive soils, it is possible for the reservoir to completely empty before the dam crest collapses (State of Colorado Department of Natural Resources, 2010).

Table 2-1: Dam Breach Weir and Piping Coefficients *(Brunner G. , 2014)*

Dam Type	Overflow/Weir Coefficients	Piping/Pressure Flow Coefficients
Earthen Clay or Clay Core	2.6-3.3	0.5-0.6
Earthen Sand and Gravel	2.6-3.0	0.5-0.6
Concrete Arch	3.1-3.3	0.5-0.6
Concrete Gravity	2.6-3.0	0.5-0.6

The commonly accepted and used in evaluating and selecting dam breach parameters in this study are:

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

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

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

Breach side slope factor – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V) (Wahl L. , 1998)

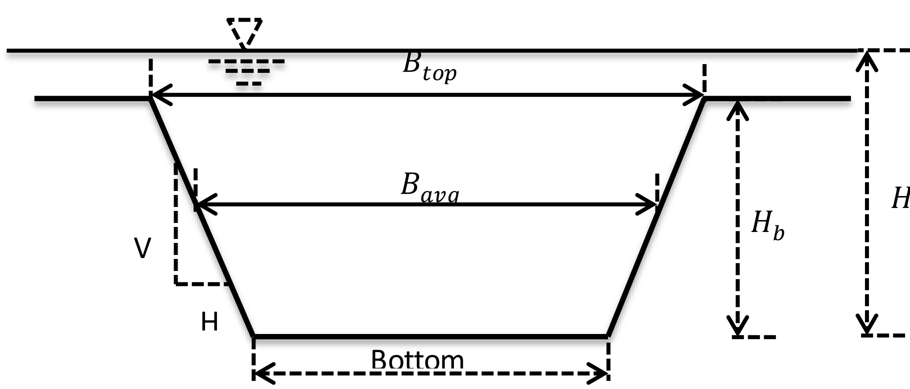


Figure 2-5: Description of the dam breach parameters

2.5. Breach Parameter Estimation Methods

According to dam breach modelling analysis methods overview made by Tony L. Wahl (Wahl T. L., 2010), three principal strategies for dam-breach flood modelling have emerged since the 1970s. The first strategy was to predict the breach outflow hydrograph directly and then use one of the available routing models to route that flood downstream (Wahl T. L., 2010) so that flooding consequences could be determined. The second approach was to parameterize the breach so that its evolution through time could be described in relative simple mathematical terms, allowing the breach outflow hydrograph to be determined by combining the description of the breach development with a weir equation or other appropriate model for simulating the hydraulic performance of the breach opening. Typical breach parameters determined were the maximum breach size, rate of breach development (or total time needed for full breach development). In this second approach, breach parameters could be determined by several different means externally to the flood routing model, but determination of the breach outflow hydrograph took place in the routing model. The third approach is to use a combined model that simulates specific erosion processes and the associated hydraulics of flow through the developing breach to yield a breach outflow hydrograph. Early models that took this approach were run separately from flood routing models, with the breach outflow hydrograph provided as input to the routing model. There is work being done now to integrate breach modelling and flood routing capabilities into a single model.

According to the State of Colorado Guidelines on Dam Breach Analysis, the four critical elements of any breach analysis are: breach parameter estimation (i.e. breach size/shape and time of failure), breach peak discharge and breach hydrograph estimation, breach flood routing, and estimation of the hydraulic conditions at critical locations. The most commonly used approaches for these required elements of the analysis are Comparative Analysis, Physically Based Erosion Models, Parametric Regression (Empirical) Equations and Predictor Regression Equations.

2.5.1. Comparative Analysis

This method compares a given dam of interest with those in a database of well documented volumes are compared with a list of similar sized dams that have failed. Dam breach parameters and peak discharges reported from the failure case histories of similarly configured dams are then directly applied to the dam being analysed. If the dam under consideration is very similar in size and construction to a dam that failed, and the failure is well documented, appropriate breach parameters or peak outflows may be determined by comparison.

2.5.2. Physically Based Erosion Models

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

These methods predict the development of an embankment breach and the resulting breach outflows using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics.

Since the 1960s there have been numerous developments of physically-based, numerical dam breach models. In 1965, the first breach model was proposed by Cristofano, 1965 (Singh, 1965), pioneering the development by others of physically based models BRDAM (1977), Dam Break Forecasting Model (DAMBRK) (1977), Breach Erosion of Earth-Fill Dams and Flood Routing (BEED) (1985), and BREACH (NWS, 1988).

Currently, the NWS BREACH model is a well-known and commonly applied physically based model developed by a Federal agency. The NWS BREACH model was developed to more realistically simulate breaches initiated by overtopping or piping in an embankment dam. A modified form of the Meyer-Peter and Muller sediment transport equation is used in this model. The NRCS SITES and WinDAM models are other Federal-sponsored models.

2.5.3. Parametric Regression/Empirical/ Equations

These equations, developed from case study information, are used to estimate the time-to-failure and ultimate breach geometry. The breach can then be simulated to proceed as a time-dependent linear process with the computation breach outflows using principles of hydraulics.

The most common parametric regression equations developed based on information from case studies of historic dam failures are shown in table below (2.2).

Table 2-2: Parametric Regression Equations for Predicting Breach Parameters (Adapted from DSO-98-004, USBR 1998) *(L.Wahl, 1998)*

Reference	Number of Studies	Relations Proposed (S.I. units, meters, m3/s, hours)
MacDonald and Langridge-Monopolis (1984)	42	Earth-fill dams $V_{er} = 0.0261(V_{out} * H_w)^{0.769}$ [best fit] $T_f = 0.0179(V_{er})^{0.364}$ [upper envelope] Non Earth-fill dams $V_{er} = 0.00348(V_{out} * H_w)^{0.852}$ [best fit]
Von Thun and [illegible]te[illegible] (1990)	57	$B_{avg} = 2.5h_w + C_b$ $T_f = 0.02h_w + 0.25$ (For erosion resistant materials) $T_f = 0.015h_w + 0.25$ (For easily erodible materials)
Froehlich (2008)	74	$B_{avg} = 0.27K_o * V_w{}^{0.32} * H_b{}^{0.04}$ $T_f = 3.664(\frac{V_w}{gH_b^2})^{0.5}$ $K_o = 1.3\ for\ overtopping; 1\ otherwise$

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time. Many federal agencies have published guidelines in the form of possible ranges of values for breach width, side slopes, and development time. These guidelines should be used as minimum and maximum bounds for estimating breach parameters.

Table 2-3: Ranges of Possible Values for Breach Characteristics of Different Agencies (USACE-HEC July 2014)

Dam Type	Average Breach Width B_{avg}	Horizontal Component of Breach Side Slope (H) H:1V	Failure Time t_f (Hrs.)	Agency
Earthen/Rock-fill	(0.5 to 3.0) x h_d	0 to 1.0	0.5 to 4.0	USACE (1980)
	(0.5 to 5.0) x h_d	0 to 1.0	0.1 to 4.0	USACE (2007)
	(1.0 to 5.0) x h_d	0 to 1.0	0.1 to 1.0	FERC (1988)
	(2.0 to 5.0) x h_d	0 to 1.0 (slightly larger)	0.1 to 1.0*	NWS (Fread,2006)

Dams that have very large volume of water and have long dam crest length will continue to erode for long durations (i.e., as long as a significant amount of water is flowing through the breach), and may therefore have longer breach width and times than what is shown in table above (2.3). According to the State of Colorado, Guidelines for Dam Breach Analysis, The MacDonald & Langridge-Monopolis (1984) utilizes 42 data sets (predominantly earth-fill dams, earth dams with clay core, rock-fill dams) to develop a relationship of the volume of water coming out of the dam and the height of water above the dam breach invert. The ranges are:

- ✓ Height of the dams:4.27-92.96 meters (with 76 %<30 meters, and 57 % <15 meters)
- ✓ Breach outflow volume:0.0037-660.0 m3*10^6 (with 79%<25.0m3*10^6, and 69 % <15.m3*10^6) (USACE-HEC July 2013)

Other researchers, Von Thun and Gillette (1990) and Dewey and Gillette (1993) used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (h:v) may be more appropriate and they found

$$B_{avg} = 2.5h_w + C_b$$

Table 2-4: Values of coefficient C_b (Von Thun and Gillette, 1990)

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

They proposed two methods for estimating breach formation time for erosion resistant and easily erodible materials.

$$T_f = 0.02h_w + 0.25 \text{ (For erosion resistant materials)}$$

$$T_f = 0.015h_w \text{ (Easily erodible)}$$

From 57 dam's data regression analysis, they had the following ranges:

- ✓ Height of the dams: 3.6-92.96 meters (with 89% <30 meters, and 75 % < 15 meters)
- ✓ Volume of water at breach time: 0.027-660 m3*10^6 (with 89 % <25.0m3*10^6, and 84%<15.0m3*10^6) (USACE-HEC July 2014)

The more recent dam breach analyses method, Froehlich empirical method (Froehlich, 2008) is widely used in todays practice in the dam breach modelling fields. It is dependent only on the volume of the reservoir, height of the breach and the assumed breach side-slope.

Froehlich (2008) method utilized 74 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock-fill data set to develop a set of equations to predict average breach width, side slopes, and failure time. According to his data:

- ✓ Height of the dams:3.05-92.96 meters (with 93 %<30 meters, and 81 % <15 meters)
- ✓ Breach outflow volume:0.0139-660.0 m3*10^6 (with 86%<25.0m3*10^6, and 82 % <15.m3*10^6) (USACE-HEC July 2013)

In 2011, three Egyptian researchers (Samir, 2011), modeled prediction of breach formation through Aswan Rock-fill High Dam. In their study, they assessed the breaching due to overtopping, numerically. For their study, one dimensional model (HR-BREACH) developed by HR Wallingford was applied. This model can simulate the failure of homogenous or composite embankment dams by overtopping or piping. It takes into account the soil mechanics principle in the breaching process and is based up on the principles of hydraulics and sediment transport.

From six flood scenarios they used, the breach width result of four scenarios were estimated within the ranges of possible values of breach characteristics guidelines published by many federal agencies, while two of them out of the range. But, the breach formation times calculated from all six scenarios were out of the ranges.

2.5.4. Predictor Regression Equations

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

These equations are based on case study data used to develop empirical equations relating peak breach outflow to dam height and/or reservoir storage volume (FEMA, 2013). The predictor regression equations provide an alternative method of computing the dam breach discharge; they can be used instead of determining breach parameters and then using a hydrologic-hydraulic model to compute the breach hydrograph.

Table 2-5: Predictor Regression Equations for Prediction of Peak Breach Flow (FEMA, P-946/July 2013)

Reference	Case Studies	Relations Proposed
MacDonald and Langridge-Monopolis (1984)	42	$V_{er} = f(V_{out} * h_w)$ $T_f = f(V_{er})$ $Q_p = f(V_{out} * h_w)$
Von Thun and Gillette (1990)	57	Z-guidance $B_{avg} = f(h_w * S)$ $T_f = f(h_w, errosion\ resistance)$
Froehlich (2008)	74	$Q_{max} = 3.1 B_{avg} H_w^{1.5} \left(\frac{\gamma}{\gamma + T_f \sqrt{H_w}}\right)^3$

The two researchers (Saqib Ehsan, 2014), were tried to model a dam break analysis, for very large dams. Mangla dam, located on Jhelum River in Pakistan, has been taken into consideration. With a height of about 125 m, Mangla dam is one of the largest earth and rock fill dams in the world. The Erosion based overtopping failure of Mangla dam with raised conditions has been analyzed by using MIKE 11 dam break model. The parametric approach has been used to estimate the breach geometry and simulate the breach growth as a linear process in order to determine the breach outflows. Different cases of dam breach have been considered for dam break simulations. The failure outflow hydrographs have been computed for different breach cases. Moreover, the time to failure and the breach erosion rate have also been determined for the considered breach cases. The maximum outflow for the worst case of dam failure is about 160,000m^3/s, which is about 2.5 times the maximum design flood for Mangla dam.

Another researcher (Xiong, 2011) described the dam break in the aspects of theories and models. Break parameters prediction, the understanding of dam break mechanics, peak outflow prediction were shown as essential for the dam break analysis. As an application example, Foster Joseph Sayer Dam Break was further modeled and analyzed using HEC- RAS based on available geometry data.

Two researchers (Brunner C. T., 2008) were also discussed the process for gathering and preparing data, creating an unsteady-flow model in HEC-RAS, entry of dam breach parameters, performing a dam failure analysis, and mapping of the flood progression.

Using two dimensional numerical models (CCHE2D-FLOOD) based on a finite volume method, Xinya Ying and Sam S. Y. Wang simulated flood inundation due to dam and levee breach. The model solves the conservative form of the two dimensional shallow water equations using the finite volume method.

Prediction of dam breach parameters using the empirical equations of Froehlich and Macdonal-Langridge-Monopolis formed the basis of the modeling, coupled with MIKE 11 software to obtain the breach outflow due to Probable Maximum Flood (PMF) (Azwin Zailti Abdul Razad, 2013). In their works, they discussed the model setup, simulation procedure and comparison of the prediction with existing equations.

(Kho, Law, Lai, Oon, Ngu, & Ting, 2009), carried out mathematical simulations on dam break or failure using BOSS DAMBRK hydrodynamic flood routing dam break model to determine the extent of flooding downstream, flood travel times, flood water velocities and impacts on downstream affected residences, properties and environmental sensitive areas due to floodwaters released by failure of the dam structure. Computer simulations for one of the worst case scenarios on dam failure using BOSS DAMBRK software accounted for dam failure, storage effects, floodplains, over bank flow and flood wave attenuation. The simulated results reviewed a maximum flow velocity of 2.40 m/s with a discharge of approximately 242 m^3/s occurred at 1.00 km downstream. The maximum discharge increased from 244 m3/s (flow velocity = 1.74 m/s occurred at 8th. km) to 263 m3/s (flow velocity = 1.37 m/s occurred at 12th. km); about a 39% drop in flow velocity over a distance of 4.00 km downstream. If the entire dam gives way instantly, some spots stretching from 0.00 km (at dam site) to approximately 3.40 km downstream of the dam may be categorized as "danger zone", while downstream hazard and economic loss beyond 3.40 km downstream can be classified as "low" or "minimal" zones.

Simulation of embankment dam breach events and the resulting floods are crucial to characterizing and reducing threats due to potential dam failures. Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at a given location. If population centers are located well downstream of a dam, details of the breaching process have little effect on the result; travel time, attenuation, and other routing effects predominate. However, in a growing number of cases, the location of population centers near a dam makes accurate prediction of breach parameters (e.g., breach width, depth, and rate of development) crucial to the analysis. If breach parameters cannot be predicted with reasonable accuracy, increased conservatism with associated increased costs may be required. This paper examines existing empirical procedures and a numerical model used to predict breach parameters, reviews new technologies relevant to dam breaches, and outlines a program for development of an improved numerical model for the simulation of embankment dam breach events.

2.6. Hydrological Analysis

In the analysis of rainfall for Arjo-Dedessa Irrigation project, rainfall reliability level and magnitude have been analyzed by Extreme Value Type III (minimum) distribution for annual monthly and half monthly intervals and the maximum rainfall magnitudes intensity corresponding to various return periods have been estimated using frequency distribution function:

$$X_T = \bar{X} + k\sigma$$

Finally, they have got the maximum rainfall magnitudes of intensity corresponding to various return periods by using:

$$I_T = I_m[1 + CV.K(T)]$$

In order to estimate floods for ungagged catchments for Arjo-Dedessa Project area, rational method, SCS method and transferring gauged data were used (OWWDSE, 2013)

Two distribution methods have been used for Arjo-Dedessa Dam flood frequency analysis. These methods are Generalized Extreme Value (GEV) distribution (Jenkinson, 1969), and Log-Logistic (LLG) distribution (Ahmed et al., 1988). Finally, LLG/PWM has been applied for estimating design floods required for the project. Then using the mean annual maximum 24-hr rainfall of the catchment and catchment characteristics, the maximum probable flood (PMF) was calculated.

2.7. Dam Breach Modeling Tools

Performing a dam breach model involves prediction of the dam breach hydrograph and the routing of that hydrograph downstream (FEMA, 2013). A number of modelling tools are available to perform dam breach modelling, ranging from simple methods to complex models. With advancements in GIS-based modelling, many models can interface with digital terrain data to produce automated dam breach inundation zone delineations.

Dam breach modelling can have tools that generate the dam breach peak discharge and/or hydrograph only; and tools that develop a breach hydrograph and perform downstream flood routing using a one- or two-dimensional hydraulic model.

2.7.1. Dam Breach Hydrograph and Peak Outflow Generation Tools

In the dam breach modeling analysis, the determination of outflow hydrograph in the event of dam breach is the major task. The volume represented by the hydrograph is the storage volume of the reservoir released during the breach. Factors that affect the shape of the hydrograph include: size and shape of the breach, breach formation time, depth of water at the dam, volume of stored water, surface area of reservoir, shape of the reservoir.

Table 2-6: Most Widely Used Dam Breach Modelling Tools (FEMA, P-946/July 2013)

Method	**Computation of Peak Breach Outflow**	**Computation of Ultimate Breach**	**Breach Hydrograph**	**Downstream Routing Capability**			
				Steady	**Unsteady**	**1-D**	**2-D**
Breach Hydrograph Generation Only							
Empirical	✓	✓					
NWS BREACH	✓	✓	✓				
USACE HEC-1 & HEC-HMS	✓		✓	Without downstream hydrologic routing	✓		
Breach Hydrograph Generation and Downstream Hydraulic Routing One-Dimensional Models							
NRCS TR-66	✓		✓	✓	✓	✓	
WinDAM	✓	✓	✓				
NWS SMPDBK	✓		✓	✓		✓	
NWS FLD WAV	✓	✓	✓		✓		

USACE HEC-1 and HEC-HMS	✓		✓	Without downstream hydrologic routing		✓	
USACE HEC-RAS	✓		✓	✓	✓	✓	
Two-Dimensional Models							
DSS-WISE	✓		✓		✓		✓
FLO-2D©	✓	✓	✓		✓	✓	✓
MIKE© FLOOD	✓		✓		✓	✓	✓

For this study purpose, United State Army of Corps Engineers, Hydraulic Engineering Center River Analysis System (USACE, HEC-RAS 4.1) was selected in order to generate dam breach outflow hydrograph and downstream routing analysis.

2.7.2. Dam Breach Flood Routing

Routing of the breach outflow hydrograph downstream to evaluate the potential consequence of dam failure is the major and main task in modeling dam breach and downstream risk analysis. HEC-RAS can be used to route an inflow flood hydrograph through a reservoir either with one-dimensional unsteady flow routing; or two- dimensional unsteady flow routing (Full Saint Venant Equations).

Full unsteady flow routing (one or two-dimension) will be more accurate for both with and without breach scenarios. The unsteady flow routing method can capture the water surface slope through the pool as the inflowing hydrograph arrives, as well as the change in water surface slope that occurs during a breach of the dam.

One-dimensional models solve either full dynamic or simplified forms of one-dimensional, cross-section-averaged shallow water equations. The one-dimensional models discussed in this study also have downstream routing capabilities. Two-dimensional models which were not discussed in this paper use full dynamic or simplified forms of one- and two-dimensional shallow water equations to solve both one-dimensional channel flow and two-dimensional overland flow. The fully dynamic one dimensional flow routing through the reservoir and downstream river reach which is applied for this study paper is shown below.

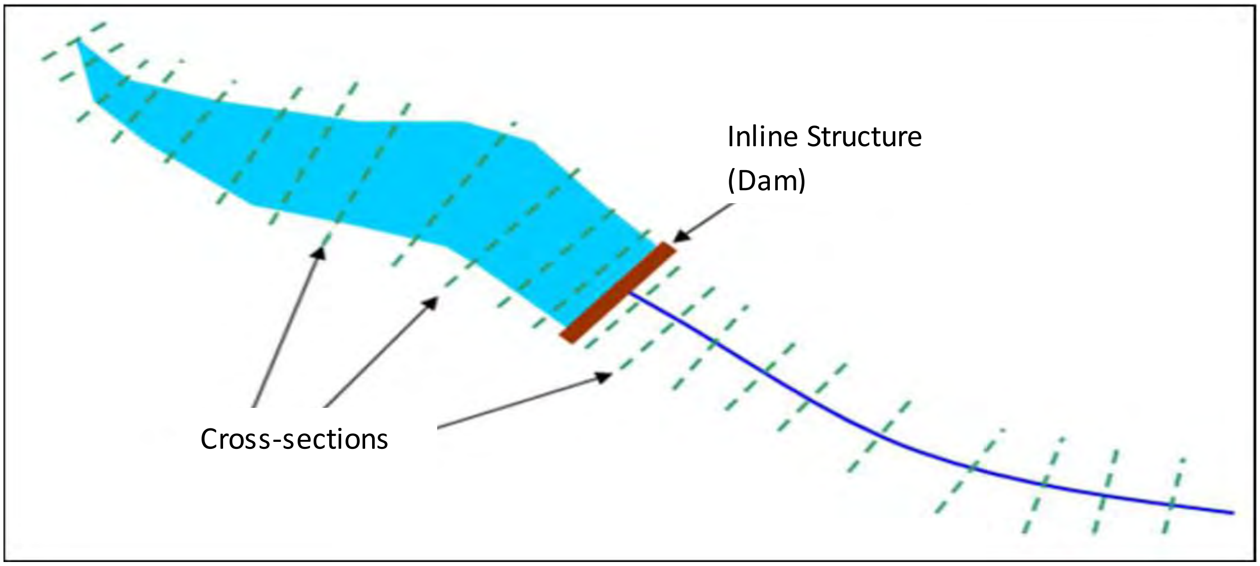


Figure 2-6: Cross-Section Layout for One Dimensional Full Dynamic Routing Through Reservoir (HEC-2014)

2.7.2.1. Basics of One-Dimensional Flow Routing

In inundation analysis, flow modelling is used to simulate the flow of a flood wave through a river reach and its floodplains. For modelling and design purposes, continuity, momentum and energy equations have been developed in the past to represent open channel flow in a mathematical way. These equations are based on three basic laws of physics, which are the continuity of mass, the continuity of momentum and the continuity of energy. Flow models simulate the flow through an open channel in a way that satisfies these basic equations for open channel flow or simplified versions of them.

In one-dimensional flow routing, flow through the river channel and the floodplains is treated only in the longitudinal direction parallel to the reach.

2.7.2.2. Governing Equations

The law which govern the flow of water in a stream are the principle of conservation of mass (continuity equation) and the principle of conservation of momentum. These laws are expressed mathematically in the form of partial differential equations i.e. continuity equation and momentum equation.

Continuity Equation

$$\frac{\partial A_T}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \qquad \textit{Continuity Equation}$$

Continuity equation describes conservation of mass for the one –dimensional system. The equation considers the elementary control volume as shown below.

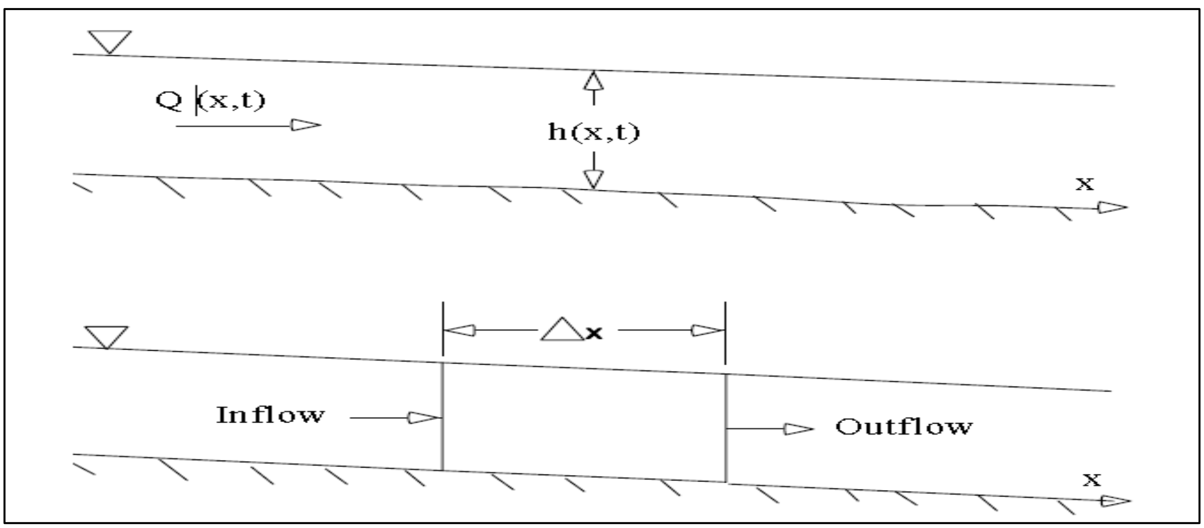


Figure 2-7: Elementary Control Volume for Derivation of Continuity and Momentum Equations (HEC-2010)

Momentum Equation

The momentum equation state that rate of change in momentum is equal to the external forces acting on the system.

$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA(\frac{\partial z}{\partial x} + S_f) = 0 \qquad \textit{Momentum Equation}$$

This equation 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 (HEC-2010)

2.8. Inundation Mapping

According to Washington State Department of Ecology Dam Safety Guidelines, the inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It should also identify zones of high velocity flow and show inundation for representative cross-sections of the channel.

Two reservoir conditions, normal pool and maximum pool storage elevation, are usually examined in assessing the downstream consequences. The inundation delineation provides the most important dataset for inundation mapping, the inundation polygon. A polygon refers to a type of layer in a GIS. Once a dam breach simulation is completed, the data can be stored in GIS to provide many potential benefits to users including the potential for improved accuracy and efficiency. Many GIS tools, including the USACE HEC-GeoRAS ArcMap extension and RAS Mapper, provide tools to automatically delineate floodplains or inundations as a post-processing function of HEC-RAS (FEMA, P-946/July 2013)

2.9. Developing Emergency Action Plan

An Emergency Action Plan (EAP) is a formal but simple plan that identifies potential emergency conditions that could occur at a dam, and prescribes procedures to follow to minimize loss of life and the potential for property damage (Washington State, 2013). Ideally, the design, construction, operation, maintenance, and inspection of dams are all intended to minimize the risk of dam failures.

According to the State of Washington Department of Ecology (Washington State Department of Ecology, 2013), the primary function of an EAP is to provide a means of notifying downstream residents of failure or impending failure of a dam, so that the area can be evacuated in a timely manner. To accomplish this, the EAP must provide procedures to evaluate those conditions at the dam that could lead to failure, and clearly identify the circumstances under which the EAP is to be implemented. Secondly, it is used to identify strategies that can be taken following discovery of an emergency situation to prevent failure, or alternatively to delay failure until after downstream residents have been warned.

3. METHODOLOGY

3.1. Study Area and Data Collection

3.1.1. Location

Arjo-Dedessa Irrigation Project Dam is found in between Jima and East Wollega Zones of Oromia Regional State. The project area is located between 8°-30'-00" to 8°- 40'- 00'' N Latitude and 36°-22'-00" to 36°-43'-00" E Longitude.

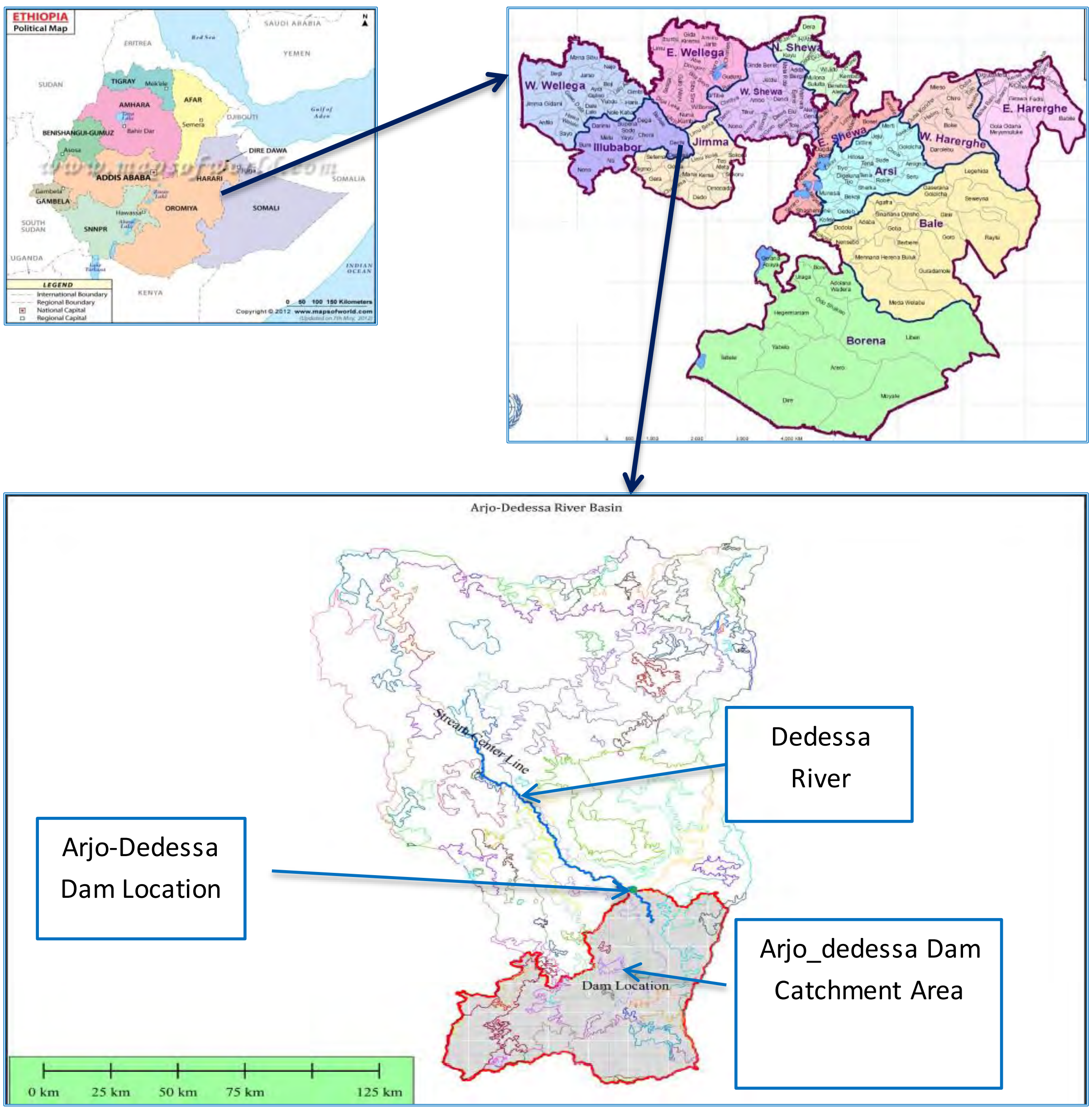


Figure 3-1: Geographical Location of the Study Area

3.1.2. Topography

The climate of the Dedessa Catchment results from its location and elevation (1220 to 3012 meters). The catchment is characterized by mountainous, highly rocky and divided topography with deep slopes. The lowest part of the catchment is characterized by valley floor with flat to gentle slopes.

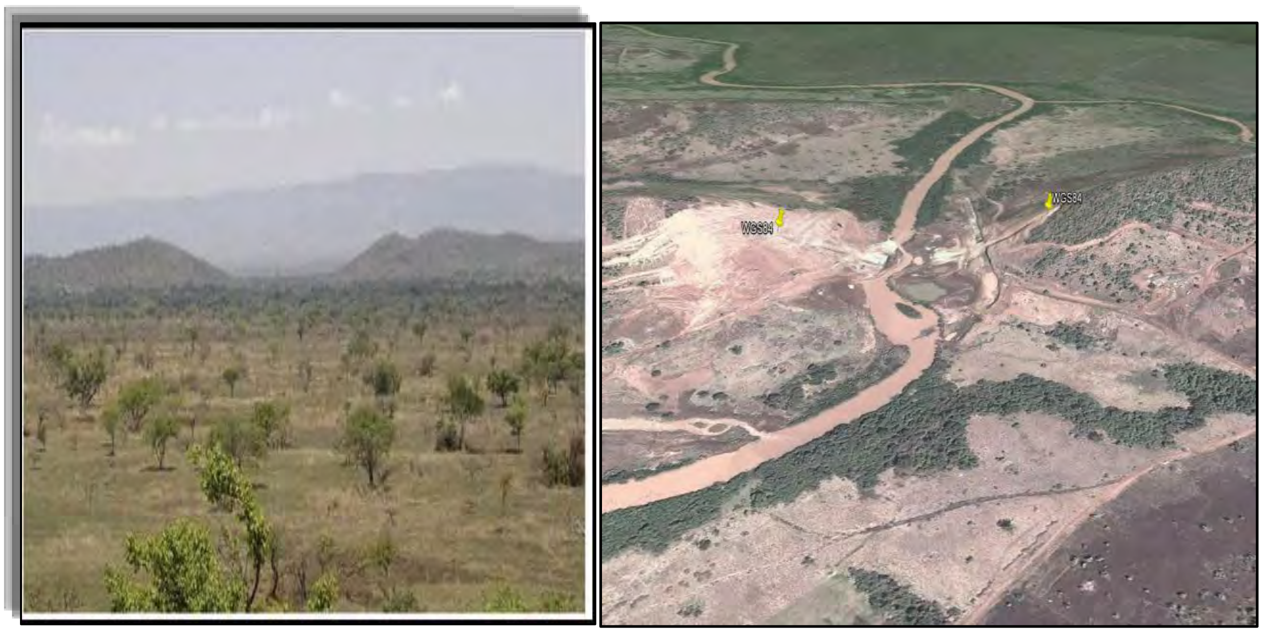
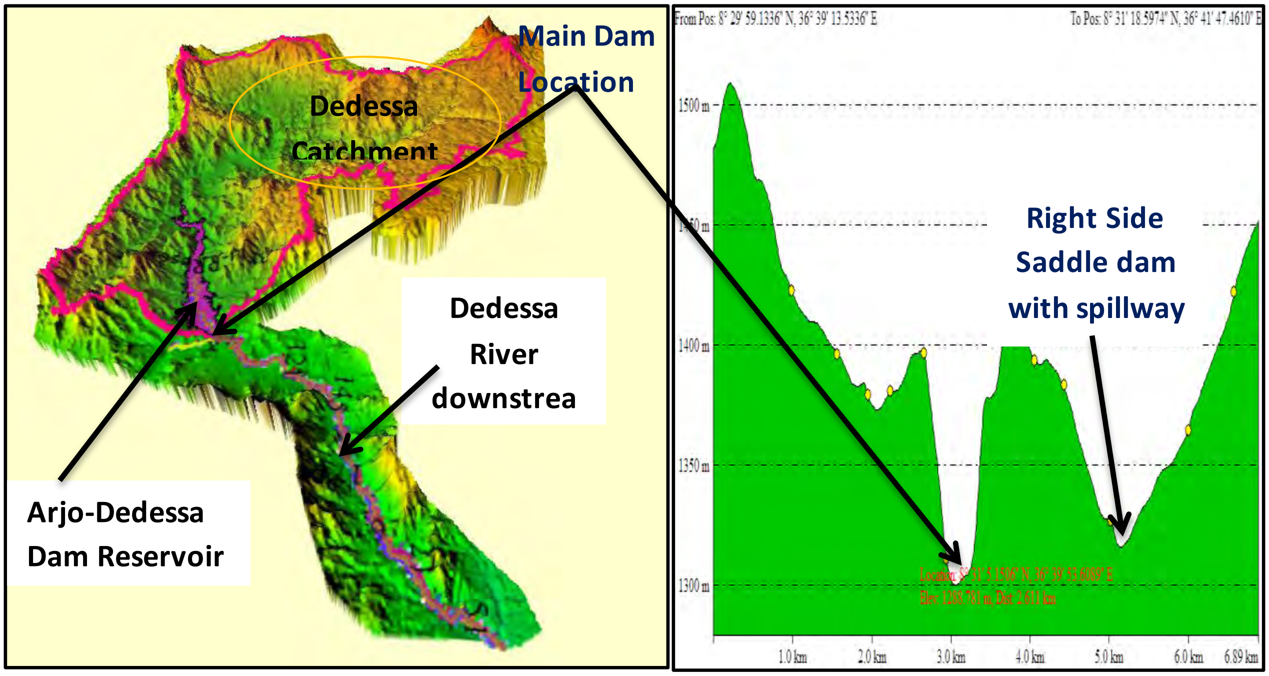


Figure 3-2: Topographic Features of the Dam Site and Reservoir Area

Arjo-Dedessa catchment area up to the proposed dam is about 5,632.64 square kilometers and the catchment area up to confluence with Abay river 34,000 square kilometers .The annual rainfall of the area is 2013 mm (Dembi) to 1417 mm (Agaro) and the average annual rainfall is 1453mm.There is an irrigable command area of 80,000 hectare on both left and right banks at 40 km downstream of the dam (OWWDSE, 2013)



(a) (b)

Figure 3-3: Dedessa Catchment, (a) DEM representation Catchment Area (b) Cross-section of the dam site (ArcGIS v.10 and Global Mapper v. 15)

3.1.3. Hydrology and Climate

The Dedessa River is the largest tributary of the Abbay River in terms of the volume of water contribution to the total flow of Abbay at the Sudan border. Draining nearly an area of 34,000 square kilometers, the Dedessa River originates in the Mt.Vennio and Mt.Wache ranges, flowing in an easterly direction for about 75 kilo meters, then turning rather sharply to the north until it reaches the Abbay River. The major tributaries of the Dedessa River are the Wama, entering from the east, the Dabana from the west, and the Angar from the east (OWWDSE, 2013)

Dedessa Catchment is situated in the south-west part of Abbay Basin. The catchment area at a gauging station near Arjo town is 9,981 km^2. The mean annual flow of Dedessa river at Arjo station is about 3,800 million m^3 having its maximum flow in August and September (52 percent of the annual) and minimum flow in February and March (less than 1.5 percent of the annual).

Most of the rainfall in the Dedessa Catchment is concentrated in the June to September period with virtual drought from November through February. Annual totals, average from less than 150 centimeters to more than 200 centimeters. The Dedessa catchment, besides reflecting a marked rainfall increase with higher elevation, also receives heavier annual quantities than most of the catchments in the Abbay Basin. Examination of the rainfall records from this area shows that this is due to a longer (May through October) rainy season rather than heavier maximum monthly quantities.

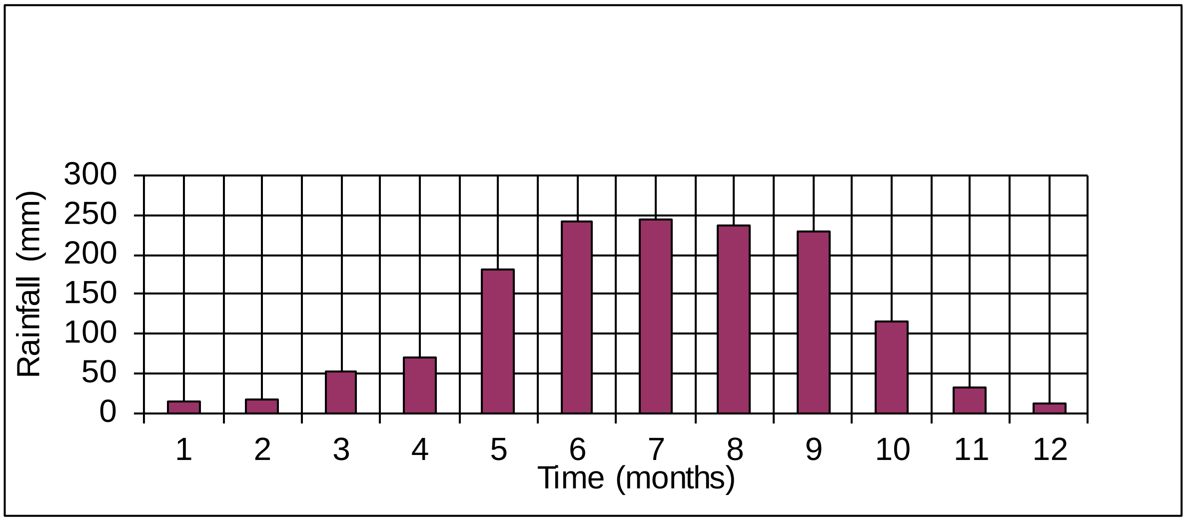


Figure 3-4: Mean Monthly Rainfall at Arjo Dedessa Projetct Area (OWWDSE, Dec-2013)

From five meteorological observation stations located in and around the Arjo-Dedessa Irrigation Project area (i.e. Jimma, Bedele, Dedessa, Agaro, and Dembi), Jimma station which is Class 1 station and includes observations of rainfall, temperature, relative humidity, sunshine duration, wind speed, and evaporation for 50 years is taken to be the most reliable Class I station from which meteorological information relevant to the project area has been derived (OWWDSE, 2013). In the project area, the mean monthly temperature variations throughout the year are 20.0^{o}C in December to 25.4^{o}C in March.

3.1.4. Geology

An area about 70 km X 80 km neighbouring the Dedessa river to the NE and SW and within a radius of 35 km from the dam axis was selected for detailed geological investigations, especially potentially active faults that might adversely affect this major infrastructure.

In the downstream course of the Dedessa valley from the dam axis, the river cuts through Precambrian granite gneisses and granites. On the upstream side, up to its head waters north of Gojeb Valley, the river cuts through Tertiary volcanic rocks, belonging to the Limu Genet and Arjo volcanic. The entire volcanic pile has a cumulative thickness of about 1000 m. East and west of Arjo town, the Precambrian gneisses and granites are locally overlain by Paleozoic-Mesozoic sandstones with intercalations of conglomerates, silt stones, etc.(OWWDSE, Dec-2013)

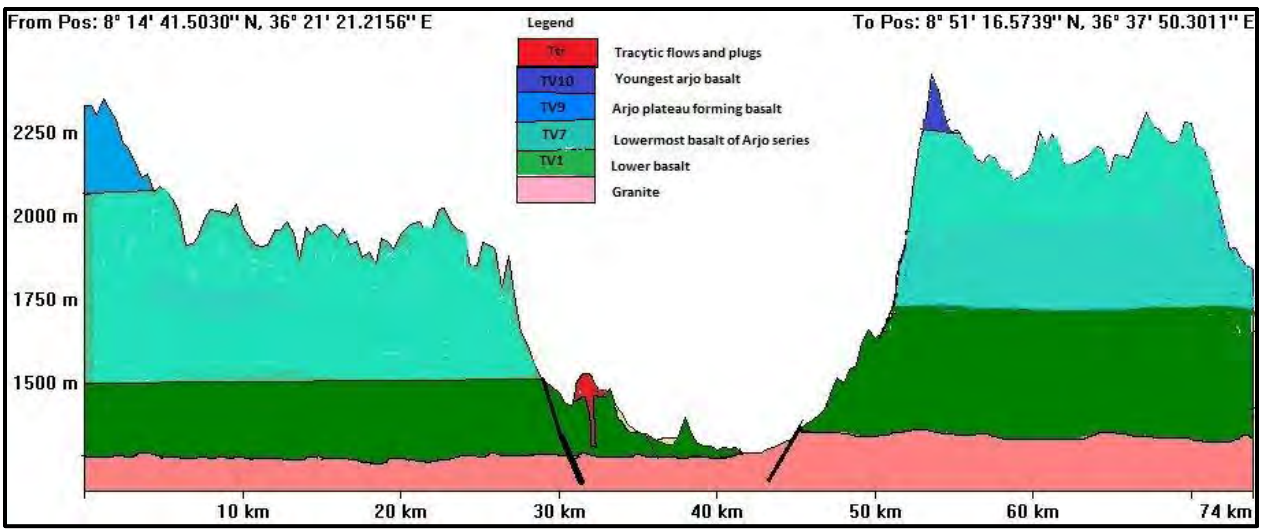


Figure 3-5: Geological Cross section of Dedessa River at Dam site (OWWDSE, Dec-2013)

3.1.5. Dam Type

Arjo-Dedessa Irrigation Dam is a high rock-fill embankment dam on Dedessa River. It comprises of 502.473 m long and 47.0 m high rock fill dam with central impervious clay core and an ogee spillway on the right bank of river in the Saddle Dam-I. Saddle Dam –I is 710.94m long and 17.0 m (deepest section) high rock fill dam. The length of Saddle is 710.94 m includes spillway, non-overflow blocks and rock fill dam on both flanks of spillway. Saddle Dam –II, 397.04 m long and 10 m high rock fill dam adjacent to Saddle Dam I Irrigation sluices for drawing water from the reservoir for the right bank primary canal provided in Saddle Dam-I adjacent to spillway.

Second irrigation sluices taking off from the left abutment of the Main dam for the left bank primary canal.

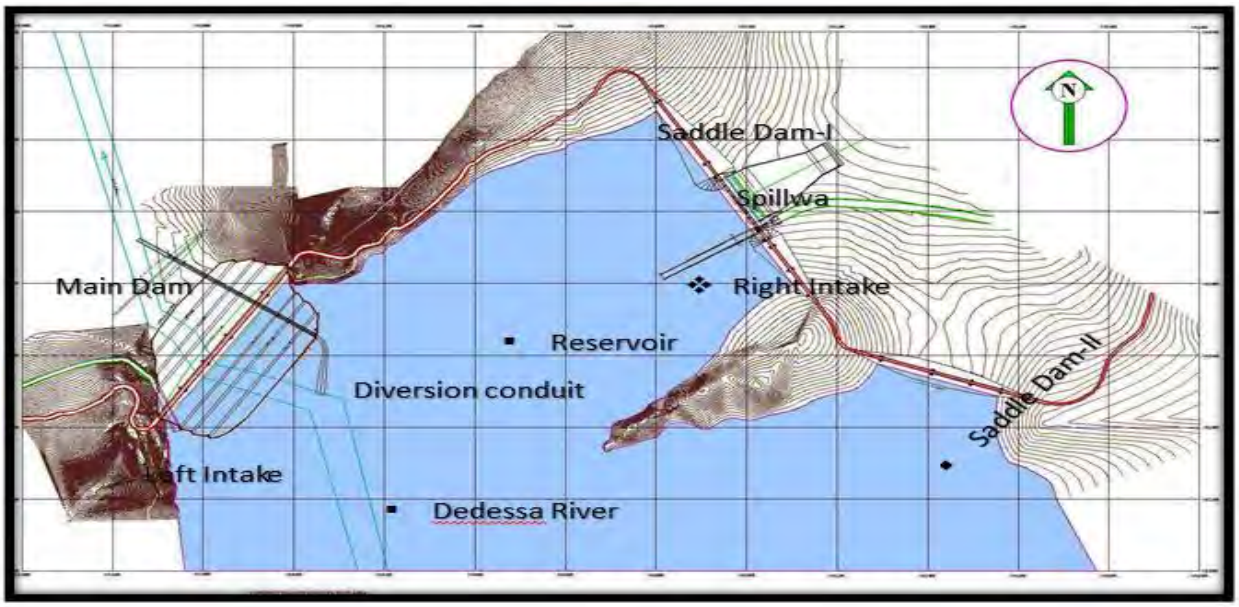


Figure 3-6: General Layout of Arjo-Dedessa Dam Main Components (OWWCE-2014)

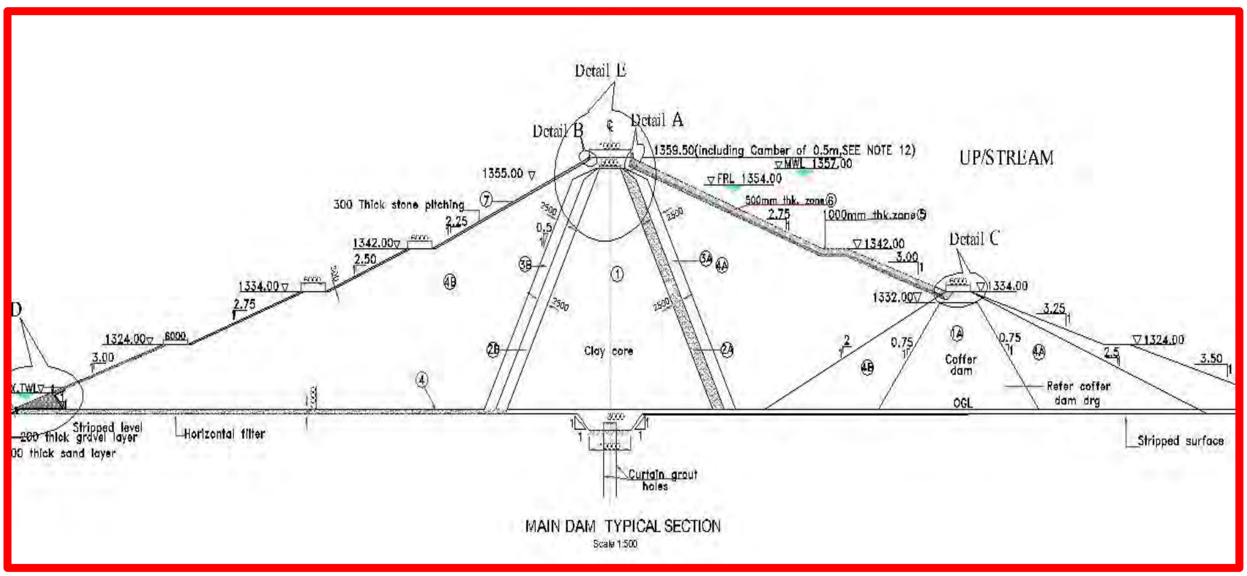


Figure 3-7: Arjo-Dedessa Dam Cross-Section, Zonings of the Main Dam (OWWDSE, Dec-2013)

3.2. Data Collection

The correctness and accuracy of the data included in breach modelling must be considered in order to predict the failure impact analysis. Some necessary information needs to be collected to model dam breach and to determine the effects of a dam failure on the downstream flood plain.

3.2.1. General Information

This section gives an overview of the study area as well as the data used for Hydraulic modeling od dam breach analysis. Floods related to dam failure are generally significantly larger than natural floods. For Arjo Dedessa Dam breach modelling and downstream analysis, hydrologic data, reservoir capacity are all used from the design report of OWWDSE, 2013.

3.2.2. Topographic Data

A study area description of the land surface is an important part in the process of quality river hydraulics model. Elevation (topography) data is used to establish the area available to transport flow downstream. In order to map the flood inundation area due to the water surface profile outflow from the dam breach, topography data or elevation data must be known. The primary topographic (elevation) data for river channel, left and right banks and each cross-section used for the analysis of Arjo-Dedessa Dam breach is extracted using a Digital Elevation Model (DEM) of resolution 30m by 30m. This is done by the incorporation of ArcGIS 10.0 and its extension tool HEC-GeoRAS 10.0 along the river up to 120 kilometres downstream of the dam.

A Digital Elevation Model (DEM) is a digital file consisting of terrain elevations for ground positions at regularly spaced horizontal intervals. Its uses range from scientific, commercial, industrial, and operational to military applications ((USGS Department of the interior, 2010). In the academy or a research institution, DEM is used primarily as an input or as a data source itself in studies along the fields of climate impact studies, water & wildlife management, geological & hydrological modelling, geographic information technology, geomorphology & landscape analysis, mapping purposes, & educational programs.

The study area ground surface is represented in ArcGIS 10.0 as a vector format. The vector format model stores data as a series of triangulated points forming a continuous network of triangles. This vector format is referred to as triangulated irregular network (TIN).

A TIN format is the data storage as a basis for river hydraulics and the ground surface is more accurately described by a minimum of data.

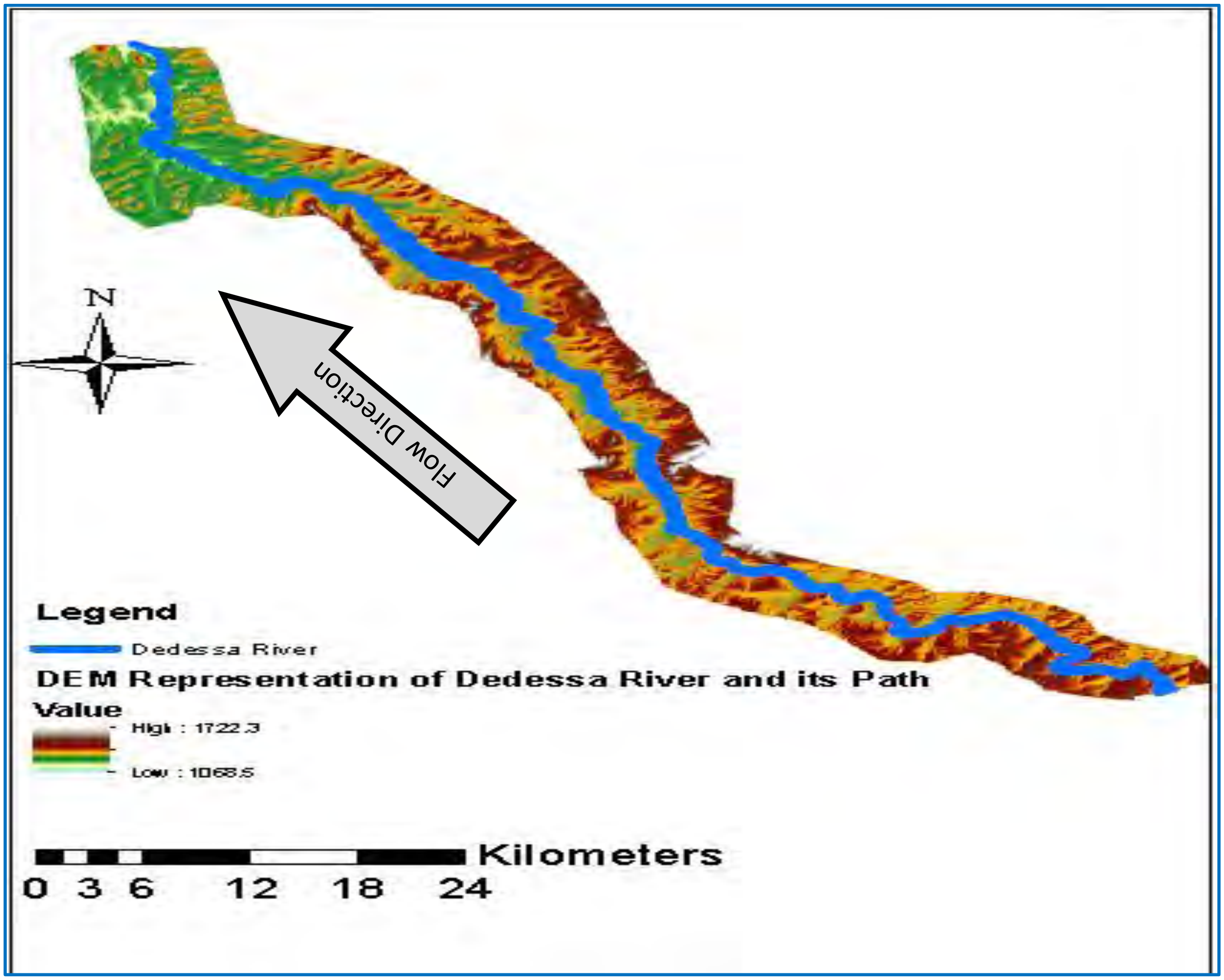


Figure 3-8: Down Stream DEM Representation of Dedessa River (Developed by ArcGIS, 2010)

3.2.3. Hydrologic Data

The hydrological data such as inflow hydrograph, maximum probable flood (PMF) and reservoir capacity used for hydraulic modeling in this study were collected from Oromia Water Works Design and Supervision Enterprise, OWWDSE. The analyses of rainfall and flood frequency for this project were also done by OWWDSE.

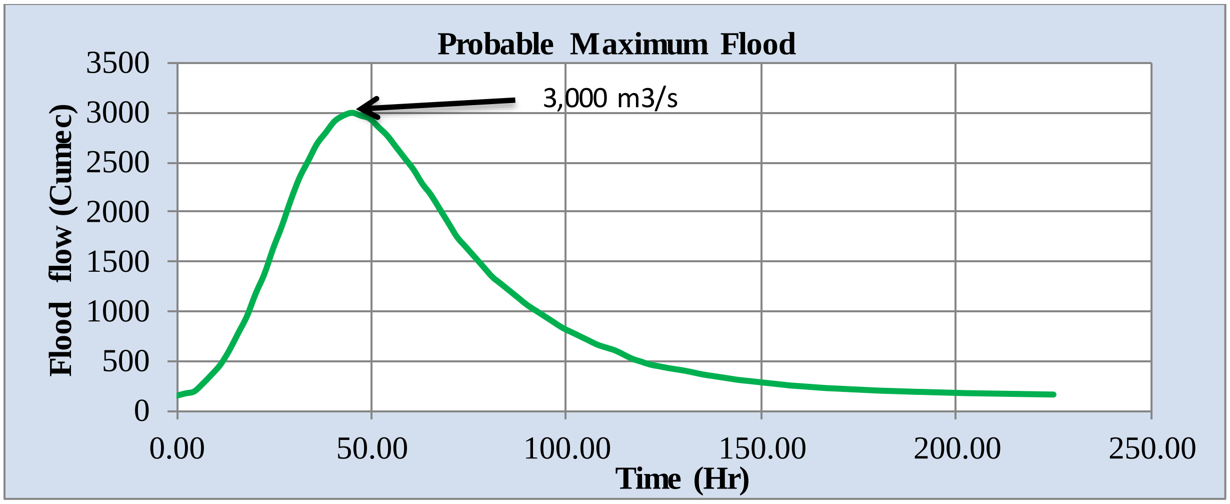


Figure 3-9: Flood Hydrograph for Probable Maximum Flood for Dedessa River (OWWDSE, Dec 2013)

Table 3-1: Spillway Capacity vs. Flood Events of Arjo-Didessa Dam (OWWDSE, December 2013).

Dam	**Spillway Design Capacity(m^3/s)**	**Peak Flow Rates (m^3/s)**			
		0.6 PMF	**10,000-yr Event**	**0.75 PMF**	**PMF**
Arjo-Dedessa	1515	1800	1948	2250	3000

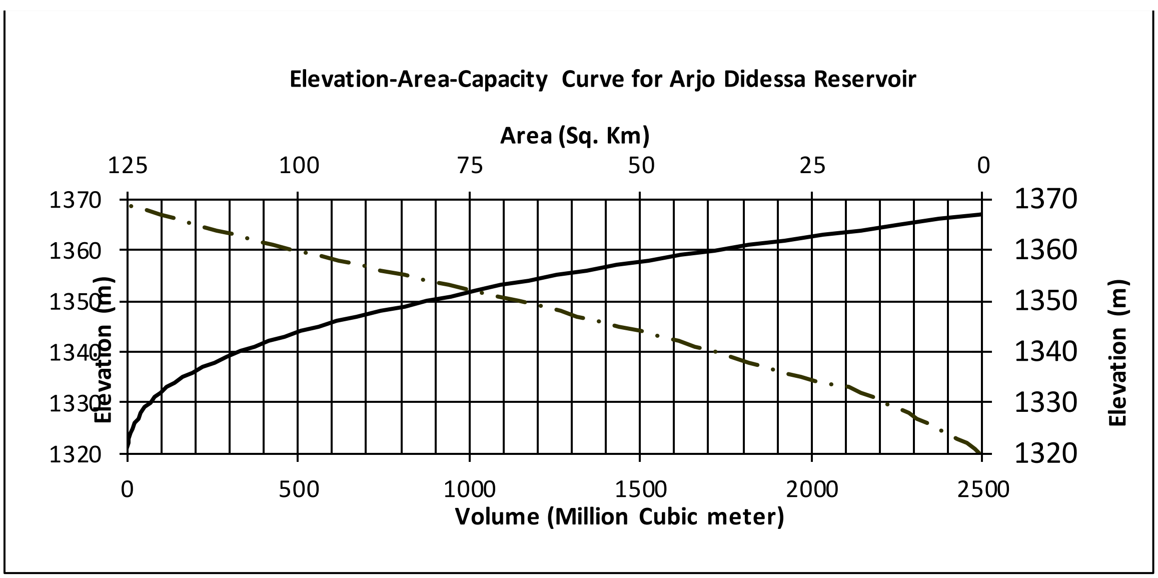


Figure 3-10: Elevation-Area-Capacity Curves of Arjo Dedessa Reservoir (OWWDSE- Dec, 2013)

3.3. Downstream Command Information

The stored water behind Arjo-Dedessa Dam will be diverted into canal system on both the right and left bank for providing water through a network of canal system for irrigation for command area of 80,000 hectare irrigable land. This irrigable command area is located at a distance of about 40 to 60 km downstream Arjo Dedessa Dam. It is found just downstream of Nekemt Bedelle Road Bridge in the right and left side bank of the river. The left bank command area covers an irrigable area of 43,030 hectare whereas the right bank command area covers 36,970 hectare (OWWDSE, 2013).

3.4. Population at Risk

East Wollega zone is one of the 17 zones of Oromia regional state with a population of 1,230,402 of 615,641 females. Eighty six percent of the populations live in rural areas (1,061,120) (FDRE, 2008)

According to the report of ''The 2007 Population and Housing Census of Ethiopia: Statistical Report for Oromia Region; Part IV: Population Size of Kebeles'', the statistical data on sex and urban/rural distribution of kebele population with households and housing units in the region are presented. In this report, the population size, number of households and number of housing units in East wollega zone of Diga Woreda kebeles also listed. Depending on this report, the population size, number of households and number of housing units of Arjo Gudatu town which is located at downstream of Arjo Dedessa Dam at a distance of 90km also known. Arjo Gudatu town has one kebele and there are also villages around the town. This town is located along the main road to Gimbi and Assossa within the low land agro ecology at altitudes of 1350-1450 m.a.s.l. (Dereje D., 2014), where the floods from Arjo Dedessa Dam breach may affect in case of the failure.

From the report of FDRE 2008, the population size of this town was 1,600 in both sexes, from which 757 were male and 843 were females. Additionally, 423 households and 410 housing units were found in this town.

Therefore, for the dam breach modeling and downstream risk analysis in the case of Arjo Dedessa Dam, the population size, number of households and housing units are the necessary conditions. Due to this population size, Arjo Dedessa Dam can be classified as High Hazard Dam

3.5. General Process of Dam Breach Modeling

In dam breach modeling and downstream risk analysis process, some procedures must be followed. First, breach parameters are estimated using three empirical equations. Then, downstream channel geometry (elevation data) is extracted from Digital Elevation Model (DEM) using HEC-GeoRAS tool in ArcMap. The preach parameter and channel geometry the, intered as an input to HEC-RAS to model dam breach. The breach outflow in then routed through downstream channel to calculate hydraulic properties at critical locations. Finally, exporting the model result to ArcMap, the flood inundation will mapped to analyze the affected critical locations. The general dam breach modeling and downstream analysis procedures are described in flow chart figure 3.11 below.

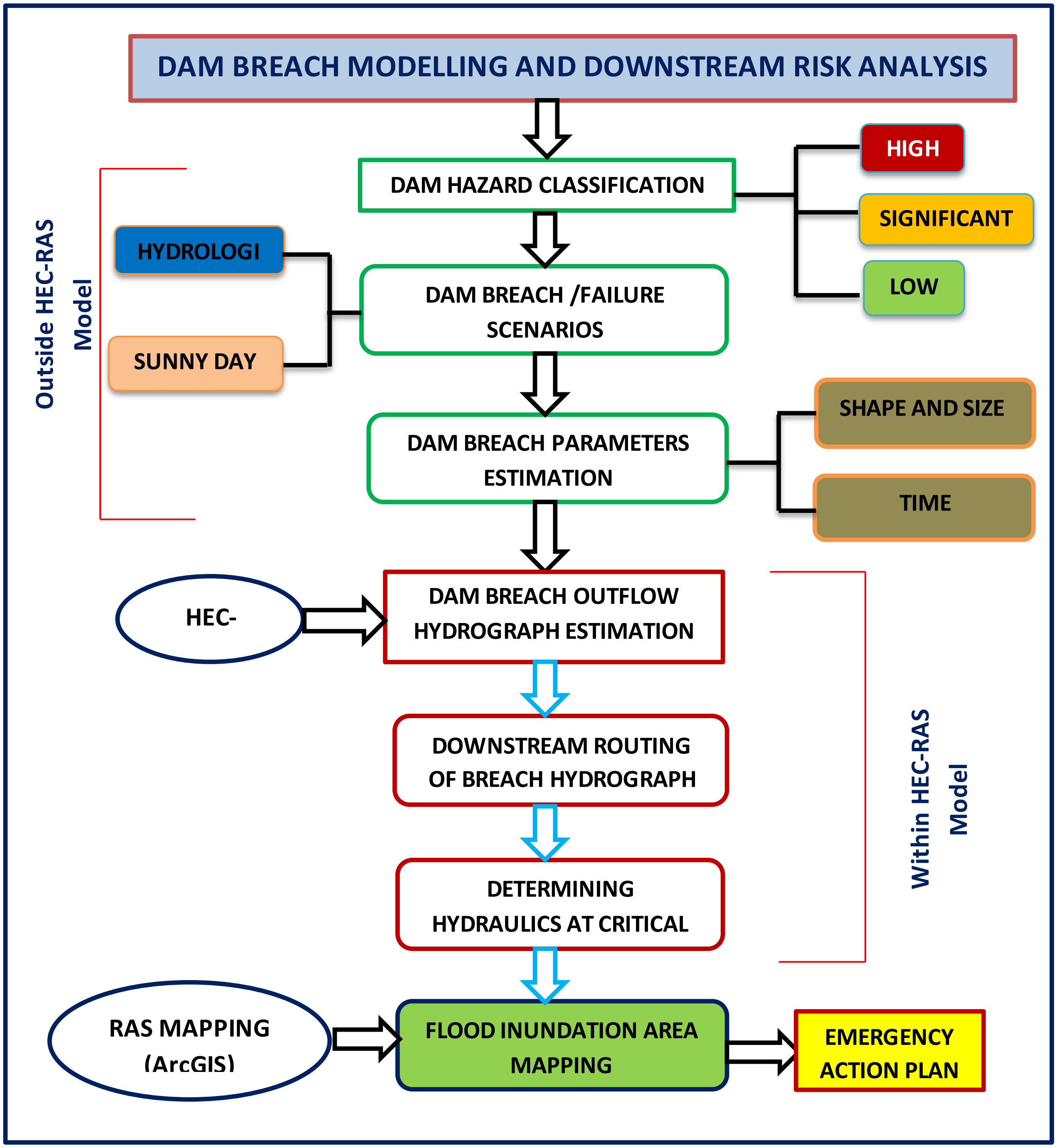


Figure 3-11: Flow diagram of General Dam Breach Modeling and Analysis Process

Arjo Dedessa Irrigation Project Dam has a reservoir capacity of 2256.30 Mm^3 at Maximum Water Level (MWL) and 1924.60 Mm^3 for Full Reservoir Level (FRL). It has also a dam height of 47m and 80,000 ha irrigable area downstream. At the downstream end of the dam, there is a small town residences and one recreation lodge. Besides these, the downstream area has different structures and population size which can be affected due to flood-flow in case the dam failure should occur. Therefore, according to the hazard potential classification guidelines and depending on the downstream population size and property damage due to dam failure flood-flow, Arjo-Dedessa Dam was classified as a "High Hazard Dam" in this study.

One-dimensional full dynamic flow routing is used for Arjo-Dedessa Dam breach analysis. This rock-fill embankment dam is modelled for overtopping and piping dam breach mode of failures. For an overtopping failure mode, the failure location is assumed at centerline of channel river bed in the downstream side of the embankment dam; whereas for piping mode of failure, the failure location assumed to be at an elevation of 1339 meters where the two right and left canal outlets were found.

The main parameters that must be estimated in this analysis are the breach formation time and the maximum size of the breach opening (breach width). These parameters are estimated by empirical (regression) equations and entered to HEC-RAS 4.1 as an input for the prediction of outflow hydrograph from the breach.

The regression equations applied for this purpose are Macdonald and Langridge Monopolis-1984, Van Thun and Gilete -1990 and Froehlich -2008.

3.6. Hydraulic Model Development

In this section, the dam breach analysis model was discussed in detail. The dam breach analysis model involves the prediction of the dam breach hydrograph and the routing of this hydrograph downstream at critical locations. For this analysis process, hydraulic modeling (HEC-RAS 4.1) was used. In addition to this hydraulic modeling, ArcMap software is used for all GIS related tasks and HEC-GeoRAS (ArcGIS extension tool) serves as the interface between GIS and the hydraulic modeling (HEC-RAS).

For the use of ArcGIS in this study, the Environmental System Research Institute (ESRI) Arc-map software version 10.0 was used. ArcMap is the main component of ArcGIS which is geospatial processing software.

In this paper, most of the ArcGIS tasks were performed using other ArcMap extension tool HEC-GeoRAS. This GIS extension tool was used to prepare a reliable model input for hydraulic model, HEC-RAS 4.1.0 within the ArcMap software environment. In addition to this, HEC-GeoRAS was used to visualize hydraulic modeling results in the form of inundation depth maps.

3.6.1. HEC-GeoRAS Development

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 (HEC, 2009). HEC-GeoRAS.10. which is compatible with Arc Map v.10 is used in this study in creating RAS Layers in ArcGIS to extract information essential for hydraulic modelling (HEC-RAS). It is used to extract elevation data from DEMs in the TIN format.

For developing each of the RAS layers, DEM were projected into a coordinate system. The stream centreline layer, bank lines layer, flow path layer and cross-sectional cut line layer were created. The development of all other RAS Layers is optional based on the data needs for the river hydraulics model.

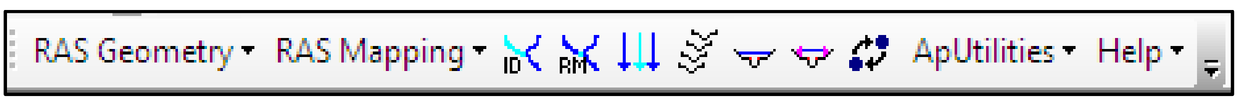


Figure 3-12: HEC-GeoRAS Tool Bar Used in ArGIS v10.0

The Stream Centreline 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, and other structures 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. The bank lines and flow path centrelines were created before laying out cut line locations.

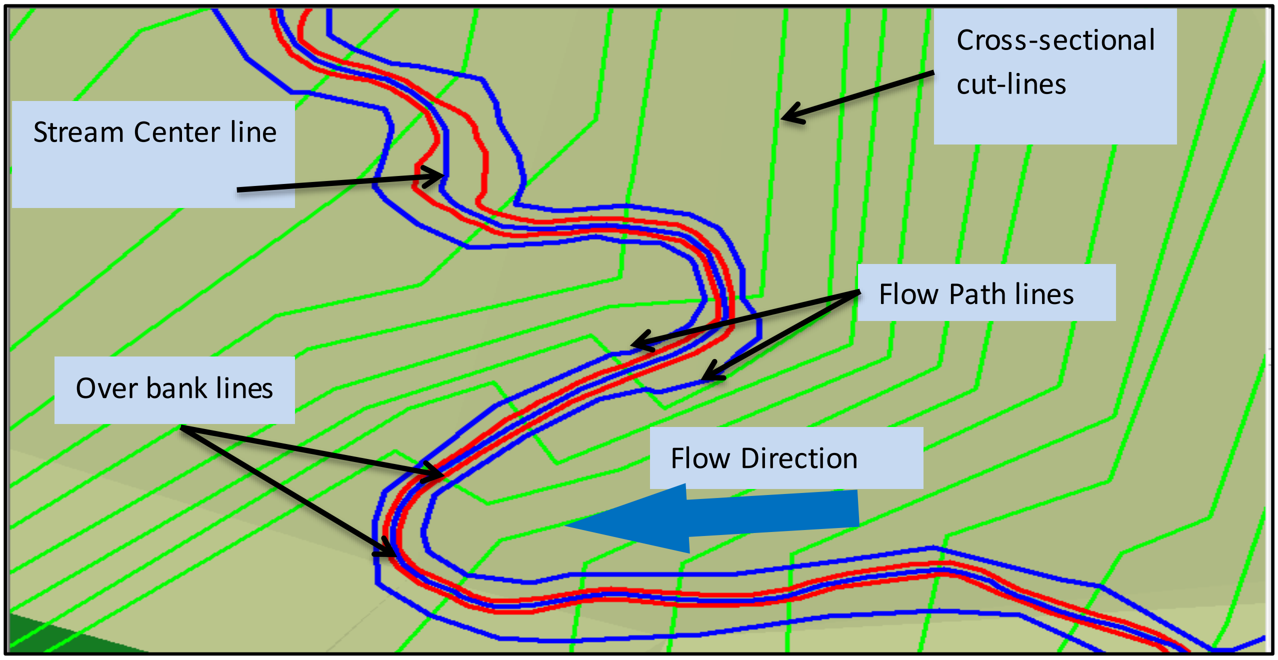


Figure 3-13: Arjo-Dedessa River Geometry developed by HEC-GeoRAS tool in ArcGIS v 10.0

After these RAS Layers were created, GeoRAS tools and menus were assigned and populated attribute data. Lastly, after all the attribute were completed, data were written out to the HEC-RAS geospatial data exchange format and were imported into HEC-RAS 4.1.0.

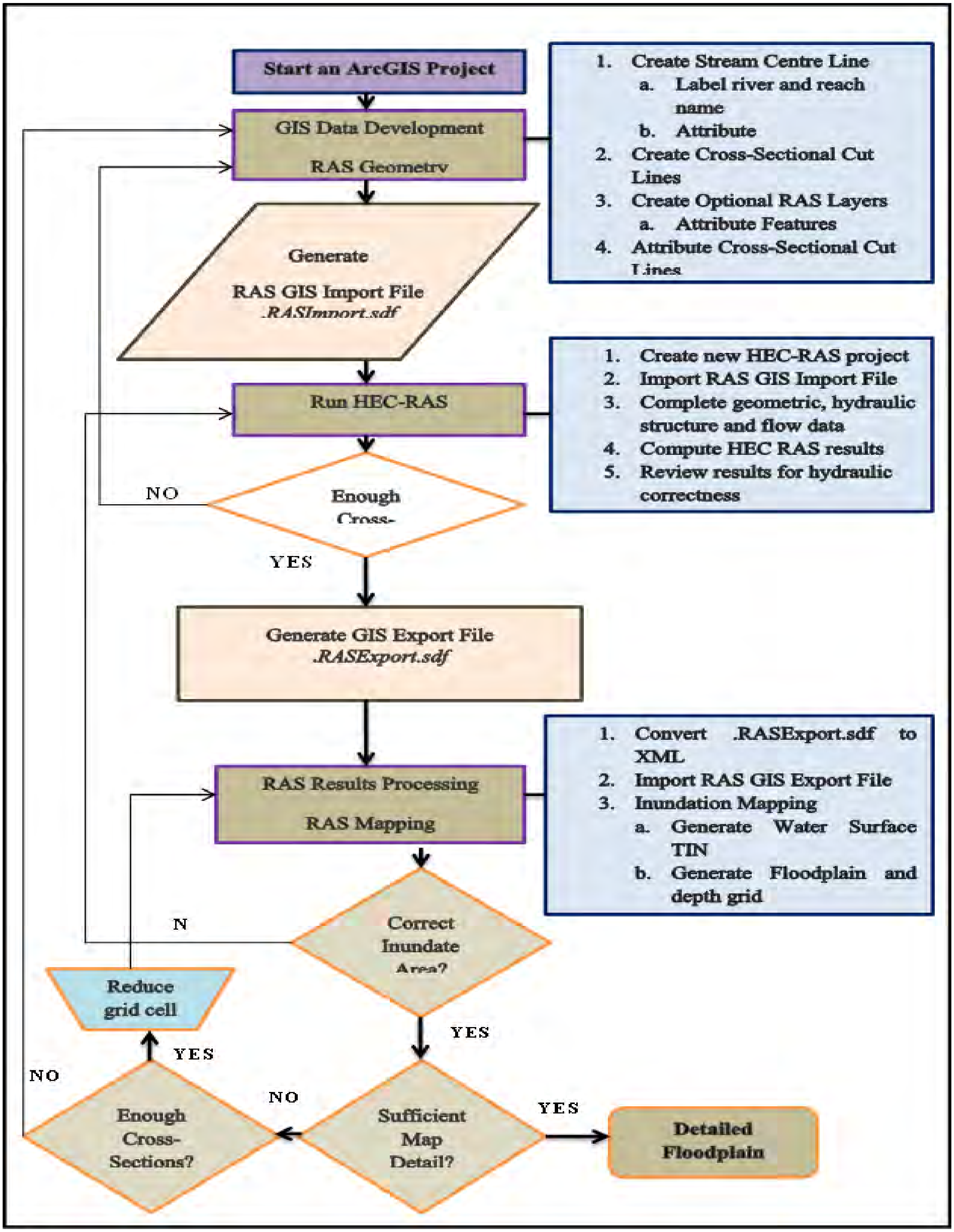


Figure 3-14: Process flow diagram for using HEC-GeoRAS (Adapted from: HEC-2009) *(Ackerman, 2009)*

3.6.1.1. Cross Sections

Cross sections are used to define the shape of the stream and its characteristics, such as roughness, expansion and contraction losses, and ineffective flow areas. HEC-RAS requires cross-sections for water surface computation. Approximately 300 cross sections were created for the model using HEC-GeoRAS at about 400m average spacing. Elevation data then were extracted from Digital Elevation Model (DEM) and imported to HEC-RAS. Each cross-section ideally is perpendicular to flow direction, and can intersect the main channel only once and may not intersect another cross-section. The cross sections were located to adequately describe geometric features such as roughness changes, grade breaks, expansions and contractions. The cross sections were drawn to remain perpendicular to the expected maximum flood wave flow line as shown in figure 3.15.

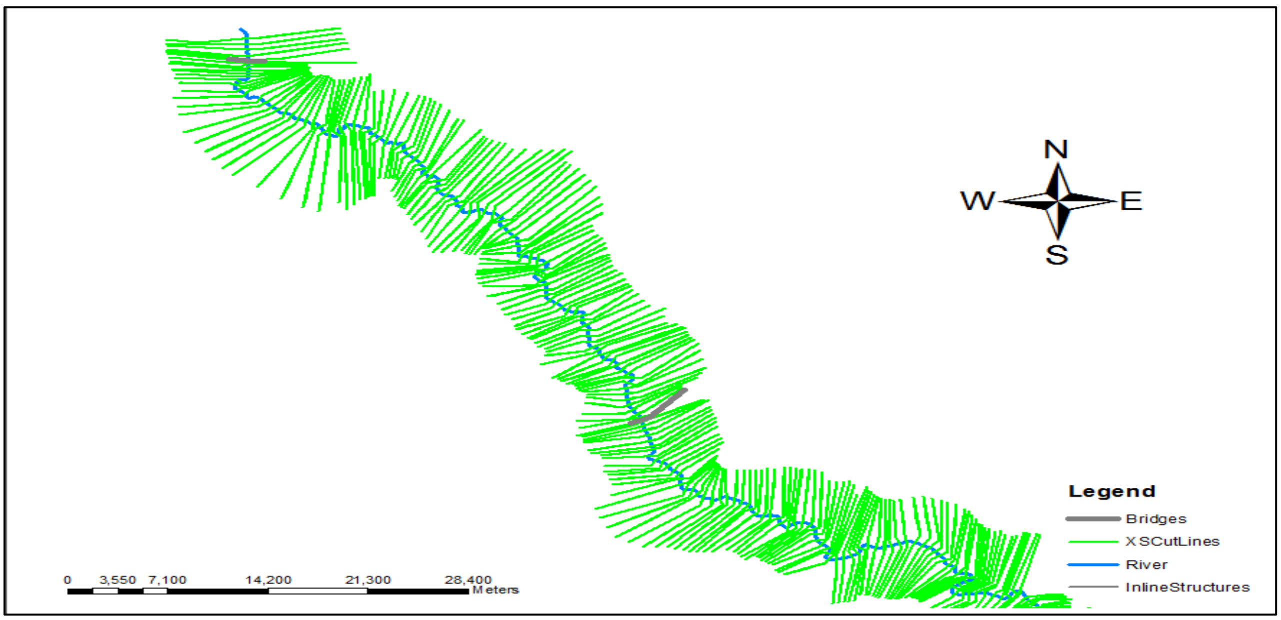


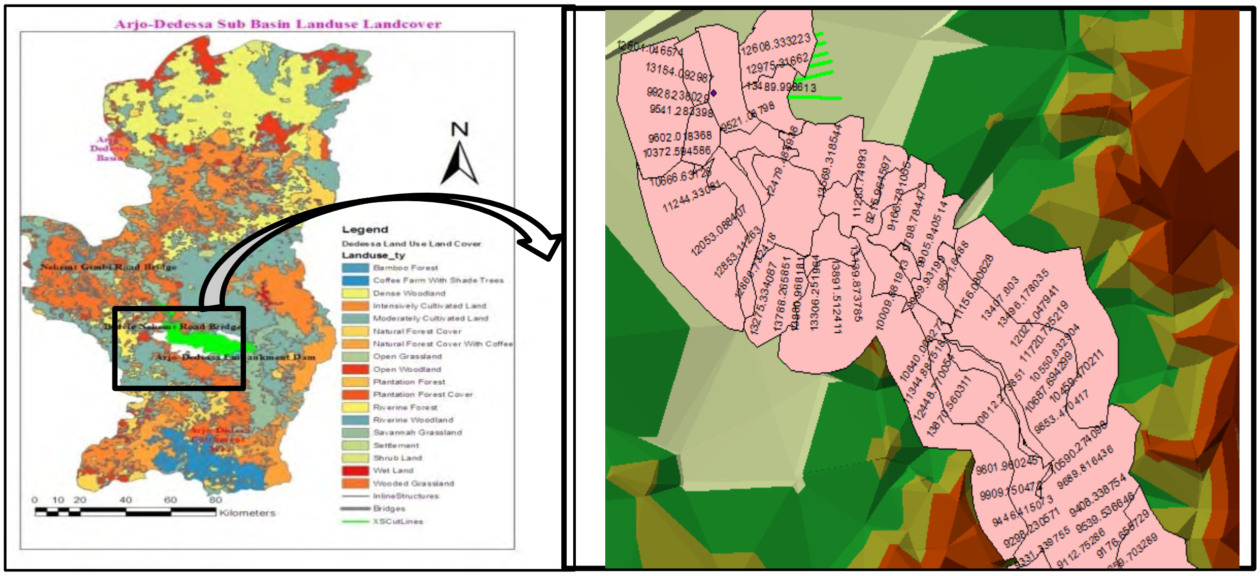
Figure 3-15: River Cross-sections Downstream of the Dam (Developed by HEC-GeoRAS)

3.6.1.2. Roughness Values

Estimation of Manning’s roughness coefficient (or Manning’s n) is very important to simulate open channel flows. As an empirical parameter, the roughness coefficient actually includes the components of surface friction resistance, form resistance, wave resistance and resistance due to flow unsteadiness (Y.WANG, 2005). Direct determination of the roughness coefficient is almost impossible in studying natural river flows, including unsteady channel network flows. The roughness coefficient (n) in natural channels is difficult to determine in field. Various factors affecting the values of roughness coefficients were presented by (Chow V. T., 1959). The friction slope may thus be seen as a very important parameter whose value must be chosen very carefully. From the assessment of land use land cover, major land cover types identified include moderately cultivated, dense woodland, intensively cultivated land, wooded grassland, open woodland, natural forest cover, natural forest with coffee, coffee farm with shade trees, riverine forest, bamboo forest, plantation forest, settlement, shrub land and open grassland (OWWDSE O. W.,

Figure 3-16: The main Land cover and land use of Dedessa River downstream of the dam

Depending on these characteristics of flood plain, the manning's n values for the stream channel downstream of the Arjo-Dedessa dam ranges from 0.03 to 0.04 to reflect the dynamic and extreme nature of a dam breach flood wave as well as different material types within the channel. The left and right overbank n values also ranges from 0.03 to 0.15 reflecting riverine forests, Savannah Grass land, moderately cultivated, etc. along the flood path. Manning's n-values were based on published values for similar conditions (CHOW, 1959).



(a) (b)

Figure 3-17: a) Land Use and Land Cover Map (b) Extracted Manning's Value for the Study Area

3.6.2. HEC-RAS Development

3.6.2.1. General – Purpose of the Model

In this study, HEC-RAS modelling developed by the U.S. Army Corps of Engineer´s Hydrologic Engineering Centre version 4.1.0 was used. This software has a capability of performing one-dimensional (1-D) steady and unsteady-flow simulations and comprises a graphical user interface. HEC-RAS also has the capability of modelling dam breach events under a wide range of scenarios.

Cross sections, stream centrelines, and other geometric features of the stream were extracted from GIS using HEC-GeoRAS and ArcGIS. Dam failure scenarios were analysed for the Sunny Day or Non-Hydrologic and Hydrologic events. The objective of this modelling effort is to evaluate the impact of a dam breach on the downstream population and property damages.

In this study, the one dimensional unsteady flow analysis of HEC-RAS model was discussed for downstream dam breach outflow hydrograph routing along Dedessa river channel.

HEC-RAS 4.1. is used to model overtopping and piping failure breaches for earthen dams. The resulted flood wave is routed downstream using unsteady flow equations. Inundation mapping of the resulting flood can be done with the HEC-GeoRAS when GIS data (terrain data) are avilable.

Dams are modelled within HEC-RAS by using the inline structure Editor. It allows to put in an embankment, define overflow spillways and weirs. The upstream reservoir of the dam can be medelled with cross-sections or by using a storage area. In using cross-sections, HEC-RAS will perform full unsteady flow routing through the reservoir and downstream of the dam whereas in using a storage area, HEC-RAS uses level pool routing through the reservoir then unsteady flow routing downstream of the dam.(HEC-2010).

Full dynamic wave (unsteady flow) routing through the reservoir pool which was performed for Arjo-Dedessa Dam breach analysis is the most accurate methodology. To model the Arjo- Dedessa Dam reservoir using full dynamic wave routing with HEC-RAS 4.1, the pool with one dimensional cross section throughout the reservoir was modelled. The Arjo-Dedessa Dam was modelled with the Inline Structure in HEC-RAS 4.1.

3.6.2.2. External Boundary Condition

For the solving of the unsteady-flow equations in HEC-RAS, boundary conditions need to be defined. In the case of subcritical flow, boundary conditions in the form of a specified flow are required at the up- and downstream end of the modeled reach. For the routing of a flood hydrograph through a river reach, the upstream boundary condition is the flood hydrograph (HEC-RAS 4.1, 2010).

External Boundary Condition (EBC), which is defined at the most upstream and downstream ends of the river system. An EBC can have either an inflowing or outflowing river reach connected to the node. An external boundary includes an inflow hydrograph, a stage hydrograph, a rating curve or normal depth.

There were two external boundary conditions for Arjo-Dedessa dam breach analysis in this paper. These are upstream and downstream boundary conditions. For unsteady flow models, upstream boundary conditions are typically input as discharge hydrographs. This input hydrographs was a PMF flood event. Downstream boundary condition for Arjo-Dedessa was set to normal depth slope of 0.0034 (0.034 percent slope) about 4 kilometres downstream of Nekemt Gimbi Road where the flood is entered to large gorge.

3.6.2.3. Internal Boundary Condition

Internal Boundary Condition (IBC), which is defined at internal nodes, when-ever two or more reaches meet. It is a node without a hydraulic structure which is used to connect two or more river reaches with different roughness or bed slopes, or canal expansion occurs. This node is denoted as a junction boundary condition.

For Arjo Dedessa Dam breach modeling, the inflow hydrograph of the Wama River was entered or attached to Arjo-Dedessa River at river station f 111423.6 which is 5 kilometers downstream of the dam for the downstream unsteady flow routing. It was entered as lateral inflow hydrograph or internal boundary condition.

3.6.2.4. Initial Condition

The initial conditions in the unsteady routing model for river reach in the steady case can be specified by two of the variables describing the reach's the water stages at the reach's end, or one of the stages and the flow discharge in the reach. In the case that the simulation starts from unsteady conditions, the initial conditions must be defined as the combination of the stages at the reach's end and the flow at one of its ends, or the flows at the reach's ends and the stage at one of its ends.

There were two initial conditions that entered in hydraulic modeling, HEC-RAS for unsteady flow analysis of Arjo-Dedessa Dam breach. These are Initial flow and initial elevation of water level in the reservoir at the time of starting breach. The initial flow estimated as the design capacity of the spillway, since the dam breach is analyzed for the worst case flood. While, the initial water level was taken as the elevation of maximum pool level (crest of the dam).

3.6.2.5. Model Limitation

The assumption that the flow in the river channel as well as in the overbanks only occurs in the direction of the longitudinal channel axis limits the applicability of the model to situations, where this assumption is true to a large extent. 1-D flow further implies that for any cross section along the modeled reach, the velocity is constant and the water surface is horizontal. Due to the complex geometry of the alluvial cone that hosts the watercourse that is modeled in this study, the suitability of the 1-D HEC-RAS model for this situation is limited. Since bank overtopping in an alluvial channel results in overbank flows that spread out over the alluvial cone, the principle of 1-D flow is violated.

Additionally, HEC-RAS Model has a limitation of labor intensive, time consuming process and stability problems may arise during the analysis.

3.7. Floodplain Mapping

In this section the description of the aerial extent of flooding downstream area for Arjo-Dedessa dam breach analysis was analysed. For this study, two failure mode scenarios were used; i.e overtopping and piping modes of failure. The reservoir conditions, normal pool (spillway crest level) and maximum storage elevation (top of the dam crest) were considered for downstream consequence of Arjo-Dedessa Dam breach.

Once a dam breach simulation completed in HEC-RAS 4.1.0, it was exported to ArcGIS and stored for the next inundation delineation process. ArcGIS tool USACE HEC-GoeRAS v.10.0 Arc Mapp extension and RAS Mapper automatically delineate flood plains or inundation as a post processing function of HEC-RAS 4.1.0

Floodplain mapping for the downstream of Arjo-Dedessa Dam was performed using the water surface elevations on the XS cut lines, within the limits of the bounding polygon. A polygon refers to a TIN (Triangulated Irregular Network) layer in ArcGIS that will define a zone that will connect the outer points of the bounding polygons.

4. BREACH PARAMETER ESTIMATION AND MODEL SETUP

4.1. Introduction

The events being evaluated in this paper are a hydrologic or PMF and a sunny day or non-hydrologic event failure mode. The necessary information required for the Arjo-Dedessa dam breach estimations are given below:

Reservoir Data:

Table 4-1: Different Reservoir Levels and Volume Capacity

Important Pool Levels	Elevation(m)	Volume(Million m^3)
Stream Bed Level	1312.00	0.00
Full Supply Level	1354.00	1924.60
Top of Flood Control	1357.00	2256.30
Top of Dam Crest Level	1359.00	2491.60

Dam Embankment Data:

The crest length and width of the dam are 502.473m and 10m. Maximum Height of the dam above river bed is 47m. The average Upstream and downstream embankment Slopes are 3.125H: 1V and 2.625H: 1V respectively. The embankment material of the dam is rock-fill embankment dam with impervious clay core. The upstream and downstream slope face of the dam are protected by dumped rock riprap with filter and 300mm thick hand placed riprap without filter layers.

4.2. Estimating Dam Breach Parameters

Two modes of failures were analysed for the study of Arjo-Dedessa Dam breach analysis. Dam breach parameters were estimated using three regression (empirical) equations. These are: - MacDonald & Langridge Monopolis (1984), Van Thun and Gillette (1990) and Froehlich (2008), are the most commonly applicable regression equations in current dam breach parameter estimation process.

4.2.1. Overtopping Mode of Failure

In this study, the failure location for overtopping failure is assumed to be at main channel centerline (at an elevation of 1312.0 meters). From the calculation of breach parameters using three methods used in this study, the following results were obtained.

Using Macdonald and Langridge Monopolis (1984), the breach bottom width and breach development time results were 382.57m and 3.9 hrs. Using Von Thun and Gillette (1990), these values were 172.4m and 1.19hrs. Finally, using Froehlich (2008) 415.97m and 5.95hrs.

4.2.2. Piping Mode of Failure

Using Macdonald and Langridge Monopolis (1984), the breach bottom width and breach development time results were 371.57m and 3.5hrs. Using Von Thun and Gillette (1990), these values were 159.9m and 1.09hrs. Using Froehlich (2008) 293.28m and 5.85hrs.

Table 4-2: Summary of Breach Parameters Estimation

Method	Breach Bottom Width (m)	Breach Side Slope (H:1V)	Breach Failure Time (Hrs.)	Peak Outflow $(Q_{max}, \frac{m^3}{s})$
Overtopping Case				
Maccdonald and Langridge Monopolis (1984)	382.57	2:1	3.9	41,923.66
Von Thun and Gillette (1990)	172.4	0.5:1	1.19	_
Froehlich(2008)	368.97	1:1	5.95	415,991.97
Piping Case				
Maccdonald and Langridge Monopolis (1984)	371.57	2:1	3.5	2600.00
Von Thun and Gillette (1990)	159.9	0.5:1	1.09	_
Froehlich(2008)				
At an elevation of 1354	251.28	1:1	5.85	247,463.94
At an elevation of 1359	272.98	1:1	5.85	319,613.69

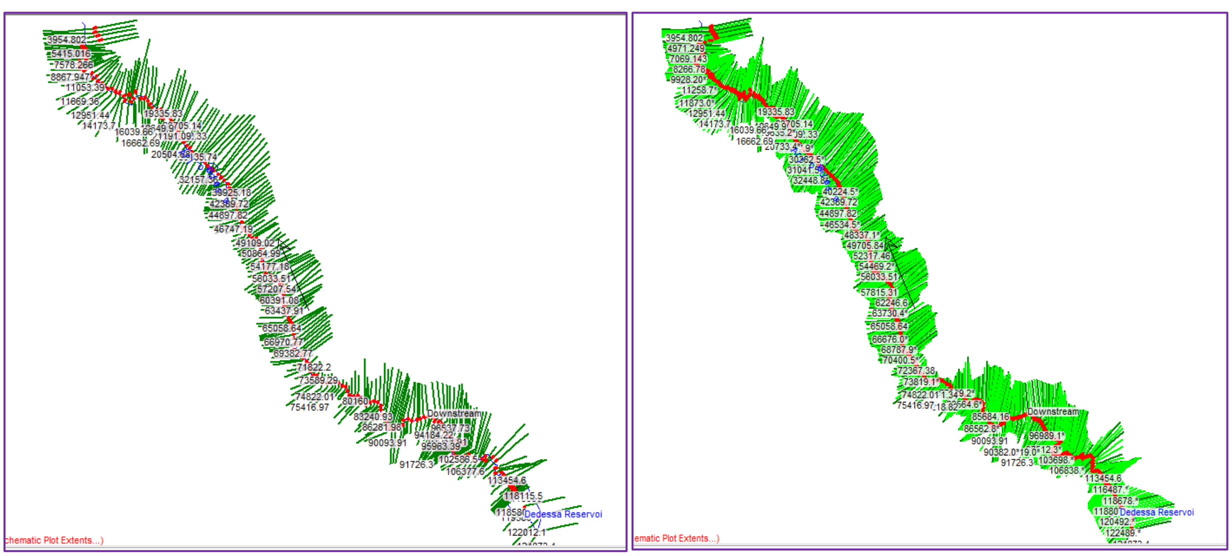
4.3. Representation (Setup) of Dedessa River and Dam in HEC-RAS Model

4.3.1. Arjo-Dedessa River

The Dedessa River Basin downstream of the of the Arjo-Dedessa Dam is moderately steep, dropping 203m in the approximately 120 km distance to the downstream of the river at Nekemt Gimbi road bridge. The river flows through several deep and zigzagging valley sections interspersed with wide, flat reaches over this interval. These aspects of the river make the dam breach modelling problem challenging as it is difficult to determine the appropriate number and placement of cross-sections for the model as well as the best model time step for the simulations.

The stability of the HEC-RAS model is function of the distance between the cross sections and the time step used in the simulation. The minimum value allowed by HEC-RAS is one second. A one minute time step is used for the Arjo-Dedessa model to achieve stability of the calculations. Other, larger time steps were tested, but time steps greater than three minutes not found to result in stable calculations.

New cross-sections with a shorter distance between them are interpolated between the initial sections at locations where the HEC-RAS solution became unstable. First, all of the initial river cross-sections were interpolated to 600m and then further interpolation to 150 m is carried out.



(a) (b)

Figure 4-1: (a) Initial cross-sections; (b) Interpolated cross sections for Dedesa River HEC-RAS model.

4.3.2. Arjo-Dedessa Dam Breach Data

HEC-RAS allows the modelling of the breach process by entering key data and assumptions regarding the dam, the reservoir and the breach most probable characteristics, as shown in Table 4.3 below. The breach formation time for Arjo-Dedessa embankment dam was estimated to be 5.95 hours for overtopping mode of failure (Froehlich equations). The bottom width of the breach is 368.97 m. In order to have a stable model, the river was considered wet at the beginning of the simulation with an initial flow of 1 m3/sec for HEC-RAS model.

Table 4-3: Arjo-Dedessa Dam breach model data for overtopping mode of failure.

Item	Value
River station of Dam	118615 (118.6 kms upstream from Nekemt Gimbi Road)
Pilot Flow	1 m3/s (the river was considered wet at the beginning of the simulation
Centre station	1298.6 m
Final Bottom Width	368.97
Final Bottom Elevation	1312 m
Left side slope	1:1
Right side slope	1:1
Full formation time	5.95 hrs.
Failure mode	Overtopping
Trigger Failure	WS Elev.

4.4. HEC-RAS Unsteady Flow Analysis Parameters

To model the dam breach process in HEC-RAS, an unsteady flow calculation is performed. A simulation period of 24 hours was used with the dam breach initiated at the start of the simulation. The Arjo-Dedessa Dam is modeled as an “inline structure” in the HEC-RAS model. The dimensions of the dam are known from the plan Fig.4.2 and the data in Table 4.3

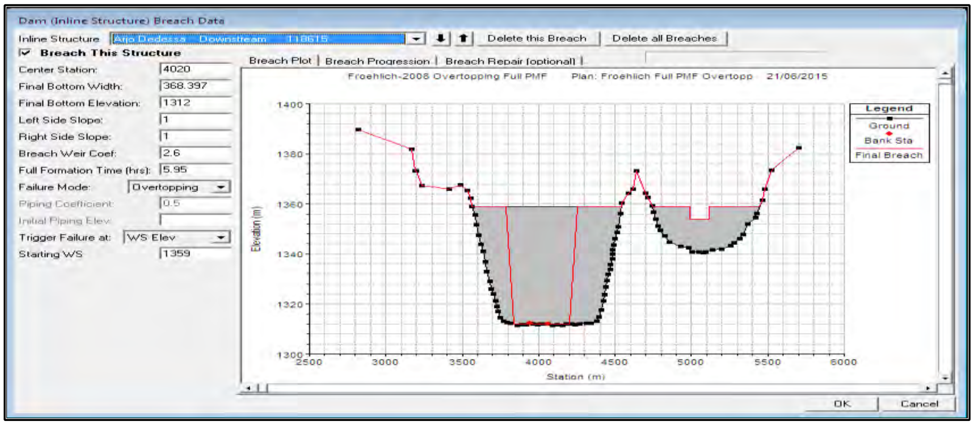
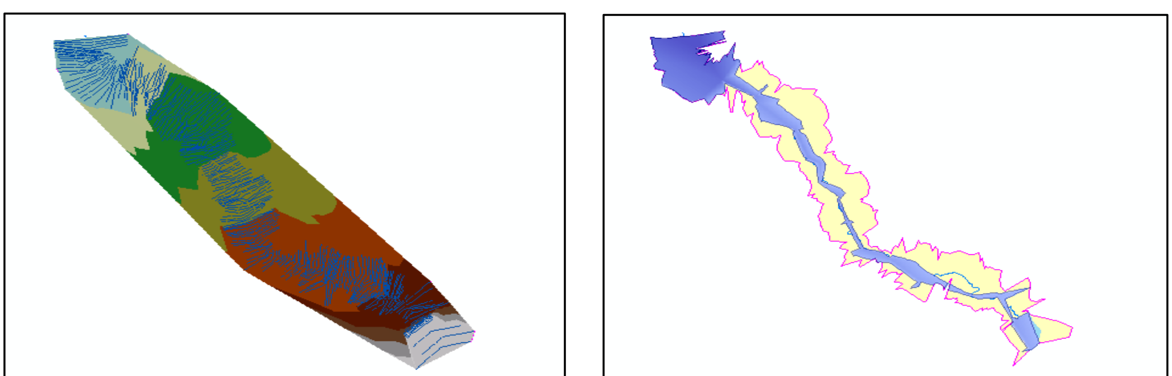


Figure 4-2: Dam Breach Plan of Arjo-Dedessa Dam, on Dedessa River (HEC-RAS 4.1.)

Boundary condition and initial condition data must be entered for the unsteady flow analysis. Boundary conditions at the upstream river reach above the dam was entered as inflow hydrograph of the Dedessa River. The downstream boundary condition at immediately downstream of Nekemt Gimbi Road was as a normal depth boundary condition with a slope of 0.0034 m/m. Also, the initial flow in the basin at the start of the simulation period was set to1515 m3/sec for the Dedessa River and 962.5 m^3/s for Wama Rive

4.5. Flood Inundation Mapping Process

The PMF and fair weather (Maximum Pool) scenarios maximum water surface elevations obtained from the HEC-RAS model were combined with topographic data extracted in ArcGIS to create a flood inundation map from Arjo-Dedessa Dam and downstream along Dedessa River using the ArcGIS extension, HEC-GeoRAS. Travel times for the PMF and fair weather breach were tabulated. Finally, the inundation maps for the two scenarios of dam breach would be mapped.



(a) (b)

Figure 4-3: (a) Inundation Area TIN Polygon; (b) Inundation area boundary and extent by depth

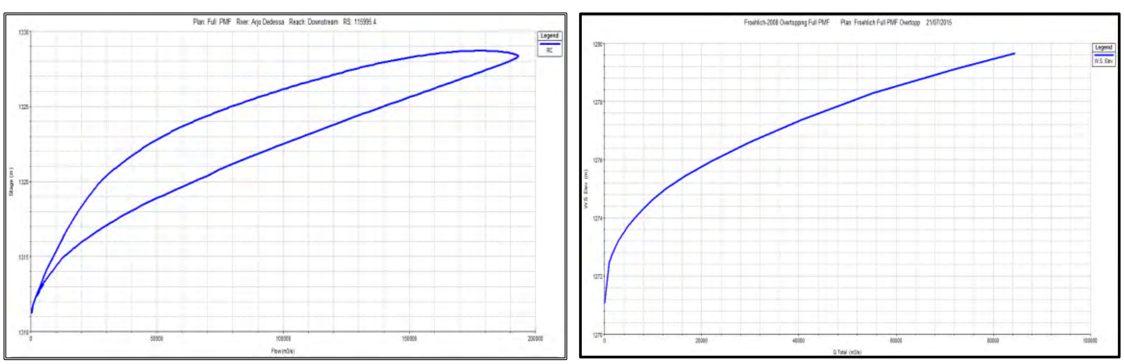
5. RESULTS AND DISCUSSIONS

5.1. Dam Breach Parameters Result

The dam breach parameters were estimated using three most currently used regression equations, i.e. Macdonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990) and Froehlich (2008). For the case of overtopping failure, the bottom breach width for three methods were 382.5m, 172.4m, and 368.97m respectively. Their breach development times were 3.9hrs, 1.19 hrs, and 5.95 hrs. Similarly, the breach width and breach development times for the above three methods in the case of piping failure were: 371.57m, 159.9m and 319.98m and 3.5hrs, 1.09hrs, and 5.85hrs respectively.

5.2. Dam Breach Simulation Result

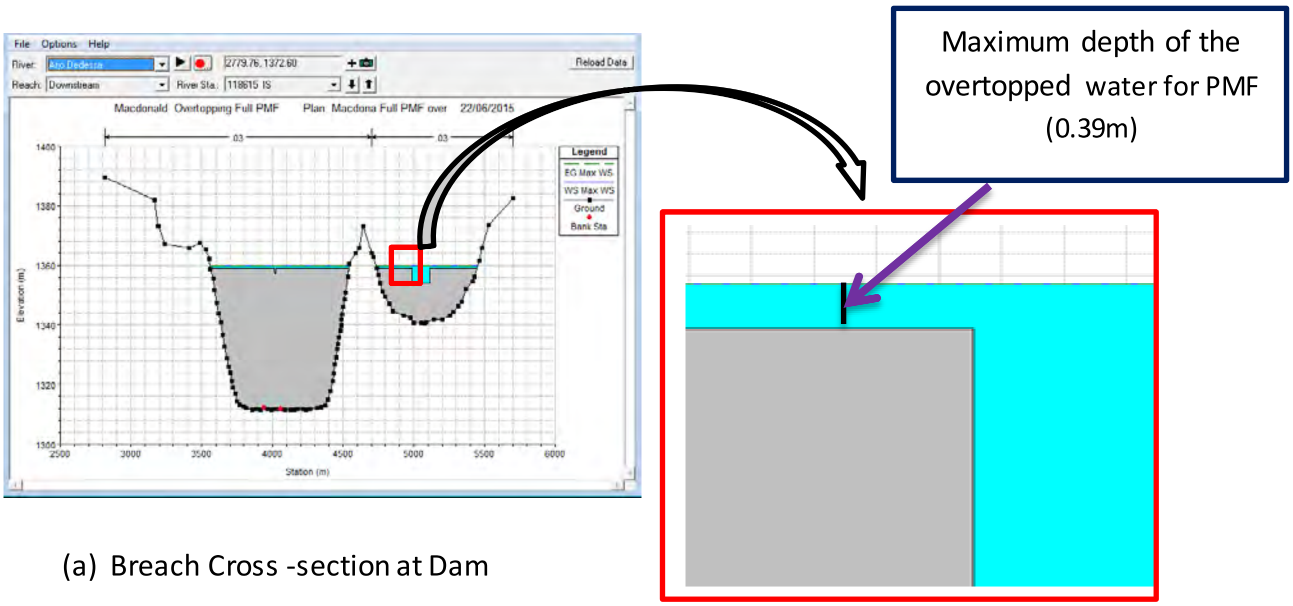
Dedessa River passes any given flow within the range of stages. The shift in stage is a result of the shifts in river meander, channel geometry or bed forms, the dynamic of the hydrograph (how fast the flood wave rises and falls); backwater (backwater can significantly change the stage at a given cross-section for a given flow); and finally, the slope of the river (flatter streams tend to have greater loops in the rating curves). Figure 5.1 below shows a looped rating curve for a single event. Within the distance of 120 km downstream of the dam, Dedessa River faces different stage shifts. Generally, the stage shift results would give greater loops in rating curve. The lower stages are associated with the rising side of a flood wave, and the higher stages are associated with the falling side of the flood wave.



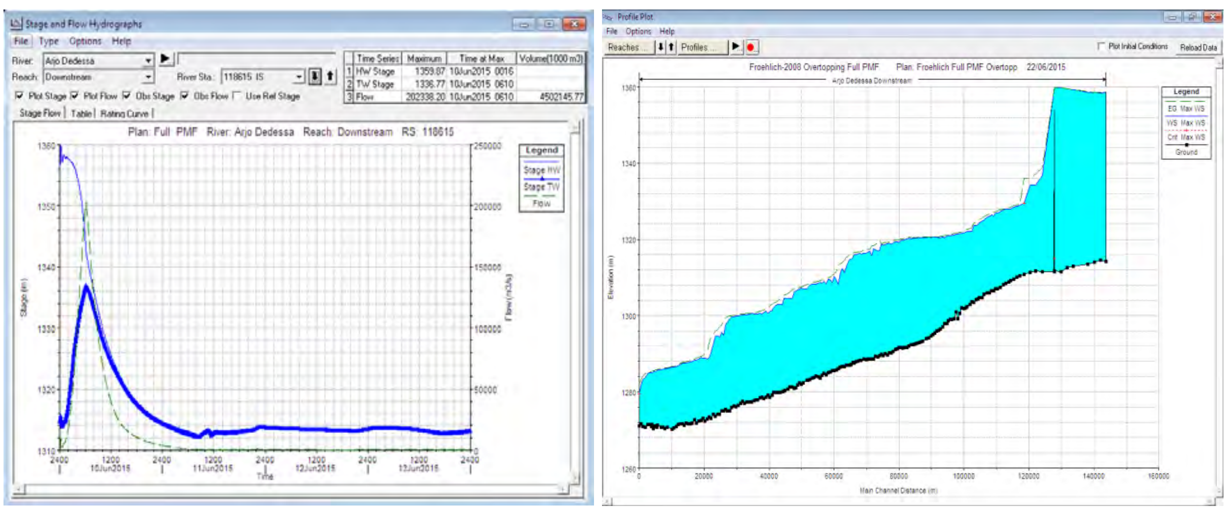
(a) (b)

Figure 5-1: Rating Curve (a) Looped (b) Normal

From the HEC-RAS model results, the breach outflow hydrographs have been computed for different failure modes (overtopping and piping) breach cases and for flood events PMF, 75% PMF, 10,000 years return period. The maximum breach outflows and time to peak flow of the two failure scenarios, i.e. overtopping and piping for Arjo-Dedessa were computed using the results of the above three methods.



(a) Breach Cross -section at Dam



(a) Breach Outflow Hydrograph at Dam (c) Water surface profile plot

Figure 5-2: Arjo-Dedessa Dam Breach Results, (a) Cross -section at Dam, (b) Breach Hydrograph at Dam (c) Water surface profile plot (HEC-RAS 4.1)

5.2.1. Overtopping Mode of Failure Results

The maximum breach outflow and arrival time to peak breach outflow results obtained from HEC-RAS model simulation at dam for three methods in case of hydrologic (PMF) breach scenario are given in tables 5.1, 5.2, 5.3, 5.4.

Table 5-1: PMF Event Breach Peak Outflows for Overtopping Mode of Failure

Methods	Peak Flow (m3/s)	Time to Peak (hrs.)
Macdonald and Langridge Monopolis (1984)	180,690.8	4.15
Von Thun and Gillette (1990)	117,585.4	1.4
Froehlich (2008)	149,503.4	6.12

Table 5-2: Maximum flow, time to peak, rate of flow and flood height for some Dedessa River stations below Arjo-Dedessa Dam (Macdonald and Langridge-Monopolis, 1984)

Stations	Peak Flow (m^3/s)	Peak Time (Hr.)	Rate of Flow (m/s)	Flood Height (m)
At dam	180,690.8	4.15	6.23	47.39
7 km	150,830.2	5.00	4.41	18.20
40 km	92,884.9	8.30	4.01	23.31
80 km	84,967.94	11.15	4.58	21.38
At 120 km d/s end of the river	78,078.23	14.45	8.45	8.02

Table 5-3: Maximum flow and time of peak flow for some Dedessa River stations below Arjo-Dedessa Dam. (Von Thun and Gillette, 1990)

Stations	**Peak Flow (m^3/s)**	**Peak Time (Hr.)**	**Rate of Flow (m/s)**	**Flood Height (m)**
At dam	117,585.4	1.40	5.14	47.78
7 km	93,884.9	3.30	3.97	14.93
40 km	65,102.22	8.40	3.50	19.80
80 km	61,975.45	11.40	4.17	18.78
At 120 km d/s end of the river	58,818.01	15.30	7.80	7.11

Table 5-4: Maximum flow, time to peak flow, and flood depth for some Dedessa River stations below Arjo-Dedessa Dam (Froehlich-2008)

Stations	**Peak Flow (m^3/s)**	**Peak Time (Hr.)**	**Rate of Flow (m/s)**	**Flood Height (m)**
At dam	149,503.4	6.12	6.45	47.87
7 km	129,884	6.50	4.26	17.17
40 km	85,603.47	10.22	3.90	22.46
80 km	79,327.19	13.10	4.47	20.86
At 120 km d/s end of the river	73,587.95	16.42	8.31	7.82

Arjo-Dedessa Dam will overtop in the case of full PMF, 0.75 PMF, events by 0.84m and 0.56m, 0.75m and 0.54m, and 0.87m, and 0.59 m respectively by using the breach parameters obtained from the above three methods in the model.

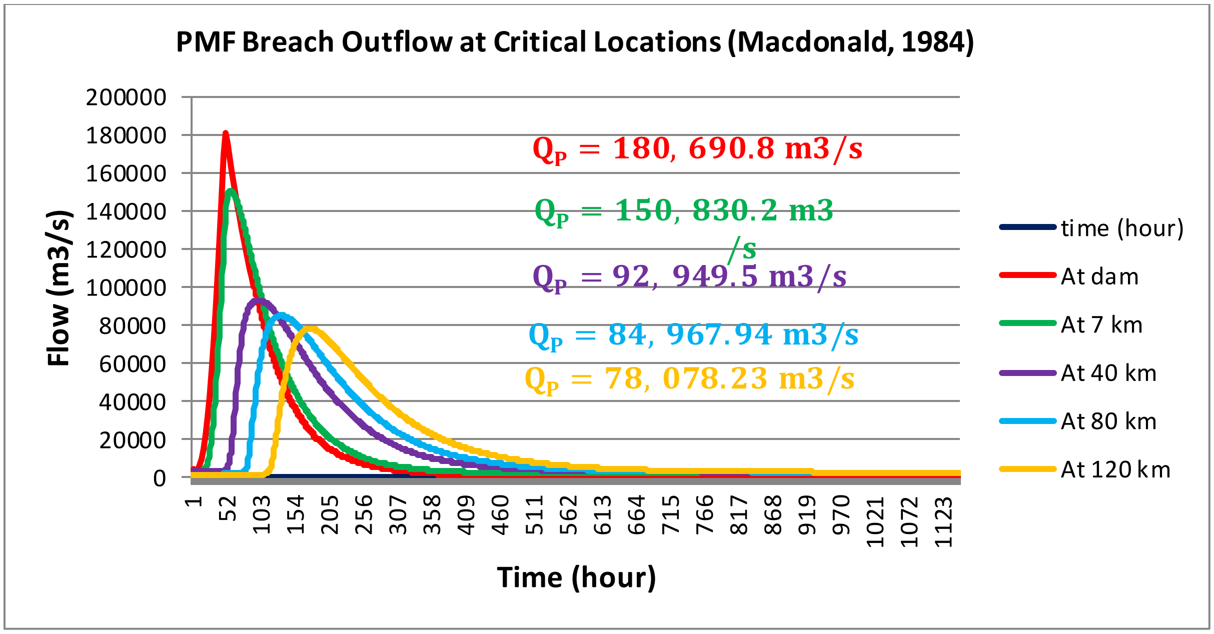


Figure 5-3: Breach outflow hydrograph at downstream critical locations (Macdonald, 1984)

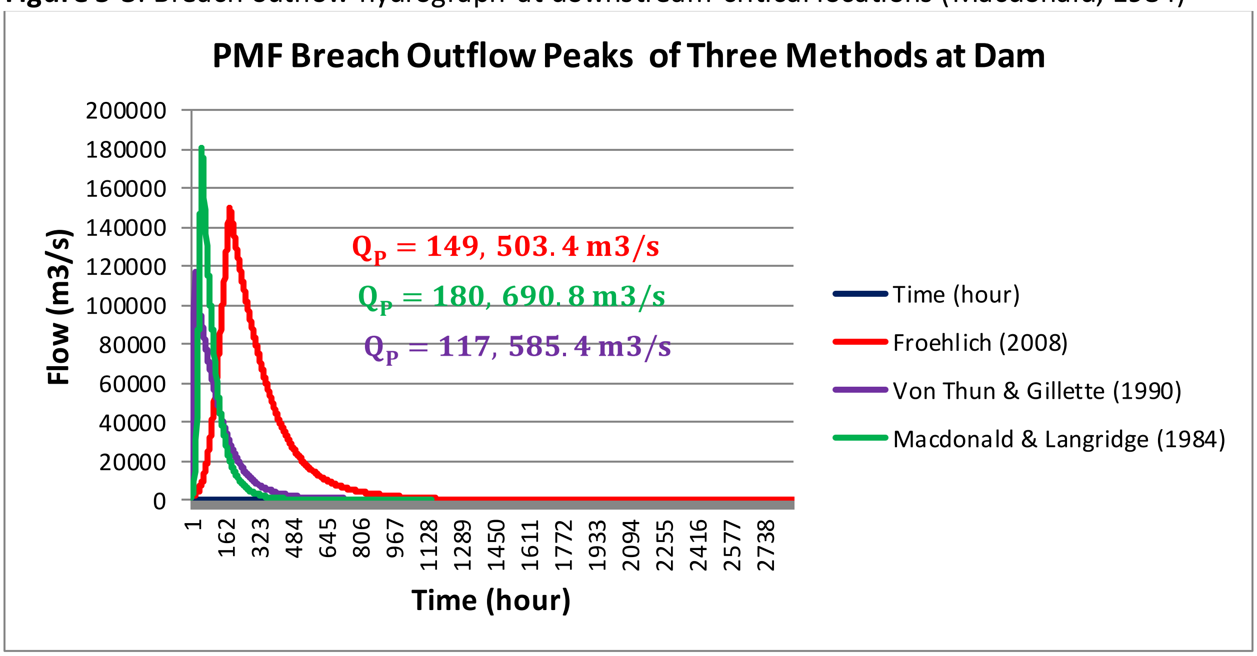


Figure 5-4: Breach outflow hydrograph at dam for Three Methods

5.2.2. Piping (Sunny Day) Mode of Failure Results

The maximum breach outflow and arrival time to peak breach outflow results obtained from HEC-RAS model simulation at dam for three methods in case the of non-hydrologic (piping) breach scenario are given in tables 5.5, 5.6, 5.7, 5.8 and 5.9 below.

Table 5-5: Maximum Pool Breach Peak Outflows for Piping Mode of Failure

Methods	Peak Flow (m3/s)	Time to Peak (hrs.)
Macdonald and Langridge Monopolis (1984)	163,026.8	4
Von Thun and Gillette (1990)	100,680	1.2
Froehlich (2008)	125,140.4	6

Table 5-6: Normal Pool Breach Peak Outflows for Piping Mode of Failure

Methods	Peak Flow (m3/s)	Time to Peak (hrs.)
Macdonald and Langridge Monopolis (1984)	142,195.3	3.45
Von Thun and Gillette (1990)	88,027.16	1.15
Froehlich (2008)	100,257.2	6

Table 5-7: Maximum flow, time to peak, rate of flow and flood height for some Dedessa River stations below Arjo-Dedessa Dam (Macdonald and Langridge-Monopolis, 1984)

Stations	Peak Flow (m^3/s)	Peak Time (Hr.)	Rate of Flow (m/s)	Flood Height (m)
At Dam	163,026.8	4	6.23	47
7 km	144,095.8	4.3	4.41	18.20
40 km	90,063.06	8.15	4.01	23.31
80 km	82,920.76	11.00	4.58	21.38
At 120 km d/s end of the river	76,178.26	14.45	8.45	8.02

Table 5-8: Maximum flow and time of peak flow for some Dedessa River stations below Arjo-Dedessa Dam. (Von Thun and Gillette, 1990)

Stations	Peak Flow (m^3/s)	Peak Time (Hr.)	Rate of Flow (m/s)	Flood Height (m)
At dam	100,680	1.2	5.23	47
7 km	84,958.02	3.3	4.01	13.99
40 km	59,445.27	9.10	2.92	20.20
80 km	56,963.57	12.20	4.05	18.19
At 120 km d/s end of the river	54,302.61	16.10	7.61	6.86

Table 5-9: Maximum flow, time to peak flow, and flood depth for some Dedessa River stations below Arjo-Dedessa Dam (Froehlich-2008)

Stations	Peak Flow (m^3/s)	Peak Time (Hr.)	Rate of Flow (m/s)	Flood Height (m)
At Dam	125,140.4	6	5.89	47
7 km	115,015.90	6.45	4.31	16.01
40 km	78,359.20	10.30	3.26	22.80
80 km	73,532.42	13.15	4.37	20.23
At 120 km d/s end of the river	68,667.45	17.00	8.14	7.59

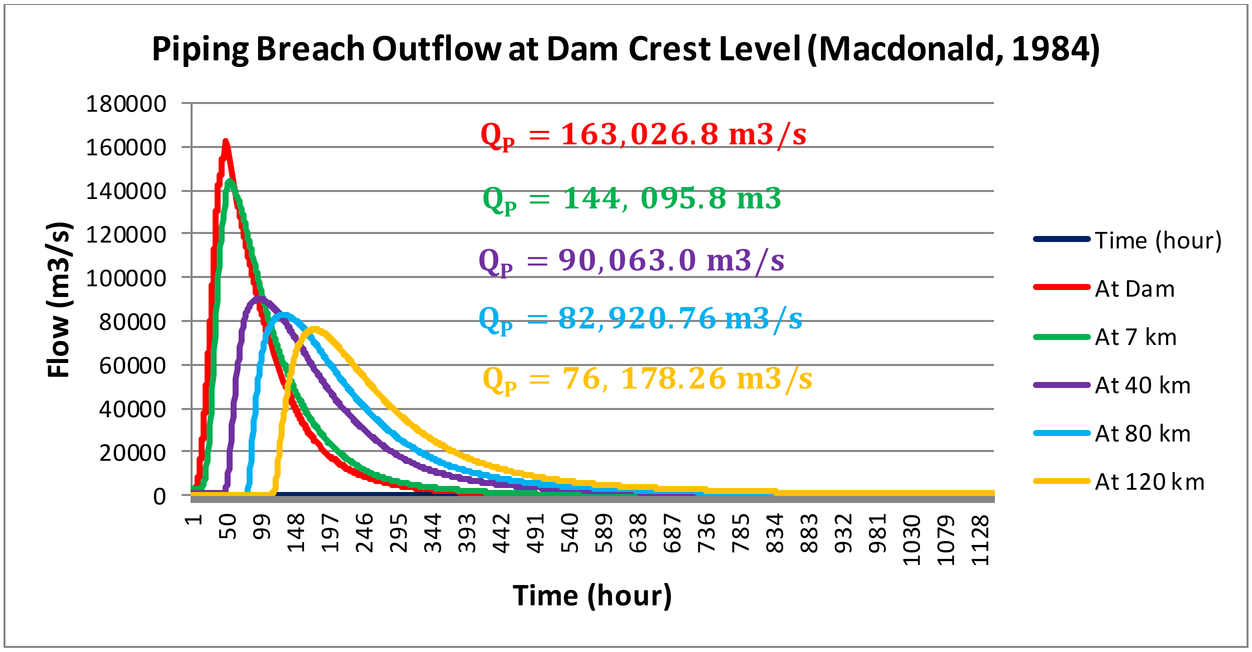


Figure 5-5: Maximum Pool Breach outflow hydrograph at downstream critical locations

(Macdonald and Langridge Monopolis- 1984)

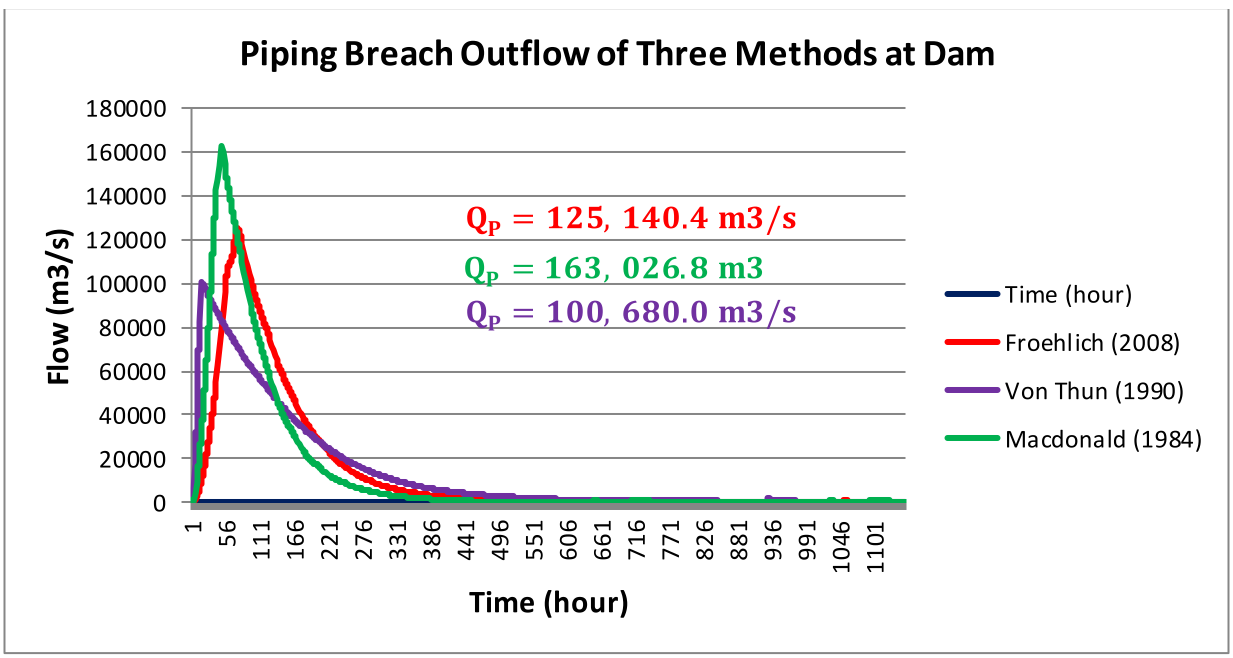


Figure 5-6: Maximum Pool Breach outflow hydrograph at dam for Three Methods (Piping)

5.3. Flood Inundation Mapping

Using the breach outflow hydrograph resulted from HEC-RAS model, downstream flood inundation extent and depth is delineated and mapped in order to differentiate the flooded areas in depth and area.

Water-surface profiles for the full PMF, 75-percent-PMF and sunny-day dam-breach scenarios using the result of three methods were developed. Maximum flood-inundation extent and depth for each critical locations for the full PMF, 75-percent-PMF and sunny-day dam-breach scenarios were also computed as shown in figure 5.7 -5.10 below.

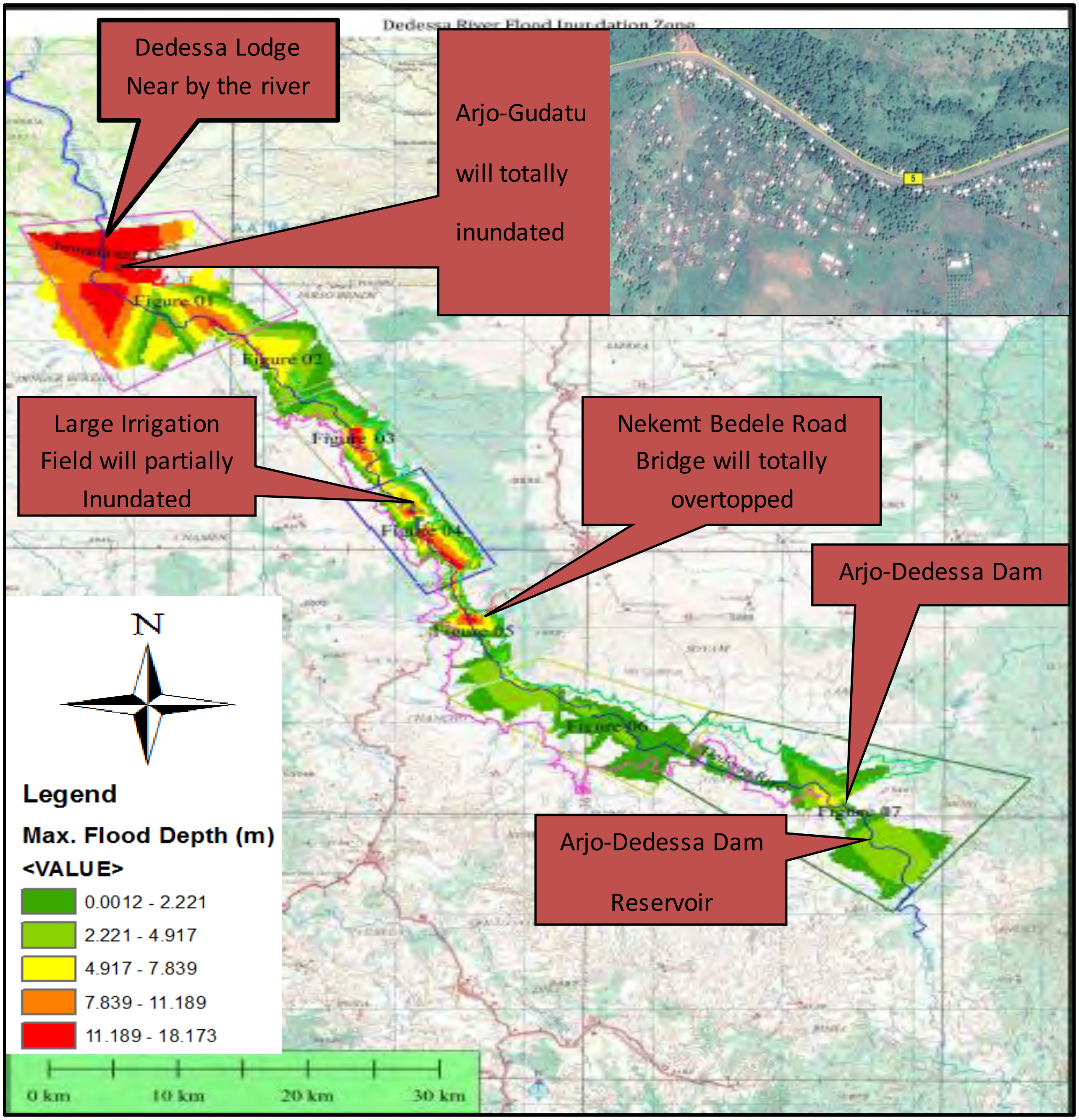


Figure 5-7: Arjo-Dedessa Dam Breach Inundated Area Map for Overtopping Mode of Failure

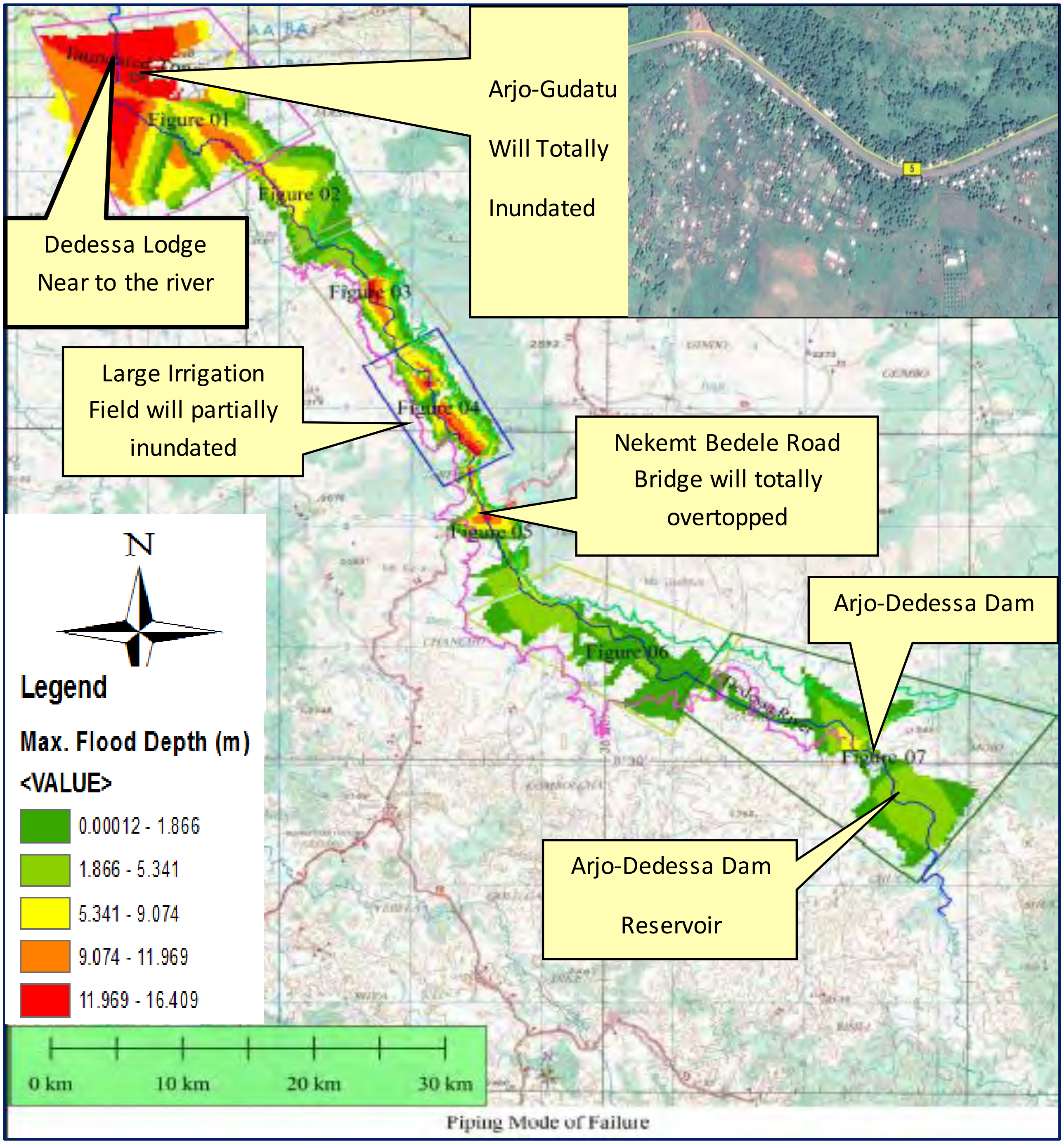


Figure 5-8: Arjo-Dedessa Dam Breach Inundated Area Map for Piping Mode of Failure

As shown in the map, from the downstream inundated critical locations which have a problem, one is found at 40 km downstream where Nekemt Bedele Bridge is found. The second area is a large irrigation field just downstream of this bridge up to 60km, and the third danger area is located at 90km downstream where Arjo Gudatu town, Dedessa Lodge and Nekemt Gimbi road bridge found respectively. Except at these three locations, the flood is confined within the main channel and there is no considerable affected area within the downstream flood path. Inundation area map at downstream critical locations are described briefly in appendix B.

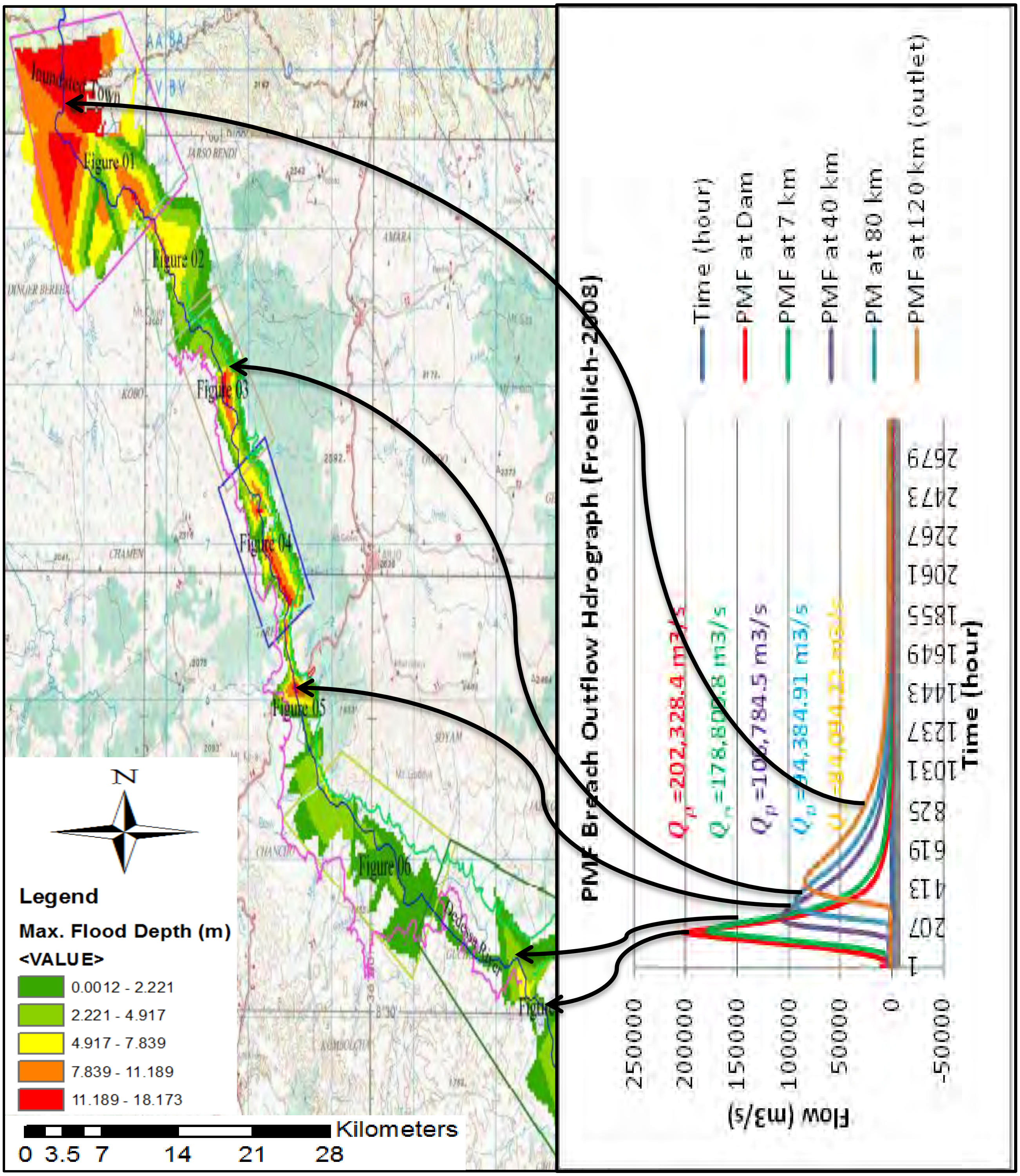


Figure 5-9: Peak Outflow Hydrographs at Downstream Critical Locations (Overtopping Case)

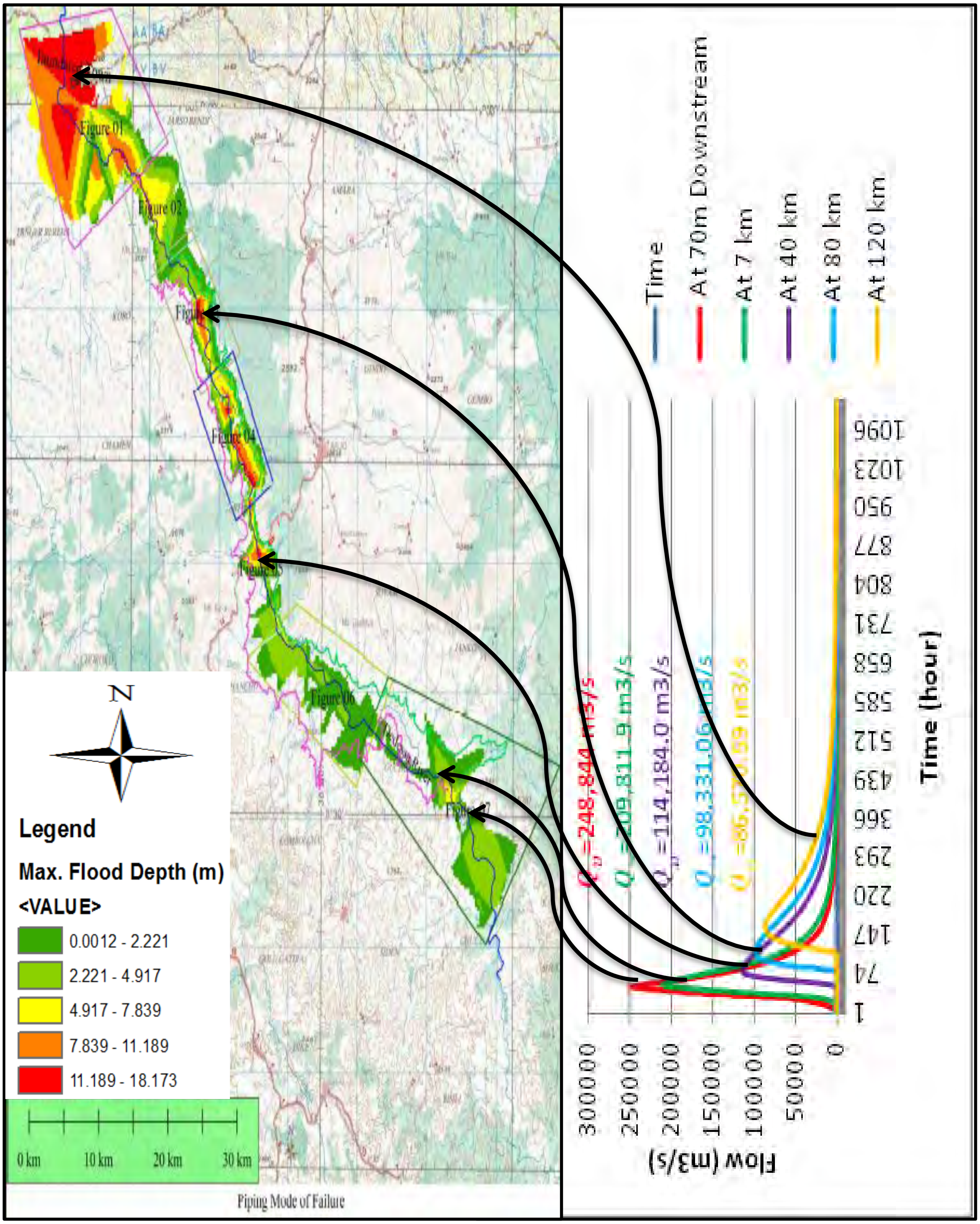


Figure 5-10: Peak Outflow Hydrographs at Downstream Critical Locations (Piping Case)

5.4. Discussion

The breach parameters estimated using three regression equations (Maconald and langridge Monopolis-1984, Thun and Gilette-1990 and Froehlich-2008) are different for the two mode of failure discussed in this paper.

For the hydrologic (overtopping) mode of failure, the first equation (Maconald and langridge Monopolis-1984) was over-predicting the breach bottom width and the breach bottom width for the second equation (Von Thun and Gilette-1990) was less than the two and is under estimate. The result from the third equation (Froehlich-2008) was fall between the two equations. The trend of these parametric equations to over-predict and under predict the breach size may be attributed to the fact that they are developed based on the assumption of breach forms. The breach development times for these methods were 3.9 hours, 1.19 hour, and 5.95 hours respectively. When these three results are compared, the result of Macdonald and Langridge-Monopolis breach width is larger and the breach development time is somewhat shorter. Both breach bottom width and breach development time for Von Thun and Gillette method is small. For the Froehlich method, even if the breach bottom width and breach development time were still large but, it doesn't mean that it over predict the results, because these parametric equations were derived from previously breached dam case studies that of mostly small dams in height and small water volumes behind the dam.

Arjo-Dedessa dam is high dam and the volume of the water in the reservoir is about 1.9 and 2.25 billion meter at FRL and MWL respectively. Therefore, in order to attain its final breach bottom width the breaching process will continue to erode for long time and may have larger breach width and longer breach development time. Again, in the case of Arjo-Dedessa Dam, since the dam is a rock-fill embankment dam consisting of impervious clay core and the amount of water behind the dam is extremely huge, and the erosion progress is not suddenly occurred, it can take more time to totally breach to its final width. In the case of piping mode of failure, except a small amount decreasing values in breach bottom width and breach time development, the results were the same with that of overtopping case.

As considered from the result of model simulation in the case of hydrologic or overtopping mode of failure (Probable Maximum Flood) event, the maximum breach outflows and time to peak were computed for three cases. Using Macdonald and Langridge-Monopolis (1984) breach parameter as model input in HEC-RAS , the maximum breach outflow and time to peak which occurred at the dam were 180,690.8 m3/s and 4.15 hours respectively. From Von Thun and Gillette (1990) also 117,585.4 m3/s. peak outflows and 1.4 hours' time to peak were calculated at the dam location. Again from Froehlich (2008), the maximum breach outflow which occurred at the dam and time to peak were 149,503.4 m3/s and 6.12 hours.

For the three methods, peak breach outflow results were different. Breach peak outflow for Von Thun and Gillette is smaller than the two. This implies that, for large breach bottom width and longer breach development time, the peak breach outflow is also large and vice versa. From the time to peak breach outflow of the three methods, one can consider the effect of time to peak in the process of warning and emergency action plan. The shorter time to peak is important for the necessary measures to be taken at downstream during failure case. Therefore, the time to peak result from Macdonald and Langridge-Monopolis (1984) was shorter than others and it is a critical one for this study.

For fair weather or sunny day (Piping) mode of failure at Maximum Pool Level, the maximum outflows and time to peak for the three methods was 163,026.8 m3/s and 4 hours at dam (Macdonald and Langridge Monopolis-1984), 100,680 m3/s and 1.2 hours (Von Thun and Gillette-1990), and 125,140.4 m3/s and 6 hours at dam (Froehlich-2008) respectively.

Whereas for normal pool level, the maximum outflows and time to peak for the three methods were 142,195.3 m3/s and 3.45 hours (Macdonald and Langridge Monopolis-1984), 88,027.16 m3/s and 1.15 hours (Von Thun and Gillette-1990), and 100,257.2 m3/s and 6 hours (Froehlich-2008) respectively.

The simulated results for overtopping case for Macdonald and Langridge-Monopolis (1984) showed a maximum flow velocity of 4.58 m/s with a discharge of 84,967.94 m^3/s occurred at 90 km downstream of the dam. For Von Thun and Gillette (1990), the maximum flow velocity of 4.05 m/s with a discharge of 61,975.45 m^3/s occurred at 90 km downstream of the dam.

Finally, for Froehlich (2008), the maximum flow velocity of 4.37m/s with a discharge of 79,327.19 m^3/s occurred at 90 km downstream of the dam.

From the peak discharge, flow velocity, time to peak, Macdonald and Langridge-Monopolis (1984) with maximum flow velocity of 4.01m/s and discharge of 92,949.5 m3/s occurred at 40 km and maximum flow velocity of 4.58 m/s and a discharge of 84,967.94 m3/s occurred at 90 km downstream of the dam is a critical method for the analysis of Arjo-Dedessa Dam Breach Modelling. The two remaining methods have smaller flow velocities and breach outflow discharges at a distance of 40 and 90 km where large irrigation field, Nekemt Bedele Road Bridge, Arjo-Gudatu town, Dedessa Lodge and Nekemt-Gimbi Road bridges are found. The arrival time to peak breach out flow result using Macdonald and Langridge-Monopolis (1984) method also shorter than the results from the two remaining methods.

It was determined that the MacDonald & Langridge- Monopolis equation produced peak flows significantly greater than that produced by the ultimate breach geometry. The time of breach development calculated by parametric regression equations for MacDonald & Langridge-Monopolis, Von Thun and Gillette and Froehlich were 3.9 hours, 1.19 hours and 5.95 hours respectively.

These parametric equations were derived using different number of data sets collected from different dam failure case studies happened previously. The ranges of the dam height and volume of water stored behind the dam from which theses data were collected are differ from that of the Arjo- Dedessa Dam height and volume of water behind the dam. The regression equations used for breach parameter estimation in this study, i.e. Macdonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990), and Froehlich (2008), were derived from case studies having dam height ranges of 4.27-92.96 m, 3.66-92.96m,3.05-92.966m and volume of water (0.0037-660)*10^6m3/s, (0.027-660)*10^6 m3/s and (0.0139-660)*10^6 m3/s respectively. From 173 data sets used, 90% of the dam heights were less than 30m and the volume of water in the reservoir were less than 660 million meter cube. In case of Arjo-Dedessa, the dam has a height of 47m and volume of water about 2.3 billion meter cube. This means, it leads to over predict or under predict breach width and breach development times.

The results of the flood inundation mapping from Arjo-Dedessa Dam breach analysis indicates that much of the severe flood inundation occurs after 40 km downstream of the dam near Arjo Sugar Factory where large Irrigation field is found in the left and right side of the river, and at 90 km downstream where a small town Arjo-Gudatu and Dedessa Lodge which are totally inundated are located as shown in figure 5.8 and 5.8 above. The PMF event, mapped in figure 5.9, indicates that there will be coverage of large irrigation field at about 40 km downstream of the dam where large left and right irrigation field are found with the ranges of flood depth from 0-18.17m. In addition to this, the two bridges found on Nekemt Bedele and Nekemt Gimbi asphalt roads are also totally submerged by breach outflow flood.

6. EMERGENCY ACTION PLAN

6.1. Purpose

The purpose of the Emergency Action Plan (EAP) for this study is to safeguard lives and secondarily to reduce property damage in the event that Arjo-Dedessa Dam would fail. To carry out this work, the EAP contains :1) procedures to monitor Arjo-Dedssa Dam periodically and during flood warnings issued by the National Meteorological Agency; 2) notify appropriate body of Oromia Regional State Government a potential dam failure; and 3) warn and evacuate the isolated residences at risk. These procedures are to supplement and be used in conjunction with Ethiopian Federal Government Concerned Body (Ministry of Water, Irrigation and Energy), the owner of the Arjo-Dedessa dam.

6.2. Flood Description

Failure of the dam could cause significant damage to large irrigation field which is found around 35 km downstream of Arjo-Dedessa Dam within danger flood reach, Nekemt Bedele and Nekemt Gimbi roads also affected by this dam failure. Again, a small town (Arjo Gudatu) and Dedessa Lodge located at a distance of 90 km downstream of the dam within the danger flood reach are affected.

6.3. Operating Procedure

1. The dam will be inspected periodically each year during the construction time and after construction for maintenance and suffering signals.
2. The dam observer will inspect the dam when the National Meteorological Agency (NMA) issues a Flood Warning for the area and also will note & record water levels in reservoir and the rate at which the pool is rising. If the dam shows signs of internal piping (muddy seepage exiting the downstream embankment), erosion, slope failures, blocked spillways, or other, the dam observer will call the local or regional concerned body to block downstream roads and warn downstream residences including the people within the Dedessa lodge in the danger reach. The dam observer may contact the Ministry of Water, Irrigation and Energy (owner of the dam) or his designated engineer to provide assistance.

If the pool level rises too within small range levels of the dam crest, the dam observer will contact the local concerned body or Ministry of Water, Irrigation and Energy (owner of the dam) or his designated engineer to provide assistance to dispatch police to block downstream roads and warn downstream residences including the people within the Dedessa lodge in the danger reach.

DAM NAME: - Arjo-Dedessa Irrigation Project Dam

Signatures of Persons Involved in Emergency Action Plan

Dam Owner By _______________Date

Ministry of Water, Irrigation and Energy

Typed Name: _____________

Title: ______________________

Phone: ____________________

(Day)________________

(Night)_______________

Local Department of Emergency Operations By: _______________ Date

____

Typed Name:________________

Title: _______________________

Phone: _____________________

(Day)_________________

(Night)________________

Local or Regional Police By: _______________Date

____

Typed Name: _________________

Title: ________________________

Phone-(Day):_________________

(Night):________________

Department of the Irrigation and Drainage Projects (MWIE) By: _____________ Date

Typed Name: _________________

Title: ________________________

Phone-Day):__________________

(Night):________________

Owner's Engineer By: ______________________ Date

____

6.4. Preventative Actions

If time allows, contact the design and consultant party of the project Oromia Water Works aand Supervision Enterprise (OWWDSE) or the representative Engineer and the Federal Ministry of Water, Irrigation and Energy, Department of Irrigation and Drainage Projects for advice on preventative actions. Listed below are potential emergency actions which may prevent or delay the failure of the dam. They should be considered based on site-specific conditions, as well as the risk of failure and risk to employees.

Possible actions to be taken in the event of: Imminent Overtopping by Flood Waters:

- Open drain or flood gates to maximum capacity.
- Place sand bags along the dam crest to increase freeboard.
- Place riprap or sandbags in damaged areas of dam.
- Provide erosion protection on downstream slope by placing riprap or other appropriate materials.
- Divert flood waters around dam if possible

Erosion of dam by seepage or piping through the embankment:

- ✓ Plug the seepage with appropriate material such as (riprap, grass bundles, sandbags, soil, or plastic sheeting if the leak is on upstream face of dam).
- ✓ Lower the reservoir level until the flow decreases to a non-erosive velocity or stops leaking.
- ✓ Place sand and gravel filter over the seepage exit area to minimize loss of embankment soils.
- ✓ Continue lowering the reservoir level until the seepage stops or is controlled. Refill reservoir to normal levels only after seepage is repaired.

6.5. Supplies and Resources

In an emergency situation, equipment and supplies may be needed on short notice. The following supplies and resources may be needed during an emergency: earthmoving equipment, sand and gravel, sandbags, riprap, pumps, pipe, labourers.

List of Contractors

It will be the responsibility of the owner (Ministry of Water, Irrigation and Energy) to maintain the list of contractors and consultants that may be contacted during an emergency condition for equipment materials, and repairs, engineers. For each contractor on the list, the following information is needed:

- Contractor name
- Contact person.

  -Address.

  -Phone number.
- Equipment & repair supplies available.-
- Arrival time to dam

Contractor: Oromia Water Works Construction Enterprise

Contact person: ________________________________ Phone No: __________________

Address: _______________________

Services contracted for: ____________________________________

Consultant: Oromia Water Works Design and Supervision Enterprise

Contact person: ________________________________Phone No: __________________

Address: _______________________

Services contracted for: ____________________________________

Recommended Steps for Emergency Action Plan Preparation

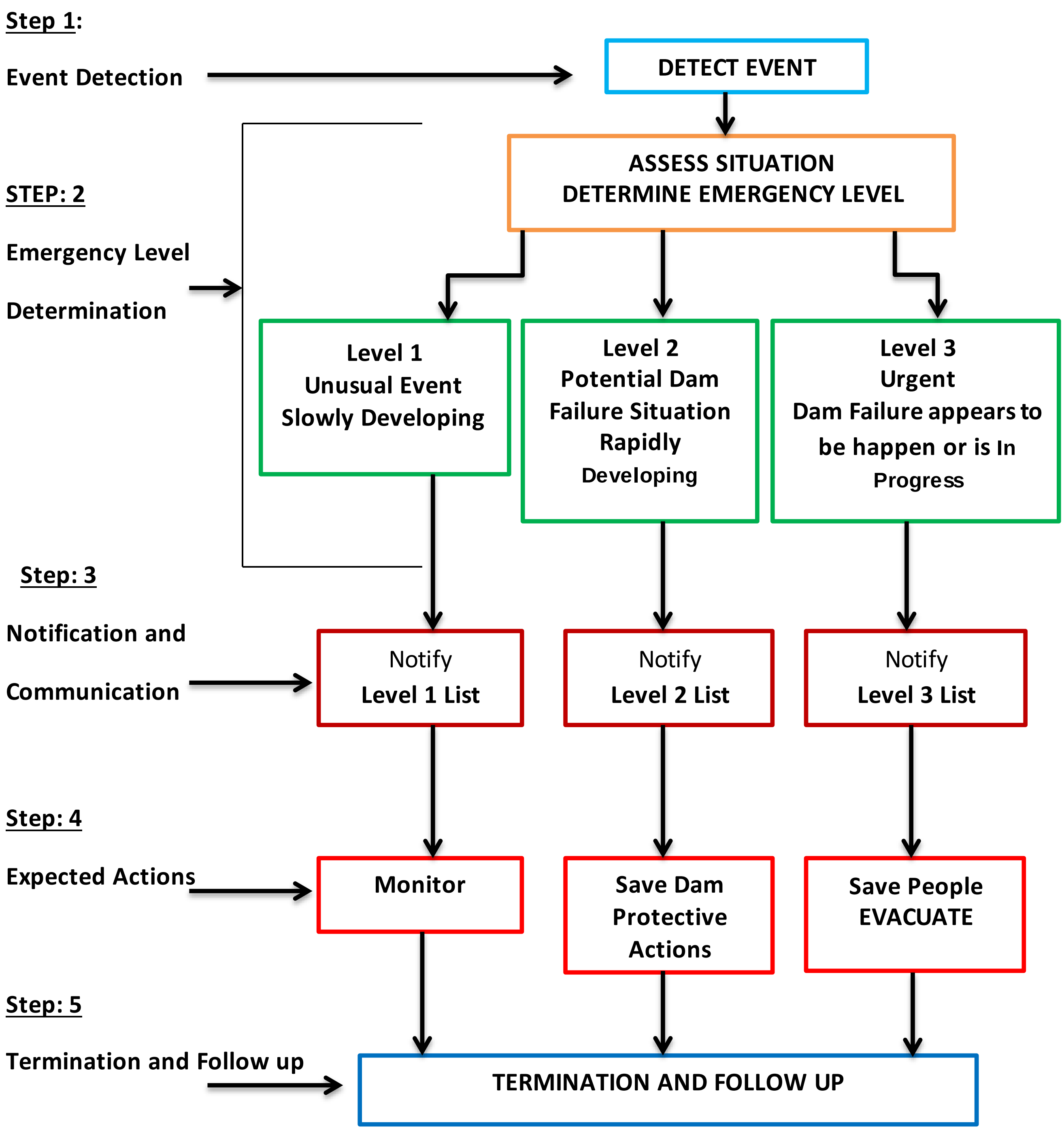


Figure 6-1: Emergency Action Plan Steps

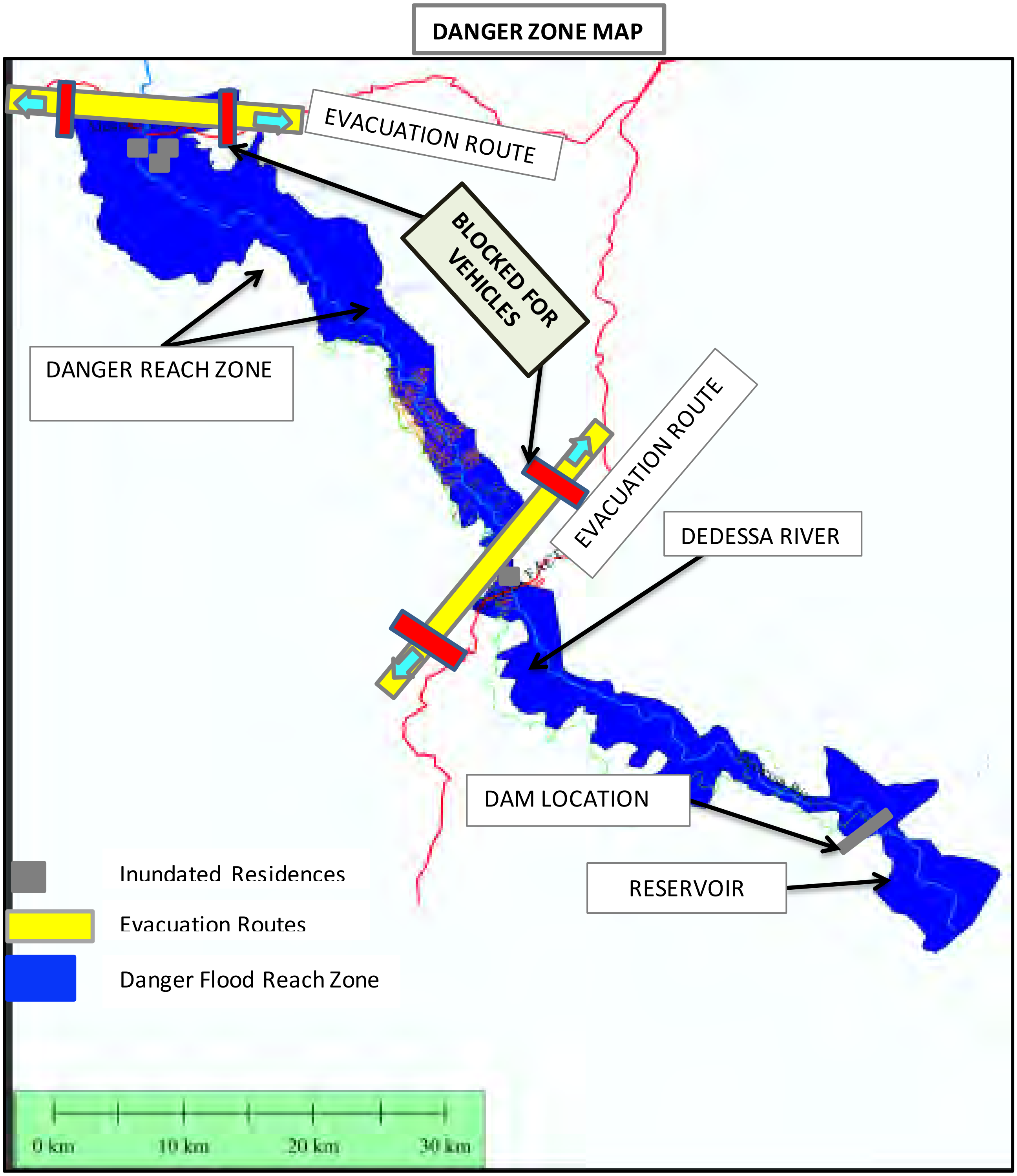


Figure 6-2: Danger Zone Map of Arjo-Dedessa Dam Breach Flood Inundation Area

7. CONCLUSION AND RECOMMENDATION

7.1. Conclusion

From breach parameters results calculated by using regression methods (Macdonald and Langridge-1984, Von Thun and Gillette-1990 and Froehlich-2008), (Macdonald and Langridge-1984 is the critical and more preferred method for Arjo-Dedessa dam breach parameter estimation. Breach bottom width estimated using this method is greater than that of Von Thun and Froehlich methods. This is due to the height of the dam and large volume of water behind the dam in case of Arjo-Dedessa dam and since it is a high rock-fill embankment dam consisting of impervious clay core, the erosion progress is not suddenly occurred. In order to attain its final breach bottom width, the breaching process will continue to erode for long time and may have larger breach width. The simulated result also showed that the arrival time of the maximum flood from the dam breach in the case of (Macdonald and Langridge-1984 is shorter than the two methods. This in other way simplifies the emergency works in case the dam failure, to make warning and evacuate the downstream people. Therefore, for Arjo- Dedessa rock-fill embankment dam (Macdonald and Langridge-1984 method is more critical and reasonable to estimate the breach parameters and to use as an input for hydraulic modeling purpose.

According to the HEC-RAS Model simulation results, Arjo-Dedessa Dam overtops in the case of the worst flood of maximum probable flood (PMF) by 0.39m, 0.78m, and 0.87m for Macdonald and Langridge-1984, Von Thun, and Froehlich respectively. In order to detect the maximum and worst incoming flood which is overtopped by 0.87m and to make safe the emergency action plan for downstream, Macdonald and Langridge-1984 is the critical one.

The Breach outflow flood from the model showed that the inundated areas ranges from a depth of 0.0 to18.17 at the river channel to flood plain areas out of the left and right banks. At 40km downstream where huge irrigation fields and bridge are found also the flood depth is different. The location where the Nekemt Bedele Bridge is found is totally inundated by a maximum depth of 18.17m. Downstream of this bridge, large part of irrigation field is also covered by a flood depth ranges of 0.0 to 18.17m. In this case from the total coverage some part found in the ranges of 0.0 to 2.209m depth. The two similar areas are covered by a flood depth of ranges 2.209m to 4.917m and 4.917m to 7.84m. The rest ranges 7.84m to 11.19m and 11.19m to 18.17m are found at around the banks and within the river channel. From this, large part of the area will be covered by flood depth of greater than 2.209 which may damage fields. At a distance of 90 km downstream of the dam, there are a small town, Arjo Gudatu and Dedessa lodge which are totally inundated in the ranges of 7.84m to 18.17m. These locations are found at an area where the river banks are becoming narrow and the incoming flood depth increases. This is why these locations will be inundated by large depth of flood. Therefore, the impact of this flood on the downstream economic loss is very large. Unless otherwise the people in the town and around the lodge get early warning and emergency action rapidly, the life loss may occur is large. The two bridges found at 40 km and 90 kms will also totally submerged by the flood from the dam breach. Due to these consequences, Arjo-Dedessa Dam hazard classification can be said to be high. inundation flood depth of the case of sunny day (piping) mode of failure is almost similar except the decrease of the maximum flood depth ranges by two meter which has no significant change effect on this depth ranges.

Finally, in order to safeguard lives and to reduce property damage in the event that Arjo-Dedessa Dam would fail, an emergency action plan which contains different procedures is prepared. In this EAP, different concerned bodies (owner, contractor, regional or local polices, etc.) will participate.

7.2. Recommendation

Arjo-Dedessa dam is currently under construction. It was started in 2011 and the completion time is on June 2016. But, still it is found on the construction of coffer dam which is part of the main dam to divert the main river. Even if the river is diverted currently, the upstream reservoir is starting to fill before the coffer dam is completed. The seepage problem may arise due to the over saturation of compaction process from the 2015 rainy season which leads to erosion and finally piping problem.

Therefore, in order to prevent this problem the construction process should be progressed according to the schedule to prevent the probability of dam failure during construction period. The dam owner (Ministry of Water, Irrigation and Energy) should properly control and evaluate the progress of the construction.

As a simulation results shown that the Arjo-Dedessa Dam will overtop during the hydrologic event of full PMF by a range of to 0.87 m and 0.39m above the crest dam. Therefore, the appropriate solution should be given for this problem from now before the construction is completed. The solution may be adding a free board of about 1 meter or it may be increasing the capacity of the spillway. Therefore, the owner of the dam (Ministry of Water, Irrigation and Energy) with the engineering party or consultant (Oromia Water Works Design and Supervision Enterprise), should properly check and take the possible action for the solution.

The breach outflow flood inundates the downstream large irrigation fields, small town (Arjo-Gudatu) and Dedessa lodge near Dedessa River. Therefore, according to the emergency action plan prepared, every stake holders should participate in emergency works in case the dam will fail. These stakeholders are the owner (MWIE), the contractor, the consultant, National Meteorological Agency, regional or local police, and others.

Recently, due to the high amount of rain upstream of the coffer dam which is under construction and the small size of the diversion pipes, the incoming flood was to pass over it. But, to prevent this failure, they create additional temporarily outlet in the direction of saddle dam II until this rainy season will pass. Therefore, the workman ship and some design problems may increase the failure potential of this dam in the duration of construction time. Therefore, the owner of the dam (MWIE), the consultant and the contractor must focus on this project until its completion time.

Finally, even if this dam was designed to pass the inflow design flood safely, the dam will overtop in the case of PMF and 0.75% PMF flood. Therefore, its design could be rechecked and the breach model also can be analyzed by other physically based methods to ensure the safety of the dam for the future.

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APPENDICES

Appendix A: Hydrologic and Hydraulic Input Data for HEC-RAS Model

Table 1: Probable Maximum Flood Inflow Hydrograph of Dedesa River (OWWDSE, Dec 2013)

Time (Hrs.)	Direct Runoff ($\frac{m^3}{s}$)	Base flow ($\frac{m^3}{s}$)	Total Flood flow ($\frac{m^3}{s}$)
0.00	0.00	152.88	153
2.25	22.78	152.88	176
4.50	42.71	152.88	196
6.75	122.43	152.88	275
9.00	213.53	152.88	366
11.25	313.18	152.88	466
13.50	455.54	152.88	608
15.75	626.37	152.88	779
18.00	797.19	152.88	950
20.25	1024.96	152.88	1178
22.50	1224.26	152.88	1377
24.75	1480.50	152.88	1633
27.00	1708.27	152.88	1861
29.25	1964.51	152.88	2117
31.50	2192.28	152.88	2345
33.75	2363.11	152.88	2516
36.00	2533.94	152.88	2687
38.25	2647.82	152.88	2801
40.50	2761.71	152.88	2915

42.75	2818.65	152.88	2972
45.00	2847.1	152.88	3000
47.25	2818.65	152.88	2972
49.50	2790.18	152.88	2943
51.75	2704.76	152.88	2858
54.00	2619.35	152.88	2772
56.25	2505.47	152.88	2658
58.50	2391.58	152.88	2544
60.75	2277.70	152.88	2431
63.00	2135.34	152.88	2288
65.25	2021.46	152.88	2174
67.50	1879.10	152.88	2032
69.75	1736.74	152.88	1890
72.00	1594.39	152.88	1747
74.25	1494.74	152.88	1648
76.50	1395.09	152.88	1548
78.75	1295.44	152.88	1448
81.00	1195.79	152.88	1349
83.25	1124.61	152.88	1277
85.50	1053.43	152.88	1206
87.75	982.26	152.88	1135
90.00	911.08	152.88	1064
92.25	854.14	152.88	1007
94.50	797.19	152.88	950
96.75	740.25	152.88	893
99.00	683.31	152.88	836
101.25	640.60	152.88	793
103.50	597.90	152.88	751

105.75	555.19	152.88	708
108.00	512.48	152.88	665
110.25	484.01	152.88	637
112.50	455.54	152.88	608
114.75	412.83	152.88	566
117.00	370.13	152.88	523
119.25	341.65	152.88	495
121.50	313.18	152.88	466
123.75	296.10	152.88	449
126.00	279.02	152.88	432
128.25	264.78	152.88	418
130.50	250.55	152.88	403
132.75	232.04	152.88	385
135.00	213.53	152.88	366
137.25	200.01	152.88	353
139.50	186.49	152.88	339
141.75	172.96	152.88	326
144.00	159.44	152.88	312
146.25	149.95	152.88	303
148.50	140.46	152.88	293
150.75	130.97	152.88	284
153.00	121.48	152.88	274
155.25	111.99	152.88	265
157.50	102.50	152.88	255
159.75	96.09	152.88	249
162.00	89.68	152.88	243
164.25	83.28	152.88	236
166.50	76.87	152.88	230

168.75	72.60	152.88	225
171.00	68.33	152.88	221
173.25	64.06	152.88	217
175.50	59.79	152.88	213
177.75	55.52	152.88	208
180.00	51.25	152.88	204
182.25	48.40	152.88	201
184.50	45.55	152.88	198
186.75	42.71	152.88	196
189.00	39.86	152.88	193
191.25	37.49	152.88	190
193.50	35.11	152.88	188
195.75	32.74	152.88	186
198.00	30.37	152.88	183
200.25	28.00	152.88	181
202.50	25.62	152.88	179
204.75	24.20	152.88	177
207.00	22.78	152.88	176
209.25	21.35	152.88	174
211.50	19.93	152.88	173
213.75	18.51	152.88	171
216.00	17.08	152.88	170
218.25	15.66	152.88	169
220.50	14.24	152.88	167
222.75	12.81	152.88	166
225.00	11.39	152.88	164

Table 2: Probable Maximum Flood Inflow Hydrograph Wama River (OWWDSE, Dec 2013)

Time (hr)	Direct Runoff (cumec)	Base flow (cumec)	Total Flood flow (cumec)
0.00	0.00	98.17	98
2.25	14.61	98.17	113
4.50	27.40	98.17	126
6.75	78.55	98.17	177
9.00	137.01	98.17	235
11.25	200.95	98.17	299
13.50	292.29	98.17	390
15.75	401.90	98.17	500
18.00	511.51	98.17	610
20.25	657.66	98.17	756
22.50	785.54	98.17	884
24.75	949.95	98.17	1048
27.00	1096.10	98.17	1194
29.25	1260.51	98.17	1359
31.50	1406.66	98.17	1505
33.75	1516.27	98.17	1614
36.00	1625.88	98.17	1724
38.25	1698.95	98.17	1797
40.50	1772.02	98.17	1870
42.75	1808.56	98.17	1907
45.00	1826.8	98.17	1925
47.25	1808.56	98.17	1907
49.50	1790.29	98.17	1888
51.75	1735.48	98.17	1834
54.00	1680.68	98.17	1779

Time (hr)	Direct Runoff (cumec)	Base flow (cumec)	Total Flood flow (cumec)
56.25	1607.61	98.17	1706
58.50	1534.53	98.17	1633
60.75	1461.46	98.17	1560
63.00	1370.12	98.17	1468
65.25	1297.05	98.17	1395
67.50	1205.71	98.17	1304
69.75	1114.36	98.17	1213
72.00	1023.02	98.17	1121
74.25	959.08	98.17	1057
76.50	895.14	98.17	993
78.75	831.21	98.17	929
81.00	767.27	98.17	865
83.25	721.60	98.17	820
85.50	675.93	98.17	774
87.75	630.25	98.17	728
90.00	584.58	98.17	683
92.25	548.05	98.17	646
94.50	511.51	98.17	610
96.75	474.97	98.17	573
99.00	438.44	98.17	537
101.25	411.04	98.17	509
103.50	383.63	98.17	482
105.75	356.23	98.17	454
108.00	328.83	98.17	427
110.25	310.56	98.17	409
112.50	292.29	98.17	390

Time (hr)	Direct Runoff (cumec)	Base flow (cumec)	Total Flood flow (cumec)
114.75	264.89	98.17	363
117.00	237.49	98.17	336
119.25	219.22	98.17	317
121.50	200.95	98.17	299
123.75	189.99	98.17	288
126.00	179.03	98.17	277
128.25	169.89	98.17	268
130.50	160.76	98.17	259
132.75	148.89	98.17	247
135.00	137.01	98.17	235
137.25	128.33	98.17	227
139.50	119.66	98.17	218
141.75	110.98	98.17	209
144.00	102.30	98.17	200
146.25	96.21	98.17	194
148.50	90.12	98.17	188
150.75	84.03	98.17	182
153.00	77.94	98.17	176
155.25	71.86	98.17	170
157.50	65.77	98.17	164
159.75	61.66	98.17	160
162.00	57.55	98.17	156
164.25	53.43	98.17	152
166.50	49.32	98.17	147
168.75	46.58	98.17	145
171.00	43.84	98.17	142

Time (hr)	Direct Runoff (cumec)	Base flow (cumec)	Total Flood flow (cumec)
173.25	41.10	98.17	139
175.50	38.36	98.17	137
177.75	35.62	98.17	134
180.00	32.88	98.17	131
182.25	31.06	98.17	129
184.50	29.23	98.17	127
186.75	27.40	98.17	126
189.00	25.58	98.17	124
191.25	24.05	98.17	122
193.50	22.53	98.17	121
195.75	21.01	98.17	119
198.00	19.49	98.17	118
200.25	17.96	98.17	116
202.50	16.44	98.17	115
204.75	15.53	98.17	114
207.00	14.61	98.17	113
209.25	13.70	98.17	112
211.50	12.79	98.17	111
213.75	11.87	98.17	110
216.00	10.96	98.17	109
218.25	10.05	98.17	108
220.50	9.13	98.17	107
222.75	8.22	98.17	106
225.00	7.31	98.17	188

Table 3: Maximum Annual 24-hr Rainfall

Year	(mm/d)	Year	(mm/d)	Year	(mm/d)
1967	47.1	1980	45.6	1993	50.1
1968	47.0	1981		1994	57.8
1969	64.0	1983	50.0	1995	61.0
1970	57.0	1984	51.7	1996	60.1
1971	70.0	1985	62.2	1997	104.5
1972	45.2	1986	64.6	1998	63.5
1973	35.9	1987	58.4	1999	63.0
1974	51.8	1988	53.2	2000	47.9
1975	36.4	1989	46.8	2001	69.0
1976	47.1	1990	49.5	2002	57.0
1978		1991	87.8	2003	
1979	45.0	1992	51.6	2004	39.1
				Average	54.14
				CV	0.306
				Skew	2E-04

Table 4: Elevation-Area-Capacity Relationships of Arjo Dedessa Reservoir (OWWDSE, 2013**)**

Elevation (m a.s.l)	Area (km^2)	Volume (Mm^3)	Elevation (m a.s.l)	Area (km^2)	Volume (Mm^3)
1312	0.54	0.0	1348	87.85	1341.2
1313	0.99	0.8	1349	90.75	1430.5
1314	2.39	2.5	1350	94.18	1522.9
1315	3.78	5.5	1351	97.23	1618.6
1316	5.11	10.0	1352	100.54	1717.5
1317	6.64	15.9	1353	103.64	1819.6
1318	8.03	23.2	1354	106.39	1924.6
1319	9.46	31.9	1355	109.00	2032.3
1320	10.87	42.1	1356	111.91	2142.8
1321	12.67	53.9	1357	115.05	2256.3
1322	14.40	67.4	1358	117.68	2372.6
1223	15.89	82.5	1359	120.20	2491.6
1324	17.74	99.4	1360	122.76	2613.0
1325	19.46	118.0	1361	125.28	2737.1
1326	23.59	139.5	1362	127.69	2863.5
1327	26.66	164.6	1363	130.25	2992.5
1328	29.05	192.5	1364	133.03	3124.2
1329	31.73	222.9	1365	135.45	3258.4
1330	34.24	255.8	1366	138.08	3395.2
1331	36.62	291.3	1367	142.38	3535.4
1332	39.45	329.3	1368	145.27	3679.2
1333	41.97	370.0	1369	148.00	3825.8
1334	44.25	413.1	1370	150.75	3975.2
1335	47.05	458.8	1371	153.73	4127.5
1336	49.95	507.3	1372	156.02	4282.3

1337	53.11	558.8	1373	159.09	4439.9
1338	56.31	613.5	1374	162.13	4600.5
1339	59.15	671.2	1375	166.96	4765.0
1340	61.56	738.8	1376	173.01	4935.0
1341	63.97	808.7	1377	178.58	5110.8
1342	67.88	874.7	1378	183.38	5291.8
1343	70.62	943.9	1379	188.10	5477.5
1344	74.35	1016.4	1380	193.28	5668.2
1345	77.88	1092.5	1381	197.36	5863.6
1356	81.28	1172.1	1382	201.96	6063.2
1347	84.51	1255.0			

Table 5: Extracted Manning's Roughness Coefficients for Cross-sections

Item No.	n-Value	Item No.	n-Value
1	0.04	21	0.03
2	0.03	22	0.035
3	0.035	23	0.035
4	0.03	24	0.04
5	0.04	25	0.03
6	0.04	26	0.04
7	0.04	27	0.035
8	0.04	28	0.03
9	0.04	29	0.04
10	0.035	30	0.15
11	0.04	31	0.035
12	0.035	32	0.04
13	0.03	33	0.03

14	0.035	34	0.04
15	0.04	35	0.035
16	0.035	36	0.035
17	0.04	37	0.04
18	0.03		
19	0.04		
20	0.035		

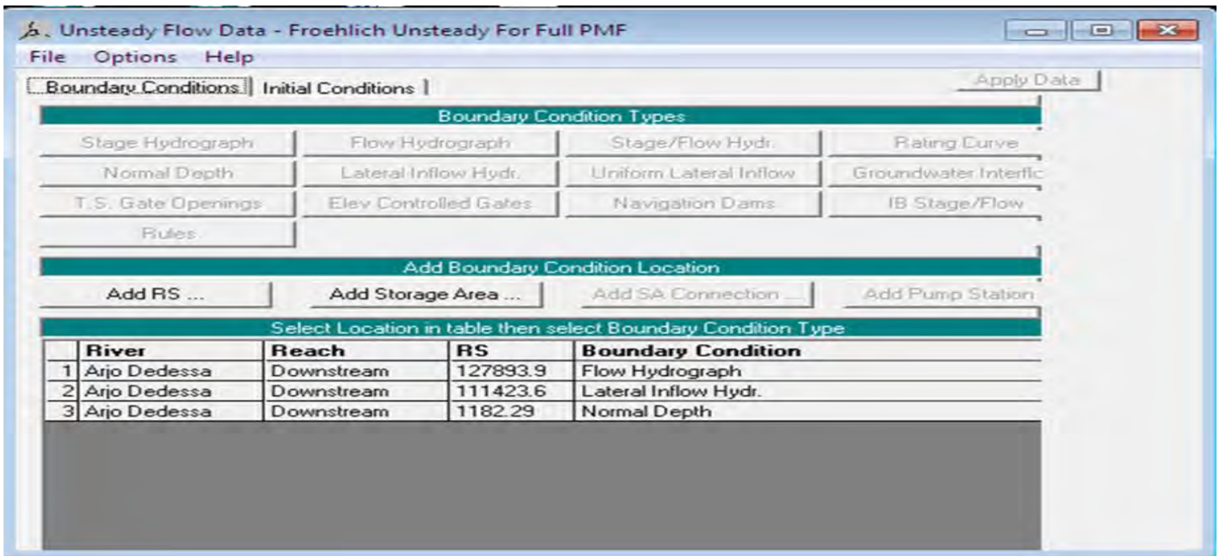


Figure 1: Unsteady Flow Boundary Condition

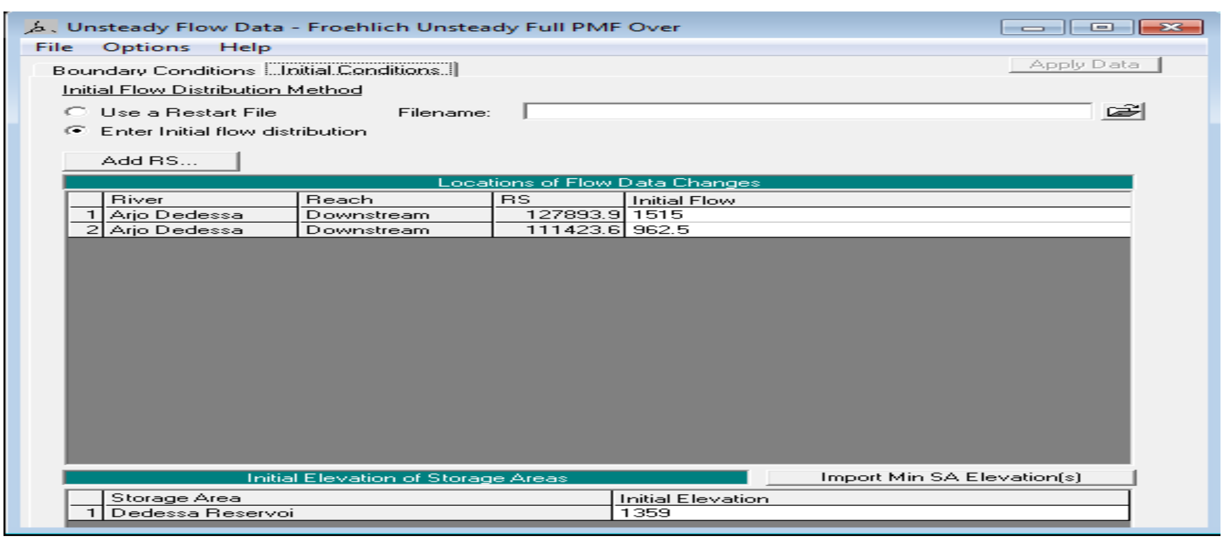


Figure 2: Unsteady Flow Initial Condition

Appendix B: Output Results of HEC-RAS Model

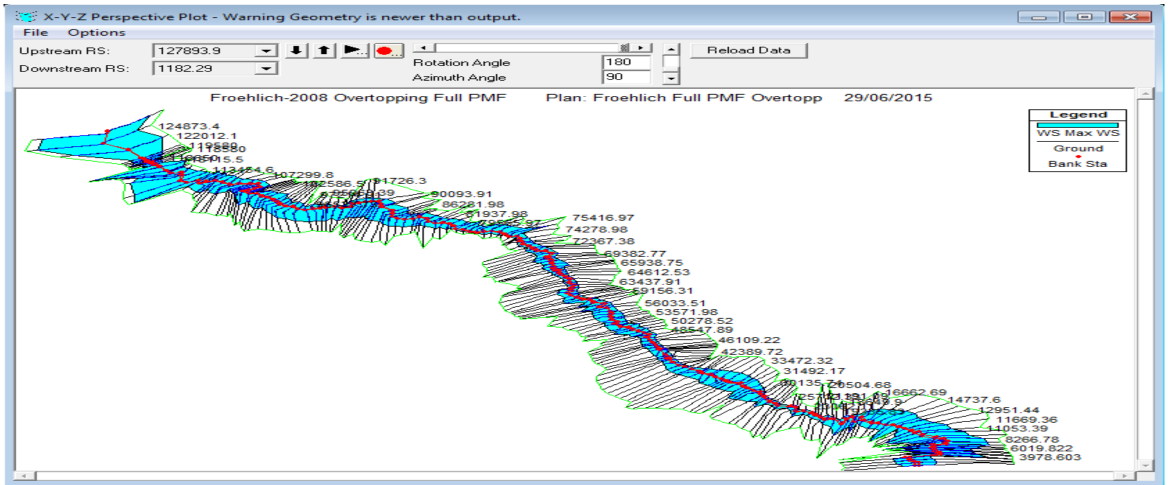


Figure 3: 3D Multiple Cross-section Plot for Dedessa River

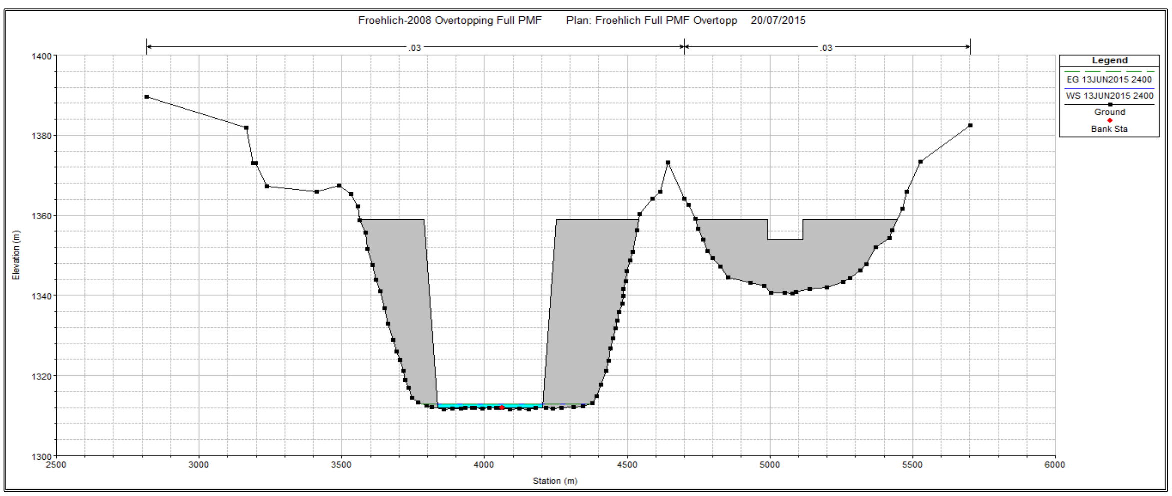
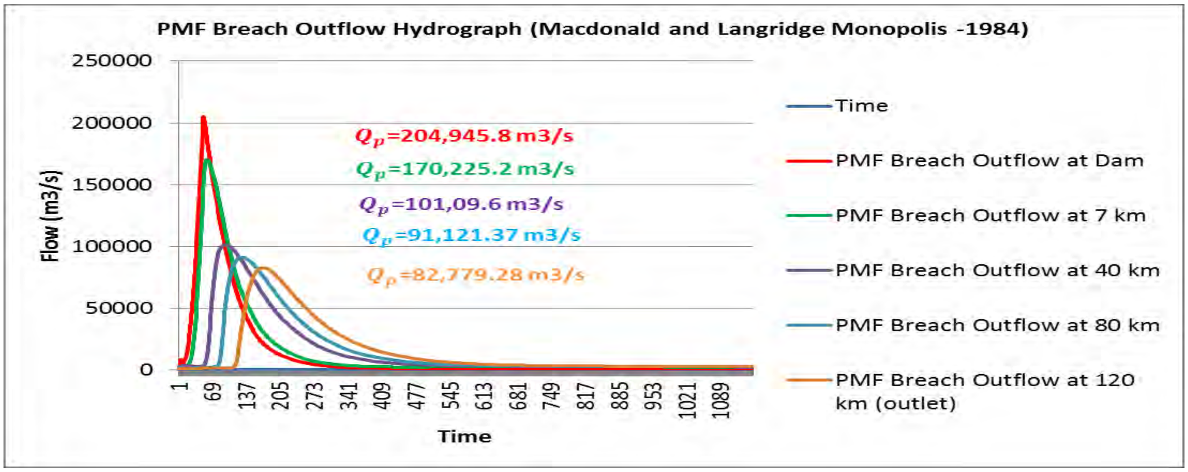
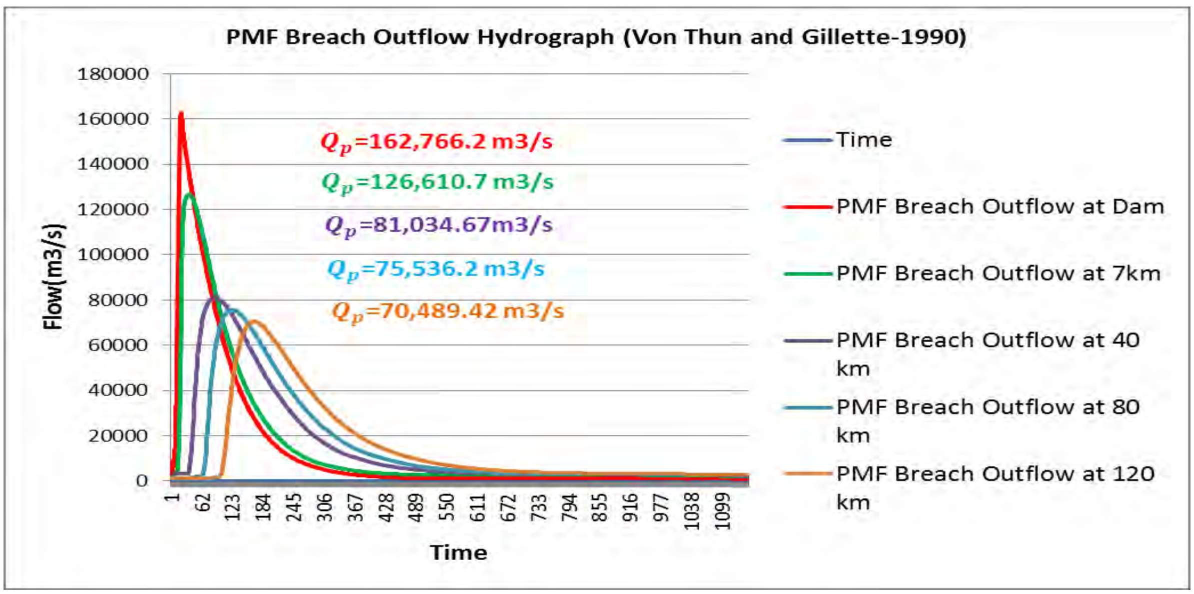


Figure 4: Breached Cross-Section of Arjo-Dedessa Dam

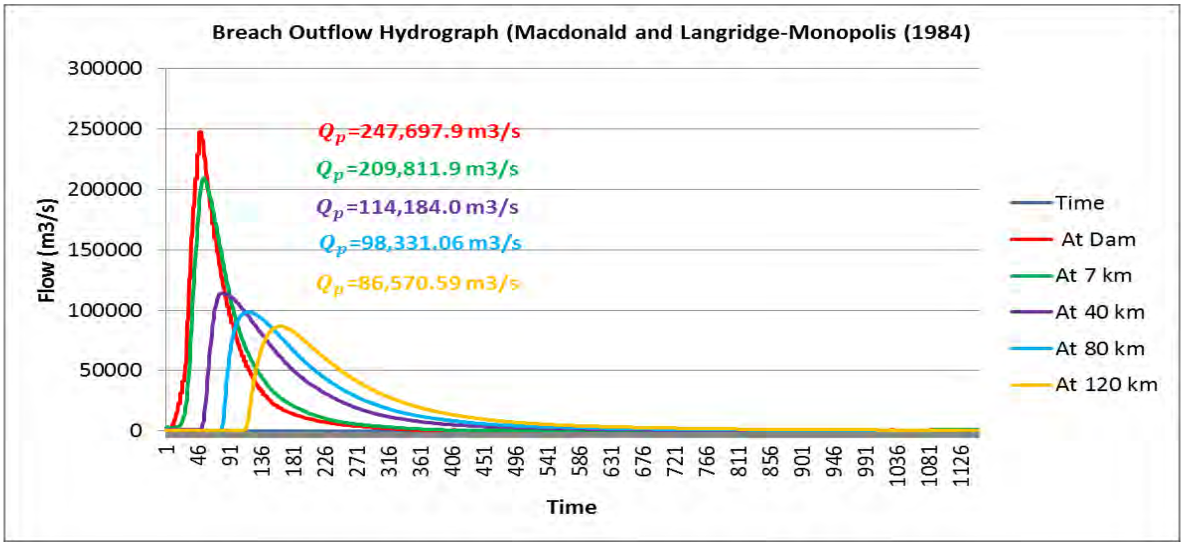


(a) At downstream critical locations (Macdonald and Langridge Monopolis 1984)

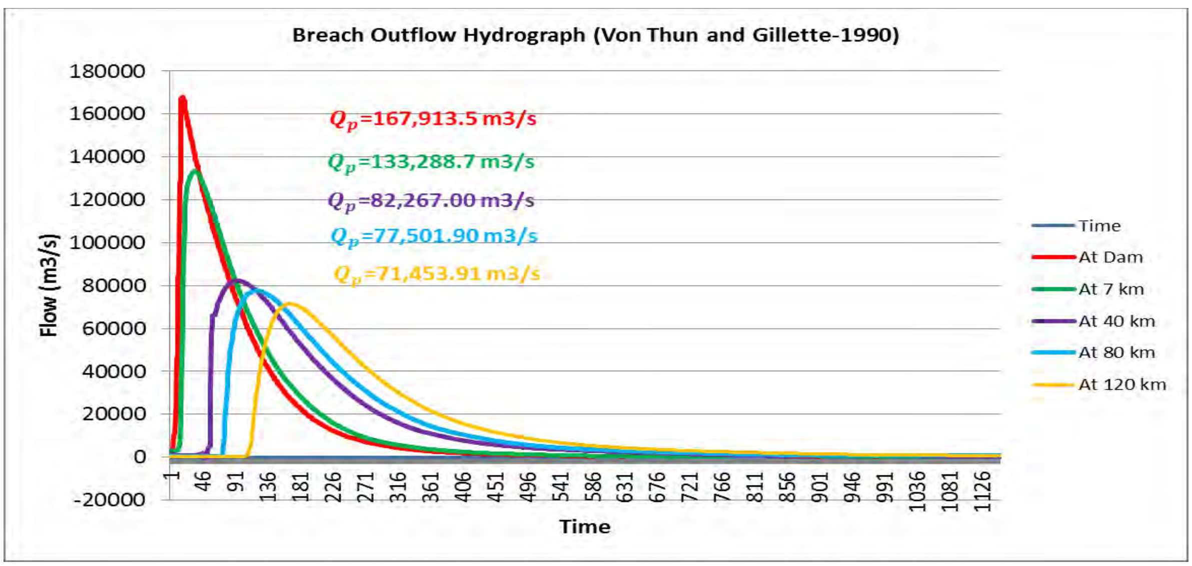


(b) At downstream critical locations (Von Thun and Gillette-1990)

Figure 5: Breach Outflow Hydrographs for Overtopping Mode of Failure (PMF)



(a) At downstream critical locations (Macdonald and Langridge-Monpolis 1984)



(b) At downstream critical locations (Von Thun and Gillette-1990)

Figure 6: Breach Outflow Hydrographs for Piping Mode of Failure (Maximum Pool)

Inundation Area

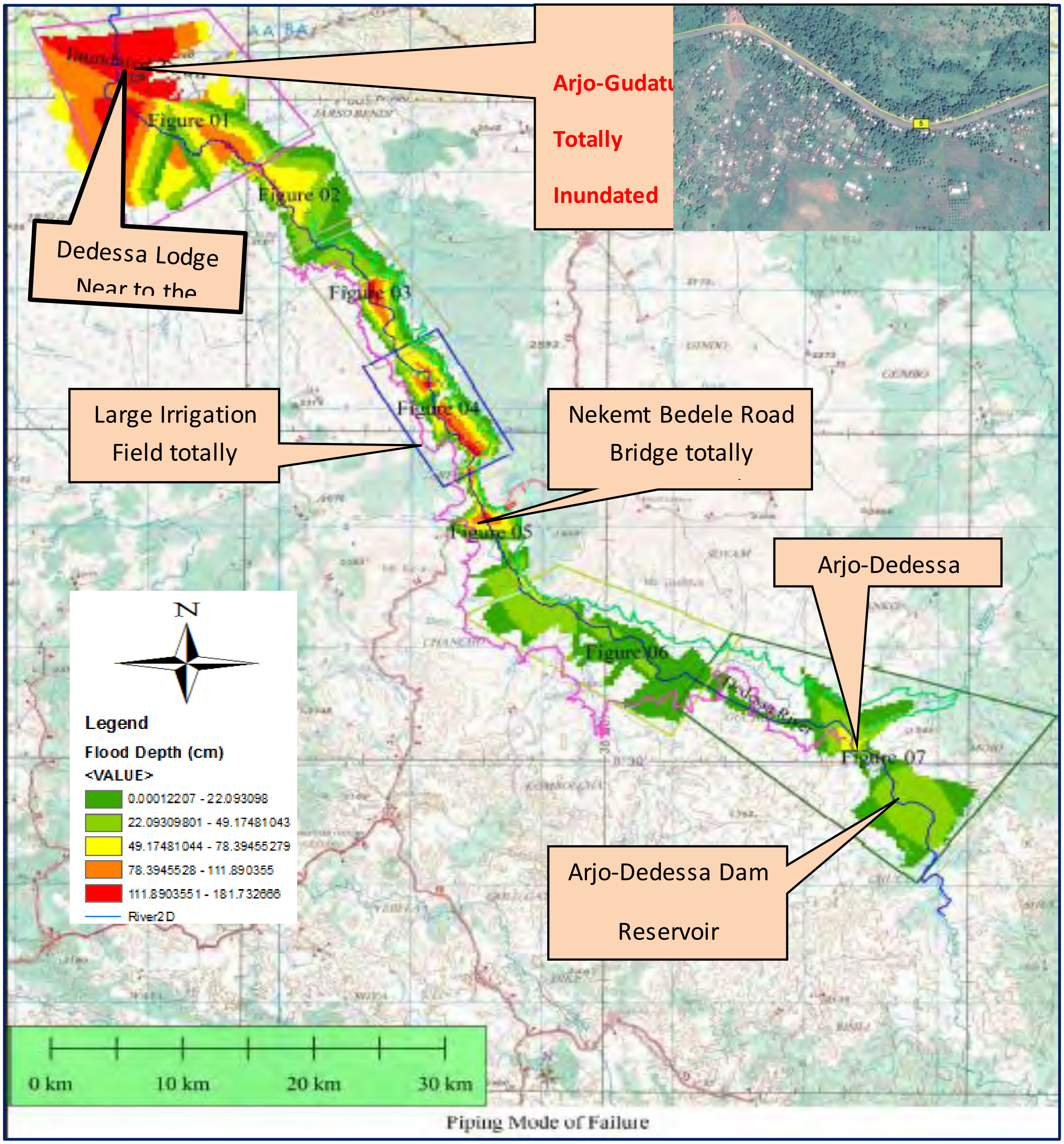


Figure 7: Arjo-Dedessa Dam Breach Outflow Inundation Map

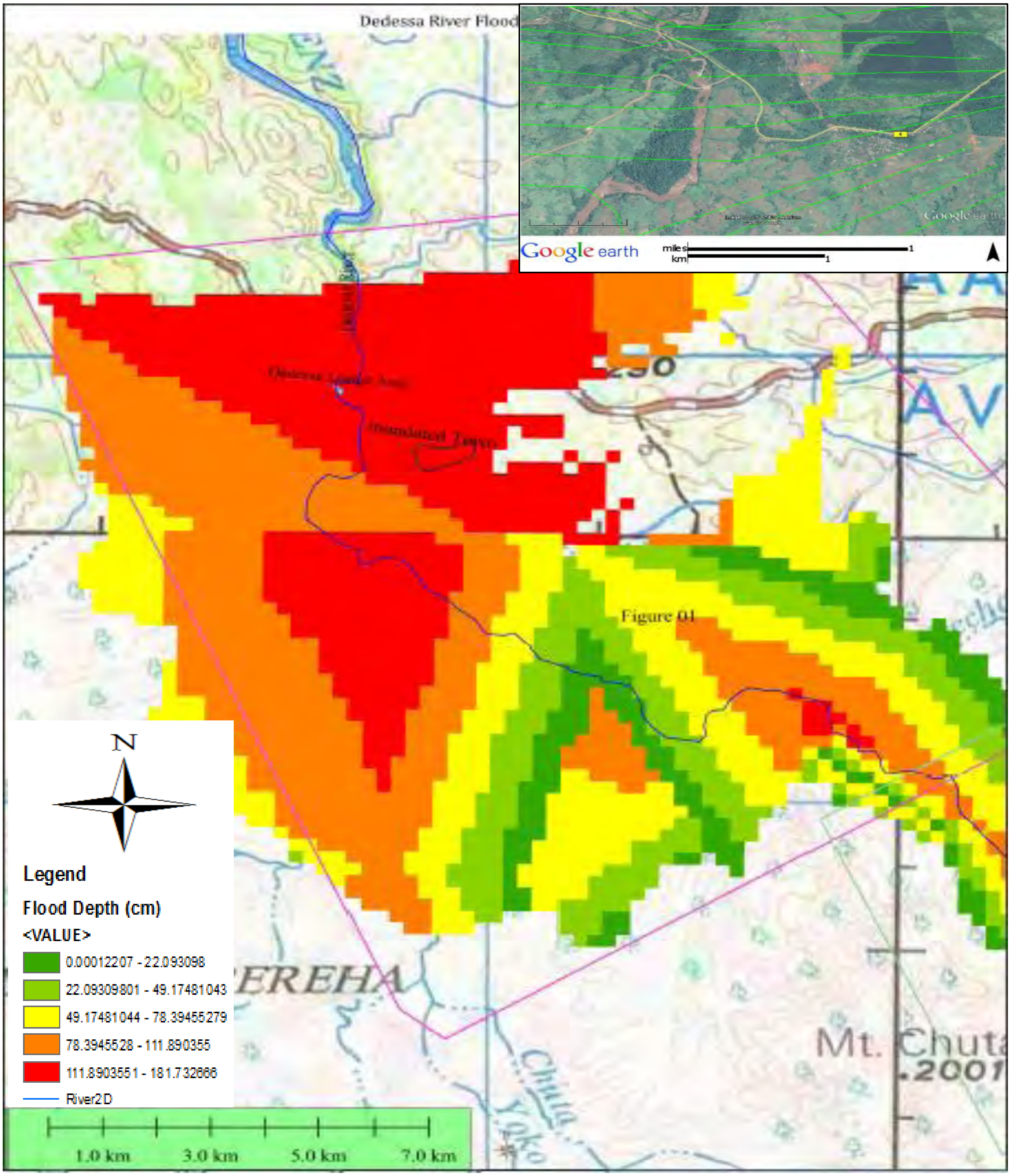


Figure 8: Inundation area one at downstream end (90 km)

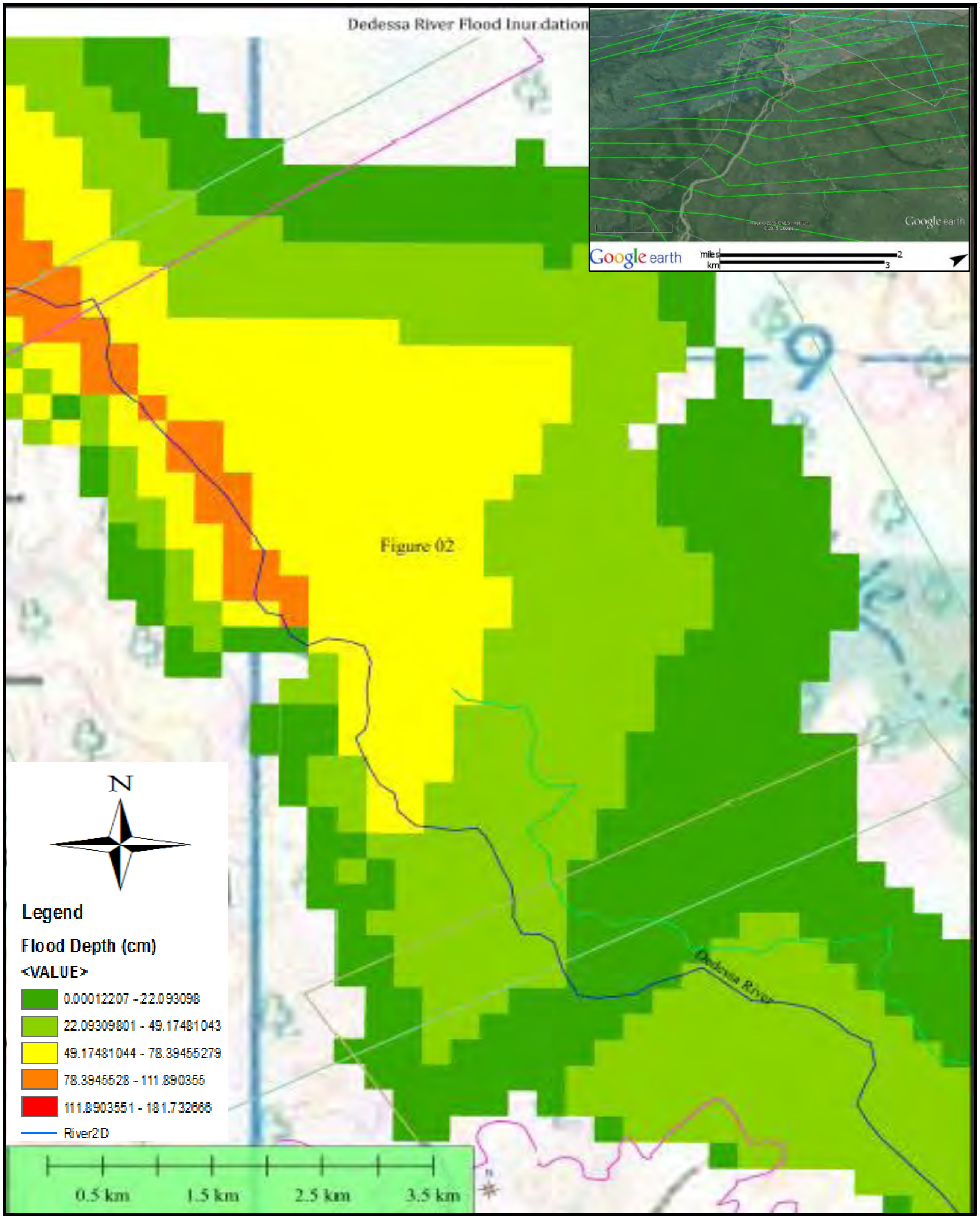


Figure 9: Inundation area two at downstream at 80 km

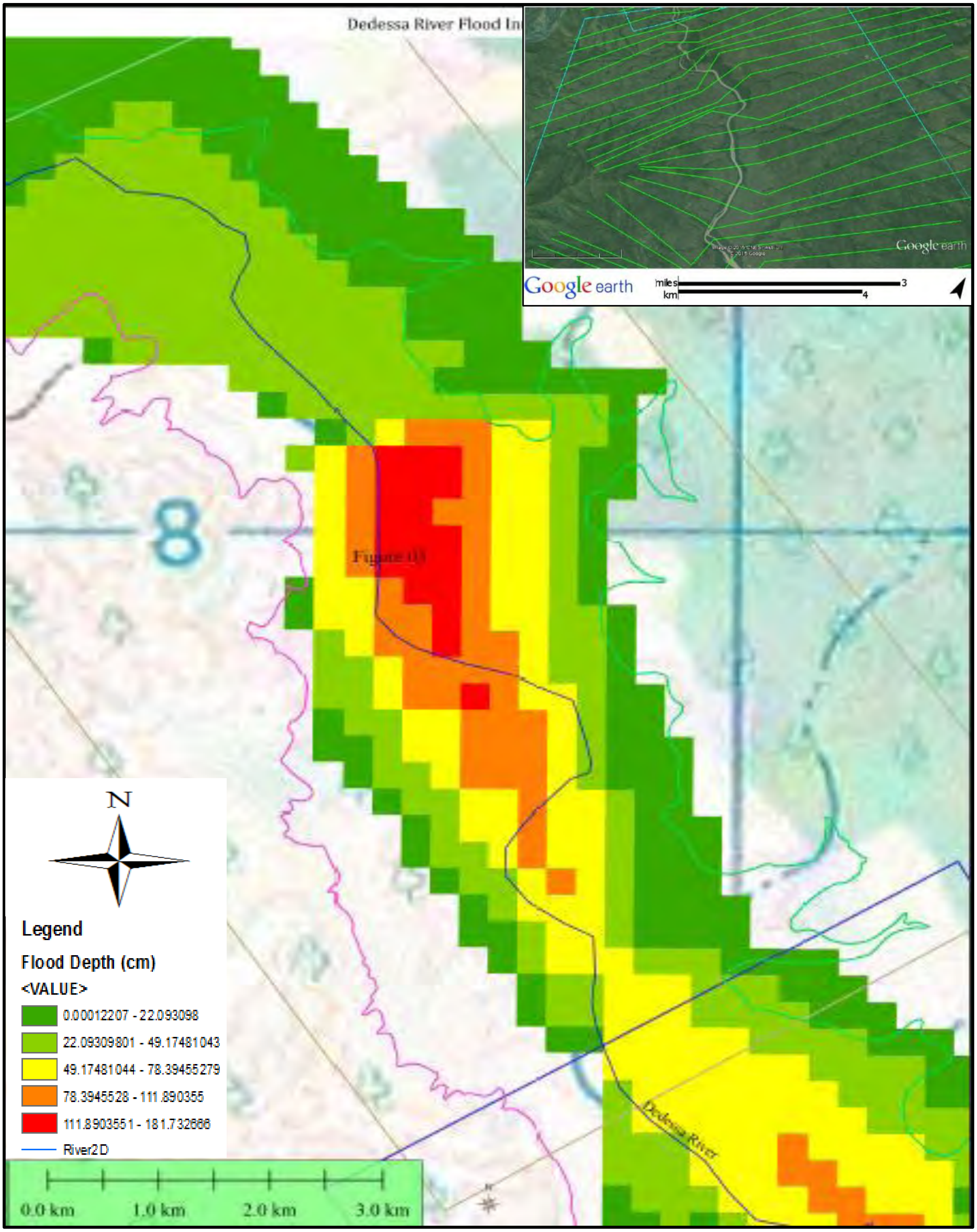


Figure 10: Inundation area three at downstream at 60 km

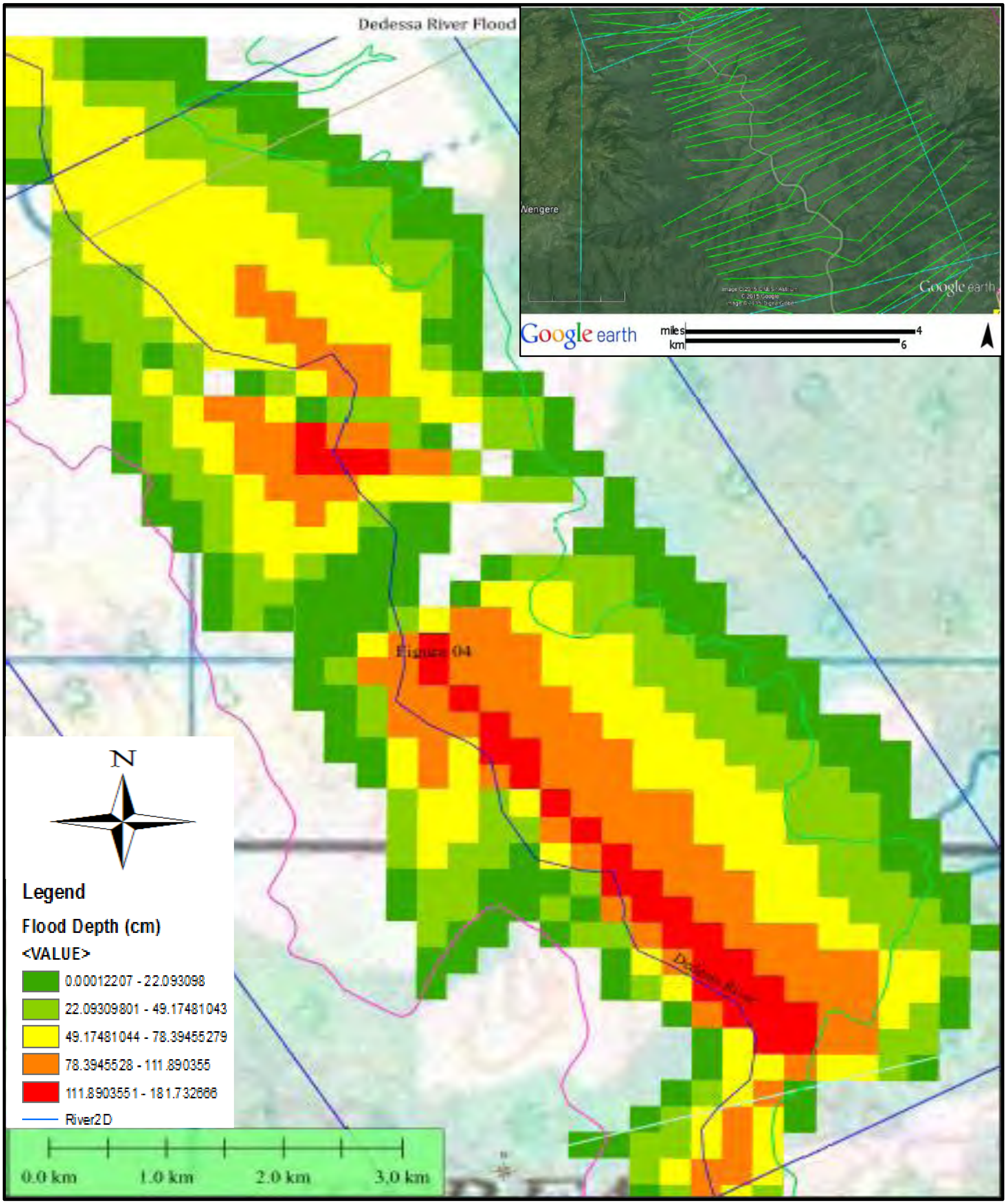


Figure 11: Inundation area three at downstream at between 40 km and 60 km

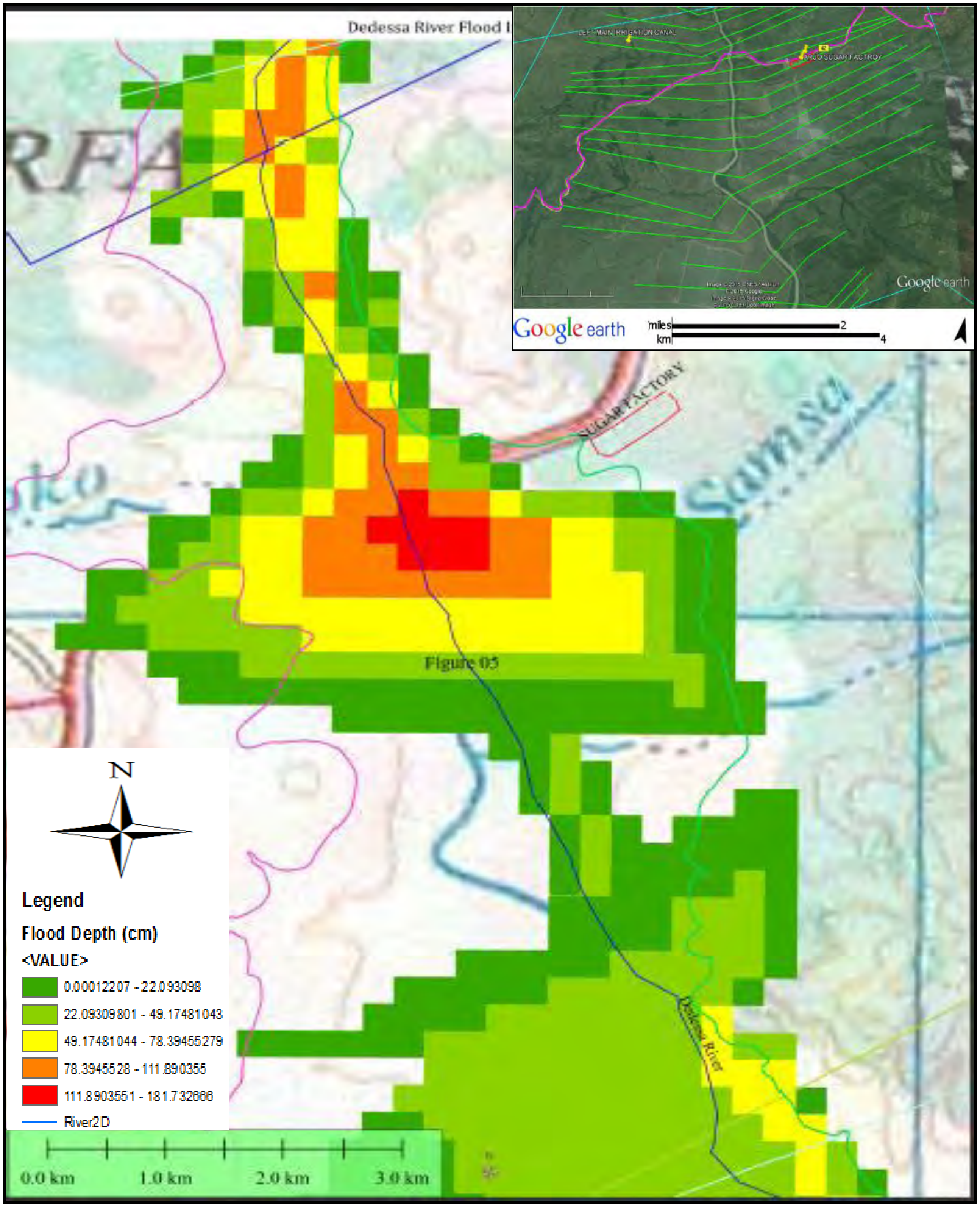


Figure 12: Inundation area four at 40 km downstream at

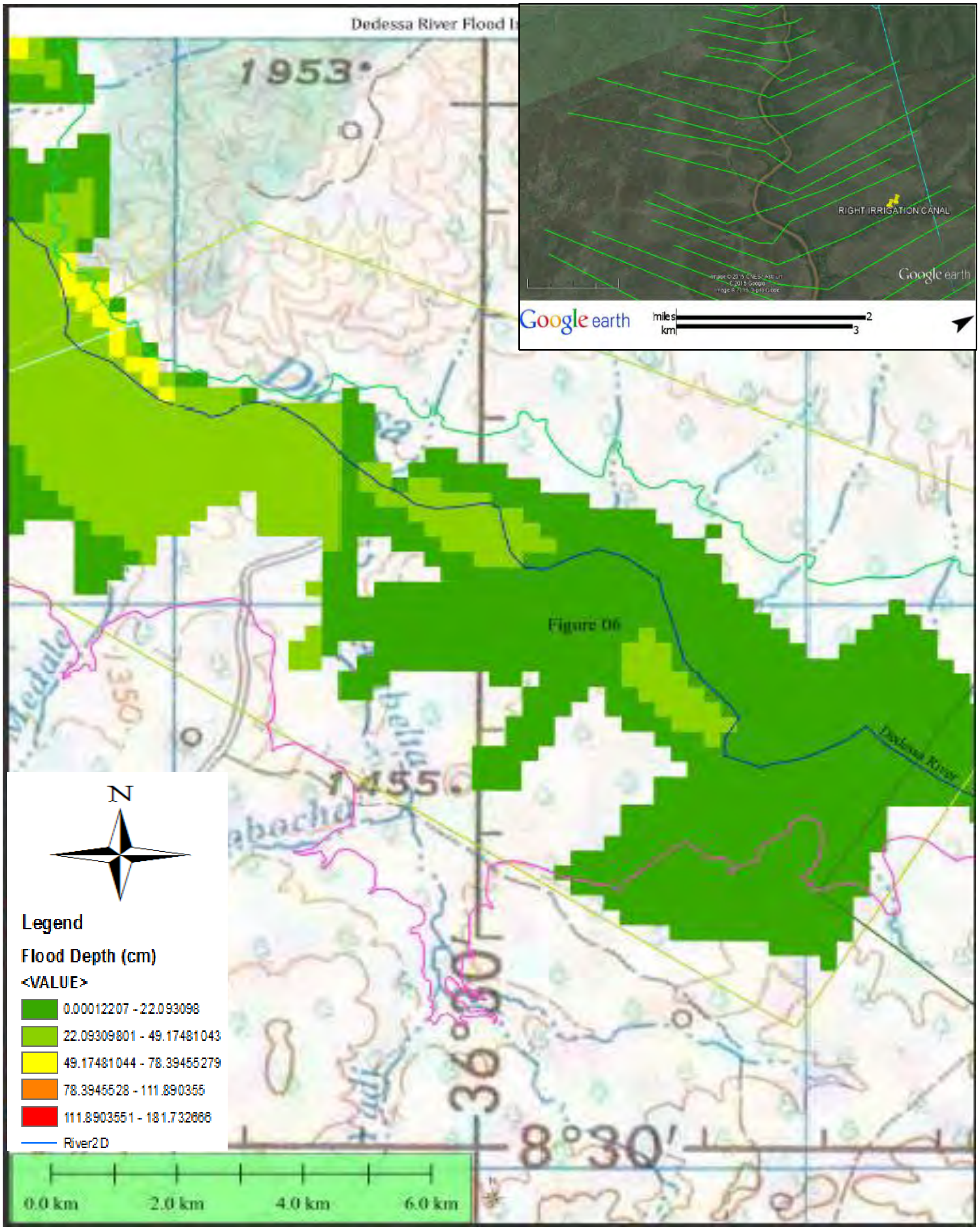


Figure 13: Inundation area five at 20 km downstream

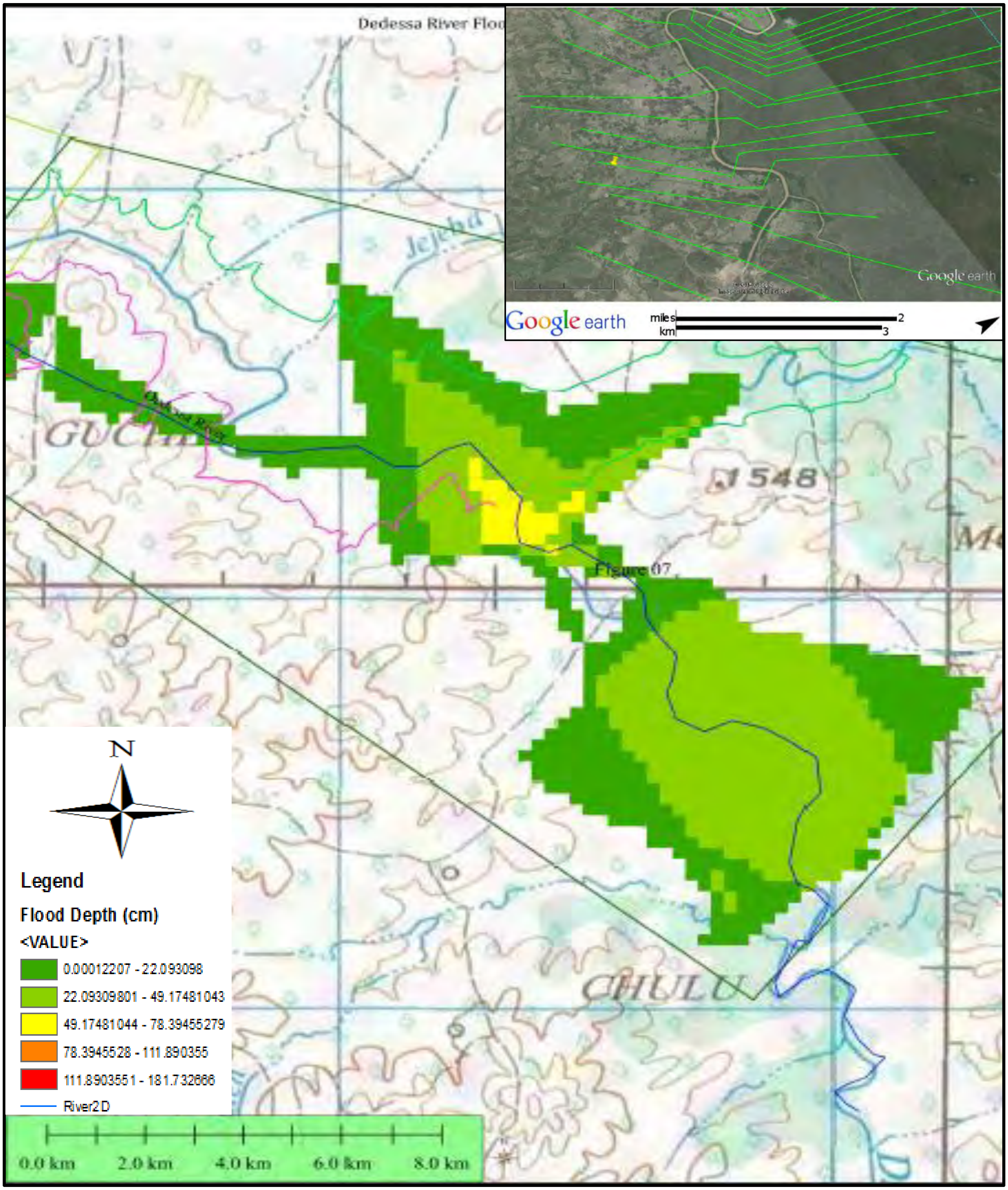


Figure 14: Inundation area six at dam and extends up to 20km downstream